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EARLY AGE SHRINKAGE and CRACKING of NEVADA CONCRETE BRIDGE DECKS

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16. Abstract

The shrinkage and cracking performance of concrete bridge decks at very early age must be minimized in order to provide durable concrete. The main objective of this research project is to develop shrinkage-compensated concrete for Nevada and recommend measures, which might be used to engineer concrete possessing excellent shrinkage behavior as well as precautions that should be taken during construction to minimize ambient effects. To accomplish this objective, a series of concrete mix designs incorporating the usage of commercially available shrinkage reducing admixtures (SRA) and shrinkage compensating cement or additive (SCC/SCA) were developed and evaluated for their suitability to be used in Nevada bridge decks. In this research program, a total of 27 mix designs were prepared using the SRA, the SCC/SCA, fly ash, and various combinations of these admixtures.

Laboratory testing based on several standard and recommended procedures was performed to evaluate the performance of each trial batch. The test program consisted of compressive strength, drying shrinkage, modulus of rupture, chloride ion penetration, cracking tendency, temperature evolution or heat of hydration, and the hardened air-void system.

Results of the experimental program reveal that both the SRA and the SCC/SCA have a significant impact on the early age shrinkage and cracking behavior and performance of concrete, therefore its durability. In order to extract full benefits from these admixtures, NDOT should adopt a shrinkage performance based concrete.

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CHAPTER 1

Introduction

1.1 Problem Statement

Concrete at early-age is a complex material. Critical tasks are performed during the first 72 hours. They comprise concrete mixing at the plant, transportation, placement, finishing, curing, and removal of the formwork. All of these field operations are critically impacted by the properties of fresh concrete. In addition, the long-term durability of concrete is greatly improved if good early-age properties are achieved in the field.[1-8]

Throughout the state of Nevada, the transportation infrastructure has been greatly affected by early age problems of concrete bridge decks. The purpose of the study described in this report was to investigate mix and curing parameters on the performance of concrete, specifically cracking in early age concrete.

1.2 Objectives and Scope

The primary objective of this research was to provide mix design alternative methods and recommendations to the Nevada Department of Transportation (NDOT). These methods and recommendations are designed to reduce early age shrinkage and cracking of concrete bridge decks throughout the state of Nevada. The objectives and scope were to be reached by performing the following tasks:

?? Perform a comprehensive literature review of the existing knowledge and practices related to early shrinkage and cracking of concrete bridge decks.

??

?? Perform field trips and obtain concrete samples of concrete bridge decks both in Northern and Southern Nevada.

??

?? Develop cost-effective, robust, and practical low shrinkage concrete mixture designs incorporating shrinkage-reducing admixtures (SRA) and shrinkage compensating cement additive (SCC/SCA).

??

?? Determine the sensitivity of different concrete mix designs to curing conditions and identify proper curing specifications and requirements.

??

?? Identify appropriate limitations for the use of the concrete mixture designs that include SRA and SCC.

1.3 The National Cooperative Highway Research Program Report 380[9]

The National Highway Research Program (NCHRP) report 380 titled "Transverse Cracking in Newly Constructed Bridge Decks" contains findings of a comprehensive study, which identified and ranked parameters that influenced the occurrence of transverse cracking in newly constructed bridge decks. The NCHRP Report 380 summarized its findings in five sections. These sections contain a literature review, a survey of state transportation agencies, field and analytical studies, a comprehensive guideline section for concrete bridge deck materials and construction, and a suggested test method for cracking tendency of concrete.

1.3.1 Literature Review

The literature search of NCHRP Report 380 indicated that early age shrinkage and cracking of concrete bridge deck was a global problem. It also pointed out factors that significantly affected the problem. These factors included cement content, fresh concrete temperature, heat of hydration, drying shrinkage, and water-cementitious materials ratio. Ambient conditions during construction and poor curing procedures were also discussed in length. The latter was singled out as being the primary cause of deck cracking.

1.3.2 Field and Analytical Studies

A field study[10] was initiated under the NCHRP project. The field study consisted of a monitoring system that measured strains and temperatures of the Portland-Columbia concrete bridge deck and steel girders, and recorded ambient conditions (air temperature, relative humidity, wind speed and direction). Built in 1953, the bridge deck was experiencing severe delamination. In 1991 the Delaware River Joint Toll Bridge Commission (DRJTBC) decided to replace the deck. Construction details of the new deck are available in the NCHRP Project 12-37 Report.[10] Analyses of the data collected from the Portland-Columbia Bridge monitoring system revealed the following:

?? Good agreement was found between time to cracking of cast-in-place concrete ring specimens (cracking tendency test, see section 1.3.4) and visual examinations of the upper surface of the concrete deck for cracking. Correlation was further validated by the analytical studies accompanying the field studies.

??

?? When properly cured (wet burlap and/or plastic), environmental conditions do not affect significantly the thermal stresses in the deck during the first four days after casting.

??

?? Hybrid superstructures (steel girders and concrete deck) undergo significant temperature variations in the deck alone. Hydration affects temperature in the concrete deck. Concrete strains due to temperature were more pronounced above the girder than between girders.

??

?? Elastic equations of stresses in a composite reinforced concrete bridge subjected to uniform and linear temperature and shrinkage conditions yield acceptable estimates of average shrinkage and thermal stresses in a bridge.

?? With material and geometry properties kept constant, a 500 ?s (microstrain) uniform concrete deck shrinkage relative to its girders in a simply supported span in a concrete bridge yield higher tensile stress than that of a steel girder bridge (1.8 ksi versus 1.4 ksi).

??

?? Thermal stresses from early hydration temperatures are largest in concrete bridges when the deck is cast at a different time than the girders are.

??

?? Daily variations in ambient temperature have a more pronounced effect on thermal stresses than seasonal variations in ambient temperature.

??

1.3.3 Survey

The NCHRP Report 380 also included a survey with responses from fifty-two agencies across the United States and Canada, plus four foreign agencies. These foreign agencies included Australia, Denmark, Japan, and Switzerland. Results from the survey analysis indicated a large variation in procedures agencies use in material selection criteria, in fresh and hardened concrete properties tested, and in curing methods. All agencies surveyed identified improper curing, concrete shrinkage, and deflection criteria for design as the three most significant factors that caused cracking in concrete bridge decks.[9]

1.3.4 The Cracking Tendency Test

A methodology to monitor the time-to-cracking is provided by the NCHRP Report 380. This methodology was used to monitor the cracking behavior of thirty-nine concrete mix designs. The following selected findings were drawn from the test:

?? Aggregate type was the most significant parameter that affected time-to-cracking. Four different aggregate types were studied and limestone-aggregate concretes were the least susceptible to cracking. It was also indicated that large aggregate size might provide substantial resistance to shrinkage of concrete. It was suggested to conduct more research on the effect of aggregate size on the overall shrinkage behavior of concrete for bridge decks.

??

?? Shrinkage compensating cement (SCC) effectively minimized surface cracking of the concrete rings and agreed well with transportation agencies studies of the impact of the usage of SCC. Though the initial expansion did not compensate for the drying shrinkage for all the mixes containing SCC, this behavior was attributed to the type and chemistry of the cement used in conjunction with the SCC. Because of mixed results obtained from other researches, it was recommended to further investigate the impact of SCC when used for concrete bridge decks.

?? Low water-cementitious ratio concretes with low cement content yielded better shrinkage behavior and concrete rings cast with these criteria cracked last. However, neither low water-cementitious ratio nor low cement content were convincing factors in time-to-cracking of the concrete rings.

??

- Neither air content nor slump impacted significantly the time-to-cracking of the concrete rings.

- Regardless of girder type and geometry, the modulus of elasticity affected both thermal and drying shrinkage of the concrete rings. Because the modulus of elasticity of the concrete is related to the modulus of elasticity of the aggregates, it was suggested that low modulus aggregates would minimize thermal and shrinkage stresses.

- High creep development during the first month of the concrete life may be produced by properly combining slowly developing mixtures including pozzolans and extended moist curing time. Concrete with high creep at early age, particularly during the first month after casting, would help minimize thermal and drying shrinkage stresses.

- It was suggested to design bridge decks for 90-day compressive strength criteria rather than the widely used 28-day strength.

- Lower heat of hydration and cement type were also factors that affected time-to-cracking of the concrete rings. It was found that concrete mix designs using cement type II showed lower cracking tendency.

- The concrete ring test showed that the addition of 28% fly ash did not affect time-to-cracking. The cracking tendency test also showed that concrete mix designs containing 7.5% of silica fume cracked sooner than mixes without silica fume.

1.3.5 NCHRP Recommendations

NCHRP recommended the usage of the cracking tendency test to identify and specify concrete mixture designs with minimum overall shrinkage performance. It also recommended that the largest possible maximum size aggregate be used. Other recommendations included using cementitious materials with low hydration properties, rapid creep gain, and slow strength development. Usage of High Range Water Reducing agents (HRWR) was also mentioned as to their contribution in providing more compact concrete. The report did not support the addition of silica fume in concrete. It discussed the benefits of providing additional longitudinal steel reinforcement to control deck cracking, with the exception of support locations where construction and congestion issues may arise.

Because ambient conditions during and after casting of the deck were rated as the most contributing factors to shrinkage and cracking problems, NCHRP recommended the following:

- Concrete placement should take place early in the evening (regardless of time of the year). Windy conditions should be avoided as much as possible, unless proper windbreaks are used.

- Maximum air temperature, maximum and minimum fresh concrete temperatures should be specified for every concrete bridge deck construction by the transportation agencies.

?? Curing should include misting, wet blanket, and curing compound. It strongly encouraged curing for 14 days, with a minimum of 7 days.[9]

1.4 Research from U.S. States Department of Transportation

The following section is an overview of studies conducted by the states of Alabama, Colorado, Minnesota, Pennsylvania, and Virginia. Each of these states performed shrinkage and cracking studies of bridge decks.

1.4.1 Alabama[11]

The Alabama Department of Transportation (ALDOT), in collaboration with the Highway Research Center, in Auburn, devised a laboratory experiment to address the issues of drying shrinkage and bridge deck cracking by investigating the effects of silica fume, shrinkage compensating cement (SCC) and shrinkage-reducing admixtures (SRA). The following conclusions were reached from their research:

?? When silica fume was added to SCC, the combination exhibited high drying shrinkage strains, but effectively enhanced the permeability resistance of the concrete,

??

?? The SCC effectively reduced drying shrinkage. However, some problems with rapid slump loss were encountered when concreting took place in hot weather. The SCC did not improve the permeability of concrete, and

??

?? The SRA effectively reduced drying shrinkage, but not as effectively as the SCC did. Set retardation was observed. Air entrainment dosage requirement were higher than for that required for the SCC mix. Compressive and tensile strengths were lower than those of the control mixture. Poor scaling behavior was also noted.

??

The recommendation was to encourage ALDOT to use SCC for concrete bridge decks and to evaluate the recommendation by testing a SCC concrete mix design on a full-scale test deck.

1.4.2 Colorado[12]

The research conducted by the Colorado Department of Transportation (CDOT) and the University of Colorado at Boulder contained three major items.

The first item was to perform an extensive literature review and extract important factors that would reduce concrete bridge deck cracking. Their factors included the use of proper cement type and content, the use of type F fly ash, and the use of silica fume provided proper curing was applied.

The second item was to perform laboratory experiments combining the selected factors and develop new mix designs for use by CDOT. The experimental phase provided the following recommendations accompanying their new mix designs:

?? Exclusive use of type II cement with a maximum content of 279 kg/m^3 (470 lbs/yd^3),
?? For mixes including silica fume, use type F fly ash at a 20% addition rate, especially suited for summer pours,
?? Silica fume content need not exceed 6%,
?? W/C (water to cementitious materials) ratio range of 0.38-0.47, and
?? Air content of 7%.
??

The third item of the report included design factors and construction practices. The design factors consisted of placing equal amount of longitudinal steel reinforcement in the top and bottom of overhangs in regions over the bridge pier, using smaller girders with wider spacing, reducing longitudinal restraints whenever possible, and consider post-tensioning the decks in the transverse direction with unbonded tendons. The recommendations for construction practices included:

?? Restricting cast to an ambient temperature from 7?C (40?F) to 27?C (80?F), maintaining the concrete temperature at or above 10?C (50?F) for the first 72 hours, and limiting the fresh concrete temperature at placement to 27?C (80?F),
?? Restricting cast when the evaporation rate is above 1.0 kg/m^2/hr (0.2 lb./ft.^{2}/hr),
?? Applying fogging for all concrete decks with no delay until the surface has been fully covered by curing compound, and
?? Enforcing a 7-day continuous moist curing when silica fume mix designs are used.

1.4.3 Minnesota[13]

The Minnesota Department of Transportation with the University of Minnesota conducted a study to identify the primary factors causing premature transverse cracking in bridge decks and establish recommendations to reduce transverse cracking. The recommendations are listed below:

?? Use a maximum cement content of 391 kg/m^3 (660 lbs/yd^3) for all mix designs for concrete bridge decks,
?? Use minimum air content of 6.0%,
?? Maximize coarse aggregate content to 1098 kg/m^3 (1850 lbs/yd^3),
?? Deck pour should only take place when the ambient temperature is above 7?C (40?F) and below 32?C (90?F),
?? Not to pour decks when the maximum ambient temperature change is expected to be equal or greater than 10?C (50?F),

?? Use of type II cement,
?? If steel girders are used as support systems, preheat girders to minimize temperature differential during concrete hydration,
?? Increase girder spacing to decrease girder restraint,
?? Avoid specifying decks with thickness of 165 mm (6 ½ in.) and less, and
?? Limit transverse reinforcement steel size to No. 5 at 140 mm (5 ½ in.), and/or No. 6 spaced at 178 mm (7 in.).
??

1.4.4 Pennsylvania[14-15]

The Pennsylvania Department of Transportation joined efforts with a private firm to study the problems associated with concrete bridge deck cracking. The project was divided into two phases.

Phase one focused on field observations of concrete bridge deck construction practices, shrinkage measurements, and subsequent recommendations for preventing detrimental thermal and drying shrinkages that can lead to premature cracking. Phase one recommendations were as follows.

?? A maximum allowable thermal shrinkage should be based on the temperature difference of the deck/beam interface, which was calculated to be 5?C (22?F). This differential should not be exceeded during the first 24 hours after the concrete deck has been poured.
?? A maximum 400 ?s value shall be recorded at the 28-day drying shrinkage test, or a maximum of 700 ?s recorded at 120 days.
?? In hot weather, concrete cover should be provided using wet burlap no later than 30 minutes after surface finishing and texturing.
??

Phase two consisted of experiments conducted in the laboratory where the effects of aggregate and cement source, and fly ash on shrinkage were examined. The results of the laboratory tests led to the following recommendations.

?? Sandstone is highly sensitive to shrinkage (low modulus of elasticity and high compressibility) and was not recommended for concrete bridge decks.
?? Aggregates with relatively high absorption were not recommended.
?? Proper coarse aggregates combinations led to reduced drying shrinkage, noting that the effect of absorption were not clear at that point.
?? Type II cement was recommended for concrete bridge deck applications, based on lower temperature measurements of hydration processes.
?? The addition of Type F fly ash increased the drying shrinkage, but did reduce the heat of hydration of concrete.
??

1.4.5 Virginia[16]

The Virginia Department of Transportation (VDOT) and the Virginia Polytechnic Institute and State University collaborated in this project. The main project accomplishment was the development of an experimental setup that measured drying shrinkage, and capillary pore pressure developments.

The project was successful at developing an experimental setup to capture the capillary pore pressure and shrinkage strains in early-age cementitious mortars. The only recommendation provided under the efforts of this research project was to use the experimental setup to further investigate the effects of factors that influence capillary pore pressure and plastic shrinkage cracking of concrete.

1.5 Concrete Bridge Deck Issues Throughout the World

As reported by the NHCRP, other countries encounter durability issues with their concrete bridge decks. Australia, Denmark, Japan, and Switzerland were the four countries surveyed by the transportation board agency, which noted their remarks in the NCHRP report.[9]

France is another country who has been engaged in fighting the problems caused by premature shrinkage and cracking of their concrete bridge decks.[17-20] The result of their efforts is discussed in more detail in chapter 2.

Switzerland has also spent a lot of resources in providing improved methods to minimize premature shrinkage and/or cracking in their hybrid bridges.[21-22] In more specific ways, it focused on the effects of partial interaction between the concrete deck and the main steel girder beams. They approached the problem through analytical studies, which attempted to define the action-effects in hybrid bridge superstructures when subjected to a variety of actions such as design, service or fatigue load. Their efforts are discussed in chapter 2.

Other significant contributions to the advancement of knowledge and construction practices surrounding the issues of shrinkage and cracking of concrete are ongoing and hold promises for the production of long lasting concrete, especially concrete for bridge decks.[23-31]

1.6 Layout of Report

It is clear that early-age shrinkage and cracking of concrete bridge decks is affecting newly constructed bridges throughout the world. An overview of the NCHRP Report 380 demonstrated the complexity of the problems associated with shrinkage and cracking of concrete bridge decks. Study reports from five States Department of Transportation were presented for review and evaluation. The studies provided recommendations to minimize early-age shrinkage and cracking for concrete bridge decks, which reflected specific needs associated to each State. However, the overwhelming amount of data and research made available both at the domestic and international levels do not answer the specific problems Nevada is facing with its bridge deck cracking problems. This research is designed to address selected issues that affect greatly shrinkage and cracking of bridge decks. These selected issues are drying, thermal, plastic shrinkage and cracking of concrete for bridge deck, and weather effects in Nevada and how these weather effects impact curing conditions and requirements for concrete bridge deck.

To accomplish this objective, an extensive background research was performed and presented in chapter 2. Chapter 3 presents observations made during field trips in selected locations throughout Nevada, results from a questionnaire sent to select U.S. States, and a discussion of current Nevada Standards and Specifications for concrete bridge decks. Chapter 4 describes the process for the selection of materials, mix designs, procedures, and tests used for the experimental program of this research. Chapter 5 presents results and discussions of the tests performed. Chapter 6 addresses the specific issue of weather effects on curing requirements. Chapter 7 proposes a new class of shrinkage compensated concrete as a high performance durable concrete for bridge decks in Nevada, and suggests recommendations as to how the new class of shrinkage compensated concrete should be implemented with appropriate curing requirements for use in bridge decks.

CHAPTER 2

Background on Shrinkage and Cracking

2.1 General

The problems associated with the first three to seven days of the life of concrete bridge decks are critical to the long-term properties of the hardened concrete. Poor construction, conditions, casting and curing, and harsh environmental conditions may all combine to produce undesirable effects such as plastic, drying, thermal shrinkage and eventually cracking of a concrete bridge deck. Other detrimental effects may be poor quality concrete as delivered by ready mix plants, and tightly imposed scheduling. In the U.S. and abroad, transportation agencies and research institutions have dedicated resources and logistics to tackle the issues of early age shrinkage and cracking of concrete bridge decks. In order to examine the extent of the problem of early age shrinkage and cracking of concrete bridge decks, parameters that contribute to the problem need to be identified and described. These parameters include drying and plastic shrinkage, thermal and hydration effects, mineral and chemical admixtures, ambient weather conditions, and aggregate size and type. The description of these parameters consists of their definition, and state-of-the-art reports on the current knowledge and research.

2.2 Drying and Plastic Shrinkage

2.2.1 Drying Shrinkage

Drying shrinkage is primarily an issue related to the cement paste and depends strongly on the amount of water present in the concrete mixture before hardening (plastic state) and remaining after hardening (hardened state).

Raina[32] relates drying shrinkage to strains associated with the moisture loss within the unloaded concrete. Factors affecting drying shrinkage are ambient relative humidity, temperature, wind velocity, and time of exposure.[33-35] The physical significance of drying shrinkage translates to a reduction in volume of the concrete. In a dry and hot ambient environment, both the rate and amount of shrinkage are expected to be greater than under moderate climatic conditions. The net effect of drying shrinkage is a reduction of concrete compressive strength. Drying shrinkage is a long-term process that evolves over weeks, months, and even years.

In the field, the reduction of volume caused by drying shrinkage in turn induces restrained shrinkage against the steel reinforcement present in the concrete. This combined effect of volume reduction and restrained movement under hot and dry ambient conditions is likely to introduce cracking, which must be prevented.

An analytical study[28] dedicated to formulate the evolution of drying shrinkage using micro-mechanical models advocate the capillary tension theory to describe the behavior of drying shrinkage.

The same study pointed out large differences in shrinkage intensity between low water-cementitious (W/C) materials ratios concrete, which showed much lower shrinkage intensity, and ordinary W/C ratios concrete that exhibited larger shrinkage intensity. In addition it observed microcracking behavior as early as 1-½ days after drying started. The studies noted that internal relative humidity of the concrete could not be directly related to drying shrinkage both in its short and long-term development.

Beyea ET al.[36] reported laboratory experiments and results of concrete drying shrinkage intensity with varying W/C ratio (from 0.3 to 0.6) submitted to varying moist curing times (1, 7, 28, and 90 days). They concluded that high W/C ratio concrete exhibited large drying shrinkage regardless of the moist cure time applied, and that low W/C ratio concrete showed a dramatic increase in water tightness (or much lower drying shrinkage) as early as after 7 days of moist cure time. Beyea's studies agreed with statements from the ACI 224 Report[37] which indicated that drying shrinkage increased at a rate of 3 ?s per one pound per cubic yard increase in water content in a typical concrete mix design.

2.2.2 Plastic Shrinkage

In contrast to drying shrinkage, which occurs within the cement paste, plastic shrinkage is associated with the evolution of shrinkage at the concrete surface. According to Nawy[38], plastic shrinkage occurs within the first few hours after placement of the fresh concrete while still in the forms. Concrete bridge decks are more prone to this type of shrinkage because of the relatively high concrete surface exposed to dry air. The moisture evaporation rate of the concrete becomes highly sensitive to the ambient conditions surrounding the concrete and affects the rate at which bleed water rises to compensate for the evaporated water present on the concrete surface.

When cracks initiate, the phenomenon spreads rapidly but is not immediately affecting the structural performance and integrity of the concrete bridge deck. Powers[39] indicates that the crack patterns are strongly affected by the nature of the restraint imposed against contraction of the concrete. Concrete bridge decks that are reinforced by a rectangular grid of reinforcement are likely to exhibit a crack pattern reflecting the restraint of movement imposed by the reinforcement. Raina[32] argues that crack pattern may in part be developed because of settlement of concrete and that these cracks may also extend through the full depth of the concrete deck. When settlement is likely, the concrete will bleed. If settlement of concrete is hampered by the concrete reinforcement or by the formwork, cracking can occur. In general these cracks are longitudinal, following the direction of the reinforcement on the top of the decks. In practice, plastic shrinkage cracking and slump cracking are often quoted as referring to the same event. In fact, both types of plastic shrinkage phenomena are associated with the bleeding of the concrete. A typical form of plastic shrinkage cracks is a series of parallel lines at roughly 45? to the edge of a slab. Alternatively, a random pattern may form, usually referred to as "map cracking" [Figure 2-1]. Plastic cracking can be avoided by attention to the mix design and by avoiding conditions, which may lead to rapid drying during the first hour or so after placing. Steel reinforcement might not help minimize plastic shrinkage cracking, rather that it may be an influencing parameter, when combined with concrete would lead to undesirable surface cracks.

Because plastic shrinkage is closely associated with weather conditions, a more detailed presentation of weather effects on plastic shrinkage is outlined in chapter 6. Chapter 6 also provides measures to prevent plastic shrinkage.

2.3 Effect of External Mechanical Actions

External mechanical actions consist of numerous interactions between the concrete bridge deck and other supporting structural members during its construction phase. These short-term actions include the casting sequence, the weight of the concrete deck section poured and that of the construction team, of the formwork and mobile construction equipment.

The general practice used to minimize the detrimental effects of pouring sequence is usually referred to as the "piano" casting sequence, which is to pour concrete at the pier sections after the mid-span sections. This method is used to achieve minimum cracking in the negative moment zones.

External loading due to construction requirements are usually limited by application of code and standard specifications. The usage of the "piano" casting sequence is generally accompanied by the limitations applied upon the external loading caused by construction requirements. Thus it is general practice to specify limits on the maximum size and weight of the poured section and corresponding formwork, the combined weight of the mobile construction equipment operating on the section in question, and, when applicable, the temporary weight of concrete trucks and accompanying operating equipment if project requirements indicate no other possible way for the trucks to approach the zone to be cast.

Issa ET al[40] showed that the sequence of pour has a significant effect on the deformation of concrete at early ages, the deflections and stresses observed in the fresh concrete due to construction loads and equipment may be significant, and that the dead weight of the superstructure becomes the most critical factor, especially as span lengths increase, more noticeable in composite steel girder and concrete bridge deck systems.

2.4 Hydration Effects on Thermal Shrinkage

Springenschmid[41] writes that temperature differences are frequent causes of cracking. The relative movement resulting from the cooling of structural members (after the peak heat generated by hydration of the cement) is a major factor affecting concrete bridge decks premature cracking. The main issue with thermal shrinkage cracking at an early age is that during the temperature rise phase, the concrete has very low modulus of elasticity because it is hardening. Restrained thermal contractions resulting from the cooling or other temperature changes constitute the main factors creating cracks at early age with structural members such as concrete bridge decks with thickness of 8 inches or more.

Fresh concrete temperature plays a vital role in the evolution of the heat of hydration. High fresh concrete temperature leads to accelerated hydration and temperature rise. It may be important to note that the tensile strength of a concrete hardening at high temperature level is much lower because of the shorter hydration process.[41] Thus, temperature change itself does not cause cracking of concrete; rather the cracking results from stresses that exceed the strength of the material. The relationships between strength development, modulus of elasticity and the coefficients of thermal expansion and contraction are very important to cracking.

Other studies[42-45] indicated that noticeable variations in the cement or clinker quality and composition must be considered to effectively determine the amount of heat liberated at complete hydration of the binding agent. They showed the most influencing parameters affecting the development of the heat of hydration for any particular hardening concrete were the chemical composition of the binder, the fineness and particle size distribution of the binder, the W/C ratio, the initial reaction temperature, and the presence and type of admixtures.

Thermal shrinkage is greatly affected by the coefficient of thermal expansion of concrete, $?_e$. There seems to be a consensus concerning the range the coefficient of thermal expansion for hardened concrete takes (5 to 15 ?s/?C). However this coefficient is not applicable to early age concrete.[46-48] Emborg[49] and Hedlund[50] determined that the temperature coefficient varies at very early ages. Coefficients of thermal expansion and contraction are presented in Table 2-1. By varying the cement type and content, they reported that the coefficient of thermal expansion could go as high as 20 ?s/?C during the first 5-8 hours after casting of the concrete.

Thermal shrinkage is greatly affected by the ambient conditions existing at the time of placement of the concrete. A study performed by Hansen ET al.[51] reported that thermal stress development in concrete placed during high temperature (i.e. T > 30?C or 85?F) is significant during the first days after placement. A conclusion reached in the same study stated that thermal stress development is most severe when the peak reached in the heat of hydration coincides with the maximum ambient temperature.

Other studies[52-55] indicated that it was not possible to predict concrete thermal behavior at early age from apparent hydration properties alone. In situ measurements of thermal properties of early age concrete bridge decks in France revealed that hydration effects (temperature, thermally induced strains/deformations, and thermomecanical properties) are not as pronounced for decks with thickness of 12-inches and less than for decks with higher thicknesses. Furthermore, they showed that as soon as a critical distance of 20-inches and greater between the exposed concrete surface and the cold zone (distance from exposed concrete surface to support system) is reached, a substantial increase of temperature ranging from 30 to 50► C is generally observed and that these effects may not be neglected in the design. This phenomenon is even more pronounced for High Performance Concrete

Moreover, depending on the cement content and type, and the ambient conditions present at the time of casting of a concrete bridge deck, thermal shrinkages can reach significant values up to 500 ?s at the center of the deck when the deck thickness has reached a critical value.[44-45, 50, 55]

Ducret ET al[56] conducted field measurements on a four-span hybrid bridge 219 meters (718 feet) long located in Switzerland. The measurement systems enabled a detailed study of the importance of thermal cracking during the hydration of the concrete deck slabs. The authors focused on the behavior of the deck during the first four weeks after casting of the concrete. They noted the following observations and conclusions.

?? Maximum difference between deck slab and ambient temperatures of 25?C.

?? An increase in stresses due to the imbalance in strains caused by the temperature increases in the deck slab and top flange of the girder.

?? A resultant compressive stress remains in the deck because the elastic modulus is greater during the cooling phase than during the heating phase.

?? Discontinuities in strains measured by the optical fibers during the cooling phase indicated formation of cracks as early as 2 to 3 days after casting of the concrete, subsequently confirmed by visual inspection. It was also indicated that vibrating-wire strain gauges measurements did not coincide with that of the optical fibers revealing that the vibrating-wire strain gauges measured only the effect of temperature.

??

To improve the reliability of their results, the authors went on to conduct simplified numerical analyses using two finite element software programs[57-58] used to study the development of stresses in deck slabs during the heating and cooling phases. The first software numerical studies supported their hypothesis that cracking can be caused by temperature effects during concrete hydration or that the effects of hydration create a stress distribution in the deck slab which increases the probability of subsequent cracking. The second software was used to compare the experimental results with results of the first software numerical analyses. To refine their model of concrete hydration, they used parameters determined by Emborg[49] and Reinhardt ET al[59]. The refinement of the models used to confirm and enhance the reliability of the experimental results led to the derivation of a simplified relationship, which gives the resulting tensile stress of concrete at the end of the cooling phase. The expression derived by Ducret ET al[56] is provided as follows:

$$?_c \; ? \; \frac{? \;\; ?? ^2 \;\; ?? \; T \;\; ?E_s^2 \;\; ??E_{c2} \;\; ? \;\; E_{c1} \; ?}{?? \;\; ?E_s \;\; ? \;\; E_{c2} \; ???? \;\; ?E_s \;\; ? \;\; E_{c1} \; ?} \qquad (2.1)$$

where

$?_c$ is the resulting tensile stress in the concrete,

? is the coefficient of thermal expansion of the concrete,

? is the coefficient of restraint defined as the ratio of the cross-sectional area of the steel beams divided by the cross-sectional area of the concrete slab,

? T	is the maximum difference between ambient and concrete temperature during hydration,

E_s	is the elastic modulus of concrete during the heating phase, and

E_{c1}, E_{c2} are the mean elastic moduli of concrete during the heating and cooling phases respectively.

Implementation of the expression presented by Ducret ET al is summarized in Tables 2-2 and 2-3. The Tables show variations in tensile stresses for two different casting methods and ? values. It is worthy to note that both the maximum difference between ambient and concrete temperature during hydration ? T and the coefficient of restraint ? have significant impact on the evaluation of the tensile stress of concrete at early ages. One of their conclusions was that deck placement using the continuous method leads to tensile forces in the deck which far exceeds the tensile strength of concrete, whether thermal effects are considered or not. In contrast, the piano casting sequence method of concrete placement for deck reduces significantly the tensile stresses in the concrete at early ages. In another study by Ducret and Lebet[60], the following conclusions were drawn.

?? A continuous casting sequence generates high tensile stresses in the intermediate support zones, which are even more pronounced in large span bridges.
?? In comparison with other actions, the most critical tensile stresses are present at the end of the construction and are caused by hydration effects and casting method.
?? Limitations of hydration effects by using low heat cement at relatively low content and the selection of the piano casting method showed significant improvement in the tensile resistance of the concrete deck.
?? The usage of these limitation methods becomes more important with longer spans and when a low probability of cracking with time is specified as summarized in Tables 2-4 through 2-7 .[60-62] The results presented in these Tables were further validated in other studies.[63-64]
??

2.5 Effect of Mineral Admixtures

2.5.1 Silica Fume

Silica fume is a byproduct of the production of silicon metal and ferrosilicon alloys. Silica fume consists of amorphous silicon dioxide (SiO_2). Its extremely small size and the high content of SiO_2 cause the silica fume to be very reactive when used in concrete applications. Very high strength concrete and high resistance to chloride ion penetration can be obtained using silica fume. Though the addition of silica fume enhances some mechanical properties of concrete, recent applications of silica fume in concrete bridge decks and subsequent deck cracking have caused the engineering community to review the applications of silica fume in concrete for bridge decks.

Early age cracking problems associated with the use of silica fume prompted the National Cooperative Highway Research Program (NCHRP) to initiate further research and experimentation and generate practical and improved recommendations for the application of silica fume for concrete bridge decks.[65] The authors of the report noted the following important conclusions: 1) cracking tendency of concrete is influenced by the addition of silica fume only when the concrete is improperly cured, 2) the early age shrinkage behavior of concrete containing silica fume is higher than that of conventional concrete and is extremely sensitive to the W/C ratio, and 3) addition of silica fume in the range of 5 to 8% reduces chloride diffusivity, increases the compressive strength and elastic modulus of concrete and that beyond these addition rates no additional benefits are noted. In addition to the conclusions drawn from the NCHRP report, Pinto et al[66] reported in their study that silica fume accelerated early cement hydration, affected both the rate at which heat was liberated and the total heat released. Other studies[67-68] reported similar conclusions.

In contrast, Alsayed[69] investigated the effect of silica fume in hot and dry field conditions. His findings indicated that silica fume addition rate of 10% (by weight of cement) greatly reduced the 3 year drying shrinkage, the rate of drying shrinkage in the first month of concrete, and the influence of curing conditions on the rate of drying shrinkage of concrete.

Folliard and Berke[70] and Folliard ET al[71] showed that silica fume, when properly combined with either superplasticizers (HRWR), or shrinkage-reducing admixtures (SRA), or shrinkage-compensating cement (SCC), reduced considerably the drying shrinkage and increased the chloride ion penetration resistance of concrete. They also showed that a proper combination of silica fume with SCC reduced the restrained expansion of concrete.

Cement contains calcium hydroxide ($Ca(OH)_2$). During the hydration process, the silica fume reacts not only with water but also with the $Ca(OH)_2$, which is consumed in most part. The reaction of silica fume with $Ca(OH)_2$ causes higher heat of hydration as silica fume and $Ca(OH)_2$ contents are increased, thus inducing considerable thermal stresses. Though NCHRP reported that silica fume did not affect cracking significantly as much as proper 7-day moist curing was provided, it must be emphasized that ambient conditions, construction scheduling, and/or structure geometry affecting the restraint conditions, are not accounted for in the results of NHCRP. Thus, real field conditions and economic constraints are more often imposing on contractors and engineers less than proper conditions for appropriate application of silica fume for concrete bridge decks.

Though silica fume enhances considerably durability parameters of concrete, the lack of qualitative information relating silica fume to early age cracking and the disagreement in studies mentioned above do not facilitate the production of specifications for the application of silica fume in the concrete industry, more notably that of concrete for bridge decks.

2.5.2 Fly Ash

Types F and C fly ashes are by-products of the combustion of coal in large power plants. Type F fly ash is generally low in lime and contains high content of combined silica, alumina, and iron. It effectively moderates heat gain during concrete curing and is considered an ideal replacement for cement in concrete applications. In contrast, type C fly ash is mostly used for early high strength concrete applications.[42]

Fly ash reacts with water and the $Ca(OH)_2$ released from the hydration of Tricalcium Silicate (C_3S). The reaction produces Calcium Silicate hydrates, the primary binder of the cementitious agent in concrete. This chemical reaction ensures that $Ca(OH)_2$ crystals do not spread to form micro cracks, which in turns enhances the concrete resistance to chloride ion penetration.[73]

Fly ash replacement of cement for concrete application produces many desirable effects. These effects include enhanced workability of fresh concrete, lower heat of hydration as well as lower hydration rate, slight increase in the resistance to chloride ion penetration, and improvement in the control of alkali-silica reaction (ASR).[74-77] For example, Shehata ET al[76] indicated that in all cases, increasing the level of replacement of a particular fly ash reduced expansion, pointing out that the minimum level of replacement required to control expansion to ? 0.04% at 2 years generally increased as the calcium or alkali content of the fly ash increased or as its silica content decreased. Thus, fly ash with higher alkali or calcium contents are less effective in controlling expansion due to ASR. It is of particular interest to Nevada concrete producers as Nevada aggregates have very poor resistance to ASR expansion, which lead to cracking of concrete bridge decks over a long period of time.

A constituent of fly ash, which produces undesirable effects, is carbon. Carbon is not a desirable component of fly ash but the efficiency of current plants do not accommodate for a total elimination of carbon in the fly ash. Today, with improved technology to extract carbon, fly ash plants produce fly ash with 1% carbon or less (ASTM C 618[78] limits the amount of carbon in fly ash by the loss of ignition to 6%). It has been shown that high carbon content (i.e. > 1%) can negatively affect air entrainment in concrete, chemical admixture dosages, and aesthetics of concrete.

2.6 Effect of Water Reducing Admixtures

In 1962, the American Society for Testing and Materials (ASTM) published the first standard specification for chemical admixtures for concrete. An updated version of the specification was published in 1990. Only two years later another edition of ASTM C 494-92 was published.[79] Advances in technology and chemistry have allowed the community interested in the utilization of chemical admixtures in concrete technology to work towards formulating revolutionary products. For this particular reason and others, the committee overseeing the use of chemical admixtures must keep up with the ongoing progress in the making and refinement of these products. It is no surprise that only two years passed between the currently used specification to that published previously. Since 1992, there has been a renewed interest in the study of these chemical admixtures and their effect on fresh and hardened properties of concrete. Many mixes have been subjected to various scenarios in the attempt to capture the impact of chemical admixtures in field applications. Because the discussion is restricted to early age properties of concrete bridge deck, care has been taken to focus on the effects of these admixtures on shrinkage and cracking of concrete.

Plasticizers (WR) and superplasticizers (HRWR) are organic admixtures that have the property of either increasing the fluidity of a fresh mixture at constant water dosage, or allowing a reduction of the water dosage at constant fluidity. The active molecules are generally one or a combination of the following categories:[80-81]

- Polymelamine sulphonate formaldehyde (melamine superplasticizers),
- Polynaphtalene sulphonate formaldehyde (naphthalene superplasticizers),
- Lignosulfonate, and/or
- Polyacrylates.

Some reports recognized the novelty in the understanding of the interaction between superplasticizers and cement phases and why some superplasticizers work better with some cements.[82-85] For a long time it was believed that the superplasticizer/cement interaction was only physical in nature.[82] Then other studies used a chemical approach to tackle the issue and discovered that superplasticizers interfered with cement hydration and sulfate dissolution.[83] The sulfate dissolution causes an unexpected loss of fluidity at the construction site even when the concrete is right out of the batching plant. This loss of fluidity may in turn affect the drying process of the concrete, and drive increased usage of superplasticizer at the construction site, thus increasing the effective cost of concrete. This interference is successfully prevented by adjusting the composition of equivalent Sodium Oxide (Na_2O) of the cement. This adjustment dramatically improves the superplasticizer and cement interaction during the mixing process, and reduces considerably the dosage of superplasticizer.[85] What is well understood today about the interaction of superplasticizers and cement is the following:

- For a given Portland Cement, the dosage rate of superplasticizer for a desired fluidity increases with the specific area of the cement;
- As the superplasticizer dosage is increased, a delay in the development of the heat of hydration is observed both in the laboratory and the field;[86] and
- The solubility rate of calcium sulfates is never constant in any case and can be strongly modified in the presence of superplasticizers.[86]

According to ASTM C 494-92, water-reducing admixtures are classified in two categories F (for superplasticizers, i.e. HRWR) and G (for plasticizers, i.e. WR). A standard procedure is provided to determine which classification the water-reducing admixture falls under. ASTM C 494 provides a Table to test the superplasticizer with a specified maximum water content equal to 88% of the control. The superplasticizer must be tested with a coarse aggregate meeting the grading requirement of ASTM C 33 for a No. 57 size. In order to obtain an 88 ? 12 mm (3.5 to 4 inches) slump concrete, an average water of 175 l/m^3 (295 lbs/yd^3) shall be used for air-entrained concrete. For a concrete with cement content of 307 kg/m^3 (517 lbs/yd^3) this would correspond to a W/C ratio of 0.57 for air-entrained concrete. To achieve the required 12% reduction in water, the use of superplasticizer would yield a W/C ratio of 0.50 for air-entrained concrete. Thus compliance with ASTM C 150-95a[87], ASTM C 494-92, and ACI 212.4R-93[88] does not imply or warranty that compatibility of cements and superplasticizers at low W/C ratios is met, and that premature slump loss or excessive retardation is avoided. Premature slump loss and/or excessive retardation can have disastrous effects on the early age properties of concrete such as unexpected high rates of drying shrinkage which can lead to cracking.

As pointed out earlier, some studies[66, 69] focused on the usage of superplasticizers to reduce the heat of hydration of cement and the rate at which the heat was liberated. These studies concluded that at high level of superplasticizers, set retardation may take place, and that a well defined combination of silica fume and superplasticizer would address the issue of drying shrinkage and minimize it regardless of ambient conditions. However it was cautioned not to assume that absolute silica fume and superplasticizer combinations would work to minimize early age shrinkage and cracking of concrete. It was also shown that superplasticizers, provided that initial moist curing is specified, reduced the total pore volume and refined the pore structure of concrete.[89]

It becomes evident that in order to acquire substantial information for decision in any particular project, it would become necessary to perform laboratory trials. Thus laboratory tests with the objective to retrieve optimum practical recommendations for the use of superplasticizers should be performed to address project specific issues, as long as these tests are included in standard specifications for concrete bridge decks.

2.7 Effect of Curing in Ambient Conditions

It is widely accepted that the most important environmental factors affecting the early age shrinkage and cracking behavior of concrete bridge decks are ambient temperature, relative humidity, and wind velocity. For this reason the American Concrete Institute developed practical and suitable guidelines for minimizing the effect of ambient temperature, relative humidity, and wind speed in hot weather.[33] Furthermore, ACI provided prediction equations for creep, shrinkage, and temperature effects in concrete structures based on very simplified assumptions for design purposes.[90]

Research regarding the short and long-term effect of curing in arbitrary ambient conditions is scarce.[33, 91 - 94] Yet it is being widely recognized that to properly engage in a successful attempt to address the issues of early age shrinkage and cracking of concrete bridge deck, agencies concerned about this problem should produce state specific guidelines based on ACI 305 and ACI 306[94] recommendations for hot and cold weather concreting, respectively. For example, the State of Alabama commissioned a research institution to develop specific bridge deck curing requirements.[93]

An independent study conducted by Holt[95] in Finland provided further evidence of early-age related problems linked to curing practices and environmental conditions. The study used an early-age shrinkage testing arrangement developed in Finland. The test consisted of measuring early-age horizontal and vertical shrinkages, settlement, capillary pressure, temperature, and evaporation of a concrete slab with dimensions 270 x 270 x 100 mm (11 x 11 x 4 in). It must be emphasized that the conclusions presented here are associated with tests performed with type III cement (rapid setting cement at 300 kg/m^3 (506 lbs/yd^3)). The project focused on the contribution of imposed ambient conditions, cement type and content, and admixture type and dosage. They reached the following conclusions.

?? That "early-age" would usually refer to the first 12 hours of concrete immediately after casting as critical changes take place during that period of time.

??

?? When precautions were taken to prevent wind to reach the concrete slab during the first 12 hours, the recorded shrinkage magnitude at 3-months was virtually equal to that recorded +after the first 24-hours of the concrete life. However when wind was allowed to run on the concrete surface, the recorded early-age shrinkage was sevenfold that of the long-term shrinkage.

?? That the rate of evaporation and the evaporation magnitude of the concrete slab were still considerable even after the first 6 hours of the concrete life, noting a direct correlation between evaporation magnitude and increasing wind speed.

??

?? As the wind speed was increased, the amount of curing compound was also increased to prevent shrinkage to rise within the first 12 hours.

??

Similar research was conducted in 1997 by Haugaard[96] in the Netherlands through the Danish Road Directorate Research Center to evaluate the current curing practices under arbitrary ambient conditions. It was confirmed that dramatic improvement in the shrinkage behavior of concrete slab occurred when curing compound was properly applied early enough to prevent the occurrence of potential cracking due to shrinkage.

Many countries, if not all, agree that ambient conditions are critical in the early age life of an exposed concrete structure such as a bridge deck. It is also agreed that these conditions are site specific. However, there seems to be some resentment in clearly establishing a universal definition for the term "early-age" for concrete. Consequently, very few projects provide adequate project specific and weather related curing requirements that would prevent shrinkage and cracking of concrete bridge decks. Yet while the contractors satisfy minimum requirements as specified by the transportation agencies, as is the case in the U.S., perhaps early-age should be clearly defined in the specifications and provisions subsequently added towards a classification of environmental exposures. This classification could include requirements that would ensure appropriate hardening process, adequate early freezing resistance after the hardening process (if concrete pour takes place in the winter), and protection against thermal and drying stresses. Because of the lack of existing curing requirement specifications related to weather effects in Nevada, chapter 6 will address this issue.

2.8 Effect of Aggregate Size and Type

Aggregates are characterized by their size, shape, elastic properties, and geomorphologic description. Combined coarse and fine aggregates generally constitute two-third of any unit volume of concrete. Because they occupy most of the concrete volume, it is important to clearly establish the place and importance of their effect in the early age properties of concrete. It is also vital to understand the aggregates contribution to shrinkage and cracking of concrete.

Most of the current state-of-the-art in the U.S. is the outcome of early work accomplished by Fuller and Thompson.[97] In Europe and particularly France, earlier work by Bolomey[98], Caquot[99], and Faury[100] outlined the foundation and basis for aggregate proportioning and optimization in the making of normal concrete.

While the selection of particularly strong aggregates is not necessary in the production of concrete, the growing acceptance of high performance concrete dictates that it becomes necessary to utilize aggregates with acceptable level of modulus of elasticity and bond resistance with the surrounding cementitious paste, as the aggregates should not become the weak link in the concrete matrix.[30] Thus if the aggregate-paste interface strength is weakened because of the relative smoothness of the aggregates texture, then micro-cracking occurs during the critical early-age of concrete or during the drying period in which the shrinkage rate is fastest. Furthermore, if during service tensile or shear stresses micro cracking occurs, it may well be caused because of the difference in modulus of elasticity between the aggregates and the binding paste.[30]

One of the most notable recommendations provided under NCHRP report 380 was to allow the largest possible aggregate size without being larger than one-third the deck thickness, or three-quarter of the minimum clear spacing between bars, adding that aggregates for bridge decks should be at least 40 mm (1 ½ in).

However, De Larrard[30] added that coarse aggregates are generally limited from one-fifth to one-third of the pipe diameter when concrete is pumped. He further discussed problems affecting the concrete paste thickness when the maximum size of coarse aggregates was increased, even though a lowering of water demand was noted. For a long time it was assumed that a reduction of water demand would increase the compressive strength of concrete at constant W/C ratio, but it was shown and is now widely accepted that there is an optimal maximal aggregate size beyond which increasing the coarse aggregate size will not improve hardened properties of concrete.

Another problem, which is familiar to the Nevada Department of Transportation, is the Alkali-Silica Reaction (ASR) expansion of concrete. The chemical reaction between the cement alkalis and siliceous agents in aggregates cause an alkali/silica gel to form. When this gel is in contact with moisture, expansion occurs creating internal stresses in the concrete that eventually lead to disastrous cracking.

A study by Zhang ET al[101] clearly showed that aggregate size and aggregate size grading affect the Alkali-Silica Reaction (ASR) expansion of concrete. It was determined that the smaller siliceous aggregates were, the greater the expansion of ASR. As the siliceous aggregate size was increased, it was observed that the cement/aggregate ratio that reaches the maximum expansion decreased. Larger aggregates effectively reduced ASR expansion at early-age, but that the ASR expansion continued to increase at later ages.

Aggregates with poor texture, with irregular shape or poorly crushed, low modulus of elasticity, and high silica mineral content or of any combination of these characteristics may lead to poor shrinkage behavior and potential cracking, especially when used for concrete bridge decks.

2.9 Effect of Shrinkage Compensating Cement (SCC/SCA)

Krauss and Rogalla[9] reported that shrinkage-compensating cement (SCC) and/or shrinkage-compensating additive (SCA) was gaining popularity in the construction industry notwithstanding the workability issues surrounding the application of SCC/SCA concrete. SCC/SCA is different from regular Portland cement in that it stimulates a moist-sensitive volume expansion (caused by the formation of ettringite crystals) of the concrete during its critical hardening process immediately after setting of the cementitious paste. This expansive property alters the tensile and compressive stresses of the steel reinforcement and concrete respectively. While the alteration is taking place, drying shrinkage of the concrete is also causing stresses. The combined action of the expansive process and the drying shrinkage minimize shrinkage levels and the potential for cracking of concrete at very early age.[102]

Though the effectiveness of SCC/SCA has been experimentally shown and successful bridge deck applications have been reported, Krauss and Rogala[9] made it clear that a level of discomfort rested upon the construction industry using SCC/SCA. Construction and reinforcement issues may have caused this discomfort, which included rapid slump loss, difficulty in surface finishing, and unidentified problems related to the interaction between the steel reinforcement and the expansion process of SCC/SCA concrete.[9, 102]

Research adding to a better understanding of the mechanism of hydration and microstructure of SCC/SCA and the subsequent impact on composite behavior of concrete and its steel reinforcement is virtually non-existent. An experiment conducted by Shuguang and Yue[103] showed the following:

?? The combination of fly ash and SCC/SCA for concrete application provided better porosity properties, i.e. smaller and more homogenous porosity, provided that proper curing was applied,
??
?? The paste-aggregate interface of fly ash and SCC/SCA concrete showed increased bonding effectiveness, i.e. an improved intensification and densification of the interface microstructure, which reduced potential for cracking, provided that proper curing was applied.
??

Thus it is worthy to note that SCC/SCA concrete show considerable improvement in the areas of shrinkage and cracking reduction, in the hardened properties of concrete vital to durability considerations, provided that proper proportioning and curing are applied.

2.10 Effect of Shrinkage Reducing Admixture (SRA)

Shrinkage-reducing admixtures (SRA) are not covered by any specifications in the available Standards. SRA was first developed in Japan.[104-105] Berke ET al[106] patented the product in the U.S. in

1996. Since its development, the construction industry has been anxiously engaged in applying this innovative admixture to concrete to reduce drying and thermal shrinkages, complying with ASTM C 494-92.[79] Many versions of SRAs are commercially available.

The SRA functions as a reducer of capillary tension that develops within the concrete pores as it dries. It can be applied in two ways. One is to simply spray it on top of the concrete surface, called the impregnation method or topical application. The second method is to integrate it in the mix during the mixing of concrete separately from any other admixtures. It has been found that the integration method provides much better results in reducing drying shrinkage.

As early as 1982, Shah ET al[107] studied the impact of SRA on restrained shrinkage cracking and other hardened properties of concrete. Their experiment was based on the usage of the steel ring test or restrained shrinkage ring as described in Chapter 1. They found that SRA reduced slightly the compressive and tensile strength of concrete, and that the variations observed in the strengths were attributed to curing length and other admixtures used. They also found that SRA considerably reduced the free shrinkage, the macro pores, and crack width of concrete, while increasing workability of concrete.

In 1997, Folliard and Berke[70] also found similar results with that of Shah et al. Folliard and Berke used the same tests setup and indicated that, 1) drying shrinkage and restrained shrinkage cracking were dramatically reduced, notwithstanding the short curing applied (24-hr moist curing), noting however that longer moist curing should be applied to improve the effects of SRA.

While the benefits of the application of SRA in concrete technology have been clearly demonstrated in field applications[108], there have been mixed findings concerning the effect of SRA on scaling and freeze-thaw resistance of concrete.[109]

The SRA provides undisputable benefits for early age shrinkage and cracking minimization for concrete bridge deck applications in environments not affected by freezing and thawing.

It was stated earlier that the SRA reduces the capillary tension of the concrete pores. The capillary tension is a function of the pore size, the angle of the menisci in the pore, and the stress state at the menisci-pore wall interface. A reduction in the capillary tension will cause the pore size to decrease thus inducing a large stress state. The larger stress state the reduction in pore size create a situation that lead to inadequate dispersion of air bubbles as the pore size falls below the size of the air bubbles. In addition the large stress state in the pore and at the menisci act as a confining compartment that inhibit air bubbles to move in the cement paste and create acceptable spacing factors. Because of these mechanisms, the introduction of an SRA in concrete that will be exposed to freeze-thaw cycles and its effect on the hardened air-system of the concrete should always be considered.

2.11 Summary

It is clear that early-age shrinkage and cracking of concrete bridge decks is affecting newly constructed bridges throughout the world. Drying, plastic, and thermal shrinkages were described and general methods to minimize these phenomena were presented. A discussion on the effects of external mechanical actions reveals that adequate considerations for the constraints imposed by the deck support system, construction methods, scheduling, and economic restrictions can effectively reduce undesirable secondary effects on the early-age behavior of concrete bridge decks. A detailed review of the effects cementitious materials showed that cement type and content, fly ash, silica fume, and shrinkage-compensating cement or additive (SCC/SCA) may be beneficial or detrimental to reducing problems associated with shrinkage and cracking of concrete. Many studies indicated that chemical and mineral admixtures used in combination, and/or in conjunction with cementitious materials, may or may not effectively reduce early-age shrinkage and cracking of concrete for bridge decks. Superplasticizers were shown to delay the peak heat of hydration and porosity of concrete. They were also shown to enhance the mechanical properties of concrete, with or without silica fume. Proper curing methods accompanied with adequate curing time were continuously showed to improve shrinkage and cracking of concrete, sometimes even virtually eliminating early-age drying, and plastic, and thermal shrinkages. Moreover, it was shown that in specific cases, if curing time considerations and requirements could not be satisfied, shrinkage-reducing admixtures (SRA) could effectively reduce drying and thermal shrinkages. Shrinkage-compensating cement or additive (SCC/SCA) was shown to provide excellent protection against shrinkage, and scaling of concrete, provided that water or moist curing was enforced. Proper selection of aggregate with adequate size, minimum siliceous mineralogy for ASR expansion considerations, excellent elastic properties for shrinkage considerations, reduced absorption capacity, and adequate shape may enhance the aggregate-paste interface that directly affect the cracking behavior of concrete at early-age.

CHAPTER 3
Cracking of Bridge Decks in Nevada

3.1 General

The transportation infrastructure of the Silver State has strived to provide its users with highly durable roads and bridges. The Nevada Department of Transportation (NDOT) has encountered shrinkage and cracking problems on some of the recently completed and ongoing bridge projects both in the Northern and Southern regions. This chapter provides examples of bridges both in Northern and Southern Nevada, which have showed cracking. The most striking examples are those that have exhibited substantial cracking at early-age.

3.2 Review of Bridge Decks in Northern Nevada

3.2.1 Construction of the Steamboat Creek Crossing Bridge Deck

The Steamboat Creek Bridge is located south of Reno, NV, on the east side of the Mount Rose Highway, and was built as an extension of the Geiger Grade to facilitate access to U.S. 395 across Steamboat Creek [Figures 3-1 thru 3-3]. It is a simple span post-tensioned box girder concrete superstructure. The second half of the concrete deck was cast June 9, 2000. The cast started around 5:15 am and completed at approximately 11:00 am.

A local ready mix plant provided the fresh concrete, specified to a NDOT[110] EA modified class concrete, which contained fly ash added at a rate of 20% for a water-cementitious materials ratio of 0.40. A more detailed summary of the mix design used for this project is presented in Table 3-1. Curing was provided through the application of a white curing compound applied as specified by NDOT [Figure 3-1(a)]. Field Engineers present during the construction deemed that the ambient conditions were acceptable. Wind speed on site averaged 1.3 m/s (3 mph), and a peak gust of 2.4 m/s (5.5 mph) was recorded.

Concrete sample taken from a truck picked at random was used to cast fifteen 100 x 200 mm (4 x 8 in.) cylinders for compressive strength and two drying shrinkage beams [Figure 3-1(b)]. A typical sight during construction is shown in Figure 3-2(a). The University of Nevada at Reno then tested all the specimens under ASTM specifications. Air content was estimated to be 4%. A slump of 100 mm (4 in.) and a fresh concrete temperature of 21► C (70► F) were measured. All measurements were taken from the same sample used to cast the specimens. A high range water reducer (HRWR or superplasticizer) was used for this project. Results from the tests will be presented in Chapter 5.

On August 15 2000, photographs of cracking at various locations on the bridge deck were taken to document the condition of the deck. A view of the entire bridge is shown in Figure 3-2(b). Figure 3-3 shows two snapshots of both longitudinal and diagonal cracks. It was observed that the cracks shown in Figure 3-3 were located on 80% of the deck.

3.2.2 Field Inspections of Other Bridges

The following information is based on the information provided by NDOT. The Bridge and Materials Divisions participated in a bridge deck inspection tour of the Reno area on July 19, 2000. The purpose of the trip was to identify construction parameters, which produced better bridge decks. Once identified, these construction parameters were to be used for the Carson City bypass project. A total of eleven bridges were visited on that day. Their names, locations, date of cast, and observations are presented in Table 3-2. Table 3-2 also shows the curing method applied to each of the bridges. It was deduced that the typical curing compound, as specified in NDOT 702.03.011[110], was used on all the bridge decks. Photographs of cracks from some of these bridges are provided in Figures 3-4 thru 3-6. Observations indicated that some of these cracks were due to concrete shrinkage. It is not possible to ascertain if the visible thin cracks that were observed during the inspection were attributed to drying, plastic, or thermal shrinkage. The inspection date and the information presented in Table 3-2 were insufficient to confirm that drying shrinkage caused the visible cracks on some of the bridge decks. The main reason for this is that additional weather data and construction records at the time of casting of these decks were not available.

3.3 Review of Bridge Decks in Southern Nevada

3.3.1 The Sahara Avenue Bridge

The Sahara bridge is a component of the I-215 Belt Project in Southern Nevada. The superstructure is a two-span post-tensioned concrete box girder bridge. The deck cast started September 7, 2000 around 8:30 pm and ended after twelve hours. During the first 4 hours of the cast, it was windy with an average wind speed of 9 m/s (20 mph).

The concrete selected for the project was a NDOT Class D modified concrete. Concrete producers used a type V cement only at 390 kg/m^3 (7.0 $sacks/yd^3$) of concrete with a water-cementitious ratio of 0.45. The mix design is summarized in Table 3-3. Fresh concrete quality control obtained on the site on the evening of the pour indicated a slump of 100 mm (4 in.) and a fresh concrete temperature of 28▸ C (83▸ F). A white-pigmented curing compound was used for compliance with NDOT specifications for curing procedures.

During a visit to the bridge eight hours after the end of the deck pour, the curing compound was still being applied on the deck surface ensuring multiple layers to provide greater protection against evaporation [see Figure 3-7]. A close visual inspection of the deck revealed the premature occurrence of both longitudinal and transverse hairline surface cracks.

A concrete sample during the deck pour was randomly picked from a truck. The sample was used to cast beams and cylinders for shrinkage, compressive strength, and resistance to chloride ion penetration tests. The University of Nevada at Reno performed these tests. The test results will be presented in Chapter 5.

3.3.2 The U.S. 95 Highway Project

The bridge on U.S. 95 was a concrete superstructure with a three-span post-tensioned concrete box girder system. Information obtained from NDOT indicated the full deck cast started July 5, 2000 at 7:00 pm and finished at 5:00 pm the next day. Starting and ending ambient temperature for the deck pour was 40► C (104► F) and 42► C (108► F), respectively. The lowest temperature during the course of the deck cast was 28► C (82► F), and contractors indicated windy conditions during the cast.

The concrete delivered for this deck project had a water-cementitious ratio of 0.49. A type V cement with a class F fly ash at 20% addition rate was used (see mix details in Table 3-4). A content of 368 kg/m^3 (6.6 sacks/yd^3) of cementitious materials for the concrete was specified for this project. A curing compound was used following NDOT specifications.

A site inspection on September 8, 2000 revealed extensive longitudinal (~70% of the surface area of the whole deck) and transverse (~30% of the surface area of the whole deck) cracking on the mid-span [Figures 3-8 and 3-9], though both types of cracking were apparent on the whole deck surface. Because the visit took place at a different time than the cast day, no sample was taken from the concrete to test for shrinkage and other relevant tests.

3.3.3 The Desert Inn Deck

The superstructure of Desert Inn consisted of a simple span post-tensioned concrete girder system. The Desert Inn deck cast started July 6, 2000 at 1:00 am and took thirteen hours to complete. It should be noted that the concrete supplier for this project was different from the one for the U.S. 95 Highway project. Ambient conditions during the pour were similar to that of the U.S. 95 bridge deck pour, except for lower wind conditions. The ambient air temperature measured at the beginning of the deck pour was 31► C (88► F).

A NDOT class D modified concrete was specified for the project. Fly ash was used at a rate of 15% for a 362 kg/m^3 (6.5 sacks/yd^3) of cementitious materials of concrete (see mix summary in Table 3-5). The water-cementitious ratio was specified at 0.45. Fresh concrete temperature was measured at 27► C (80► F). A white-pigmented curing compound was applied.

On September 8, 2000 a site inspection indicated an almost crack-free deck except for a few localized hairline cracks [Figure 3-10]. The bridge was already partially opened to traffic. No other significant deck deterioration was observed.

3.4 Survey of Selected U.S. States

In order to collect data of existing practices and measures taken to minimize early age shrinkage and cracking of concrete bridge decks throughout other U.S. states, a survey was prepared and sent to western states that would have climate similar to that of Nevada. The survey consisted of twenty-seven questions consolidated under seven categories. The categories are cracking and curing considerations, reinforcement, girder type, cementitious materials, aggregates, admixtures, and the NCHRP Report. The nine states that were selected for the survey were Utah, Texas, Washington, Wyoming, New Mexico, Montana, Idaho, Colorado, and California. The survey is provided in Table A3-1 of the appendix.

3.4.1 Cracking and Curing Considerations

The degree of cracking was ranked from mild, moderate, to severe. The states of Colorado and Texas indicated that cracking was not a major problem. Only New Mexico reported severe degree of cracking, and the remaining states were split between mild and moderate degrees of cracking.

All the states specified measures to prevent drying and plastic shrinkage cracking. California, New Mexico, and Utah indicated that no precautions such as windshields, sunshades, or fog mists were used during hot weather concreting. Of the states who pointed out that precautions were taken during hot weather concreting, measures such as wet curing for 7 days, fog mists when hot and wind exceed 9 m/s (20 mph), use of wet burlaps, limitations on evaporation rate, monitoring of environmental conditions prior to casting, and utilization of curing compound were specified. It may be noted that the state of Montana indicated that 14 days of wet curing.

3.4.2 Reinforcement

The states were asked to answer if they specified minimum clear cover or bar size for transverse reinforcement. All the states responded that minimums of 50 to 63.5 mm (2 to 2½ inches) of top cover were specified. Six states indicated that they did not limit their transverse reinforcement size to No. 15 metric bars (#5 English units). Table 3-6 identifies the states that did or did not limit transverse reinforcement size.

3.4.3 Girder Types

The states were asked to estimate the percentage of girder type used for support of their bridge decks. They were asked to make the distribution on the following girder types: steel, concrete box, pre- and post-tensioned boxes, and others. All the states indicated that both the steel and prestressed girder types were the most commonly used girder types. The combined distribution for these support conditions ranged from 50 to 100%. Percentage points for each state are summarized in Table 3-7.

3.4.4 Cement Type and Content

Texas was the only state that specified the usage of type III cement. All the states specified using Type II cement. No states indicated the usage of type K cement. Minimum content ranged from

332 to 375 kg/m^3 (560 to 632 lbs/yd^3) of concrete. The state of California was the only one to indicate a maximum cement content of 475 kg/m^3 (800 lbs/yd^3) of concrete. Specified strength criteria of concrete at 28 days ranged from 25 to 31 MPa (3650 to 4500 psi).

3.4.5 Aggregates

Texas and Wyoming indicated that they specified the utilization of limestone, river gravel, dolomite, and slate for coarse aggregates. The remaining states did not provide any indications of the type of coarse aggregates they were using. A detailed summary of the questions related to aggregates type and sizes is presented in Table 3-8. California, Idaho, Texas, and Wyoming specified using nominal maximum size of aggregates of 38 mm (1½ in.), while the remaining states specified 19 mm (¾ in.) aggregates. None of the states indicated the use of coarse aggregate's modulus of elasticity and thermal expansion coefficient as criteria in the selection process of their aggregate.

3.4.6 Mineral and Chemical Admixtures

All the states indicated using fly ash, and six states pointed out that they used silica fume, slag, and other types of mineral admixtures. The names of the six states are provided in Table 3-9. Fly ash replacement rates in percent by weight of cement ranged from 16 to 35%. The high replacement rate of 35% was specified by the state of Texas. Wyoming did not answer whether it was using pozzolans or not.

Chemical admixtures were specified by all the states. These chemical admixtures ranged from mid-range water reducers to superplasticizers or high range water reducers (HRWR), air entraining agents, accelerators, retarding agents, and others. As Table 3-10 shows, only four states specified using superplasticizers (HRWR). All the states indicated using air entraining agents.

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New Mexico, Utah, and Texas indicated that they were not familiar with the report. Utah indicated that a related research was conducted by trying to implement the use of type K cement, and did not provide details on results from their research. Colorado indicated that it conducted the Cracking Tendency Ring Test suggested by the NCHRP Report and did not find conclusive findings from the recommended test.

3.5 Current Nevada Standard Specifications

It is vital to identify the current standard specifications the Nevada Department of Transportation (NDOT) employs to address the issues of early-age shrinkage and cracking of concrete bridge decks. A review of NDOT Standard Specifications[1] indicated that there were a few available provisions, which directly or indirectly addressed these issues. In order to clearly establish the needs for additional provisions, and/or improve existing ones, it was necessary to identify the existing NDOT provisions.

Cementitious materials must comply with NDOT 701[110] and ASTM C 150[87], which cover cement types IP, II, III, and V. NDOT does not have provisions for SCC/SCA cement types.

NDOT 702 sets specifications for the usage of concrete curing materials, and both chemical and mineral admixtures. Curing details for methods and procedures are provided in NDOT 501.03.09. The section, for example, indicated curing using the water and curing compound methods. These two methods may be effective in providing adequate curing in any weather conditions if NDOT Sections 501.03.09 and 501.03.10 are strictly enforced. Concrete temperature at placement during cold and hot weather is limited to at least 10?C (50?F) and no more than 32?C (90?F). Furthermore, NDOT does not have specific curing requirements based on weather effects that account for the interaction between concrete and air temperature, wind speed, and ambient relative humidity.

Details regarding the usage of chemical and mineral admixtures are found in NDOT 501.02.03. However, there are no current provisions that cover the usage of shrinkage-reducing admixtures (SRA), silica fume, and shrinkage-compensating cement or additives (SCC/SCA).

NDOT 706 contains provisions for the usage of aggregates for concrete. Coarse and fine aggregates are specified according to NDOT 706.03.01, and NDOT 706.03.03 respectively. These specifications cover gradation and properties requirements as provided in NDOT 501.02.02. Table I of NDOT 501.03.04 provides the classification for concrete. Concrete bridge deck qualifies for a Modified EA class concrete. For this class of concrete, the specified maximum size aggregate is 19 mm (0.75 in. or size No. 67). However, flexibility is provided in that the maximum size of coarse aggregates may reach 25 mm (1 in. or size No. 57).

NDOT 501.02.04 requires that concrete for a bridge deck project be subjected to compressive strength, slump, air content, and unit weight tests. While all the specified tests provide important properties of both fresh and hardened concrete, the specifications did not include critical test methods that would reveal additional indications of mechanical and durability properties of concrete.

Will and Sanders[111] recommended NDOT to adopt additional test methods and proposed a performance grading system to evaluate mechanical and durability properties of concrete. These tests were shown to address both mechanical properties and durability issues and were based on the concept of high-performance concrete adopted by the Federal Highway Administration.[112-113] The recommended performance grading system is derived from FHWA with additional features, which addressed needs specific to Nevada. The philosophy of the performance grading system is to enable the engineer to specify a performance grade for a particular concrete bridge deck project. Depending on the situation, the grade would reflect the levels of performance of the concrete under specified performance parameters. Table 3-11 shows a summary of the recommended performance grading system suggested by Will and Sanders.

It is clear from the recommended performance grading system that it attempts to address critical durability issues, including shrinkage and cracking potential through the application of test methods such as ASTM C 666[114] for freeze-thaw durability considerations, and ASTM C 157[115] for drying shrinkage. The recommendations also included a footnote addressing the issue of ASR expansion. The footnote suggested to using ASTM C 1260[116] for testing aggregates potential for expansion due to Alkali-Silica reactivity. Will and Sanders added that aggregates with 0.10% expansion at 16 days may be acceptable, and that combinations of aggregates that yield an expansion of 0.05% at 3 months and 0.10% at 6 months may also be acceptable.

3.6 Summary

Both northern and southern bridges of Nevada suffer from various degrees of premature cracking. It is clear that there is a lack of important data taken at the time of cast for any specific deck project to produce meaningful and accurate analysis of the causes of severe cracking, which have occurred on some of the projects mentioned in this chapter. Selected western states were questioned about their current practices regarding the construction of concrete bridge decks. Last, a brief overview of existing NDOT specifications related to the construction of concrete bridge decks was presented.

CHAPTER 4

Materials, Mixture Proportions, and Experimental Procedures

4.1 General

Durability and high-performance are two desirable attributes of concrete. Chapter 2 showed that there are many parameters that affect early-age shrinkage and cracking of concrete. However, it is not possible to investigate all these parameters in this research. The laboratory tests in this research focused mainly on drying and thermal shrinkage of concrete. For the purpose of this research, the selection of the materials, including the shrinkage-reducing admixture (SRA) and shrinkage-compensating additive (SCC/SCA), was based on local availability and approval from NDOT.

One of the objectives of the study was to provide NDOT with mix designs alternatives using the SRA and/or the SCC/SCA. Mix designs were produced to investigate the effects of the SRA, SCC/SCA, fly ash, and SRA + SCC/SCA combination on early-age shrinkage and cracking of concrete for bridge deck. Appropriate tests for fresh and hardened properties of concrete were selected and applied on the trial batches. The silica fume was not included in this study because of additional variability it would have introduced, which would have led to additional testing.[117]

This chapter describes the materials, mix proportioning and design, mixing procedure, and laboratory testing performed by the University of Nevada, Reno. It also describes additional tests performed by W.R. Grace. Additionally, Chapter 4 describes the tests performed on concrete samples collected from the field observations both in Northern and Southern Nevada (see Chapter 3 Sections 3.2 and 3.3).

4.2 Cementitious Materials

4.2.1 Cement

The cement was selected in accordance to NDOT 701[110] and ASTM C 150 specifications. Nevada Cement supplied the type II cement. A type II cement was selected for its good properties such as low heat and rate of hydration, sulfate attack protection, and low alkali content. Table 4-1 shows the chemical and physical test reports provided by Nevada Cement. Generally cement occupies between10 to 14% of a unit volume of concrete. This relatively low volume of cement is sufficient to provide all the required hardened properties of concrete. Three cement contents were investigated. The selection of the cement contents was based on current concrete mixing plants dosage practices, i.e. dosage of cement by the sack per cubic yard of concrete, and economic considerations.

The selected contents for the trial batches were 339 kg/m^3 (6.0 sacks/yd^3), 390 kg/m^3 (7.0 sacks/yd^3), and 418 kg/m^3 (7.5 sacks/yd^3) of concrete. The effects of low and high cement contents on hardened properties, namely drying shrinkage, heat of hydration, and cracking tendency were investigated. The objective of choosing the low cement content of 339 kg/m^3 of concrete was to determine if desired properties of concrete could be maintained with low cement. If the objective was satisfied, then economic savings could occur.

4.2.2 Fly Ash

The effects of fly ash on early-age shrinkage and cracking of concrete were discussed in Chapter 2. It was made clear that fly ash reduced both the rate and evolution of the heat of hydration and setting time of concrete. It was also made clear that fly ash provided suitable protection against alkali-silica reaction (ASR).

Shehata and Thomas[76] indicated that effective protection against ASR expansion could be provided if low-alkali cement and low calcium fly ash are combined properly. They suggested that the level of alkali content remains below 0.66%. They also pointed out that for a fly ash with high content of calcium, the effectiveness of reducing ASR expansion is lowered and may be explained on the basis of a pore solution chemistry. In addition they showed that if fly ash could reduce the concentration of hydroxyl ions (OH^-) of the pore solution of cementitious pastes to less than 0.6 mol/l, ASR expansion could be reduced as well. ASR expansion was reduced as total silica content of the fly ash increased (increase in amorphous silica only). They recommended using type F fly ash and an addition rate of 25% by mass of cement.

Will and Sanders[111] further showed how much of a particular fly ash is required to limit the risk of ASR expansion to an acceptable level. They indicated, as did Shehata and Thomas[76], that an addition rate of 25% of fly ash is adequate to provide an acceptable level of performance against chloride ion penetration without negatively impacting hardened properties of concrete (i.e. compressive strength and modulus of rupture).

It was thus decided that the pozzolan to be used in the trial batches would be a type F fly ash that met ASTM C 618[78], ACI 211[72] and 232[73] requirements and guidelines. NDOT 501.02.03 specifies that pozzolans addition rate be not more than 17% by mass of cement. NDOT further requires that the replacement of cement with pozzolan be at a rate of 1.2 kg (1-lb/lb) of pozzolan per kg of Portland cement. Based on the discussion from Chapter 2 on the use of fly ash, and recommendations from Will and Sanders, a 25% addition rate at a 1.0:1 mass ratio was selected for use in all trial batches containing type F fly ash. The chemical composition of the selected fly ash is available from Table 4-2. Industrial Services Group (ISG) Resources provided the fly ash. The effects of fly ash on heat of hydration and drying shrinkage were investigated.

4.2.3 Shrinkage-Compensating Additive (SCC/SCA)

Shrinkage-compensating concrete, when properly proportioned and cured, reduces considerably drying shrinkage, has a higher abrasion resistance, and provides adequate resistance to freezing and thawing.[102] The selection of the SCC/SCA was based on guidelines provided by ACI 223,[102] ASTM C 806,[118] ASTM C 845,[119] ASTM C 878,[120] availability considerations, and approval from NDOT. Other necessary guidelines were provided by CTS Cement Manufacturing Company. To the author's request, CTS performed an X-ray fluorescence analysis on the SCC/SCA to reveal its chemical properties, which are provided in Table 4-3.

ASTM C 806 was used to determine the restrained expansion requirements between the SCC/SCA and the cement supplied by Nevada Cement. A sample of the selected cement for the research was sent to CTS Laboratory for testing. Results of the restrained expansion test are shown in Table 4-3. Table 4-3 indicates good compatibility of the SCC/SCA with Nevada Cement type II, as it meets the minimum 0.04% expansion required at 7 days. After reviewing the results of the analysis, CTS recommended that an addition rate ranging from 15 to 20% of SCC/SCA be used with the cement for this project. The study did not focus on the effects of variations of SCC/SCA content, rather on the SCC/SCA effectiveness to minimize early age shrinkage and cracking of concrete. For this reason, an addition rate of 15% was used for all trial batches containing SCC/SCA.

The SCC/SCA met all the requirements of ASTM C 845. The requirements are presented in Table 4-4. ASTM C 878 is the standard test method for restrained expansion of shrinkage-compensating concrete. ASTM C 878 is very similar to ASTM C 157, except that ASTM C 878 uses a restraining cage or steel threaded rod. Thus, ASTM C 878 measures restrained drying expansion or shrinkage, while ASTM C 157 measures unrestrained drying shrinkage. For the purpose of comparing SCC/SCA concrete mix designs with those not containing SCC/SCA, ASTM C 157 would be able to measure shrinkage on all trial batches and produce uniform and comparable laboratory results.

A bridge deck construction project in California made use of the SCC/SCA. A typical mixture design associated with a particular CALTRANS (California Department of Transportation) project is summarized in Table 4-5, and related test results are available in Table 4-6. This particular project completed in 1995 was unique in that two bridge decks were poured the same day. One was a normal concrete mix design; the other was the same mix design with the SCC/SCA. CTS reported that to this day, no cracks have been noted on the deck built with the SCC/SCA concrete. It was not clear whether the other deck that did not contain the SCC/SCA cracked or not.

4.3 Coarse and Fine Aggregates

The discussion of the effects of aggregate type and content for concrete bridge decks revealed that mineral and mechanical properties, shape, and texture have direct or indirect implication in the early age shrinkage behavior of concrete. Because of the many variables involved with aggregates, it was decided that this research would not focus on the effects of aggregates on shrinkage and cracking.
The aggregate supplier selected for the project was Rocky Ridge of Northern Nevada. The supplier was submitted to and approved by NDOT. NDOT tested the aggregates for specification requirements found in NDOT 706. Test results showed that the Rocky Ridge aggregates met NDOT specifications. Details of the test results are summarized in Table 4-7. The aggregate densities and absorption properties showed in Table 4-7 were used in the design of the trial batches throughout the research.

4.4 Chemical Admixtures

4.4.1 Shrinkage-Reducing Admixture (SRA)

Shrinkage-reducing admixtures (SRA) are commercially available in the U.S. W.R. Grace Products supplied the SRA for the project. The manufacturer rated the solubility for the SRA as moderate. The SRA is a liquid admixture, which contains no expansive materials. The SRA is composed of 100% active liquid with a specific gravity of 0.95. W.R Grace recommended that the SRA be used at an addition rate range of 1.0 to 2.5%. In this study, an addition rate of 1% was used. The focus was not to observe the effects of variation in SRA dosage, rather to observe if the SRA was efficient in reducing drying and thermal shrinkage. The addition rate was approved by NDOT.

The SRA contains no water, but the usual high rate of addition leads to an adjustment in water content. For this reason, the manufacturer recommends that when using an SRA, the SRA volume should replace the same amount of water for any concrete mixture design. This recommendation was also taken into account in the design of trial batches throughout the project. The SRA has minimal effects on slump, but has retarding effect in set time, thus improving workability (i.e. associated lower rate of slump loss).

The SRA does require adjustments in dosage of air entraining agents to achieve specified level of air content. These adjustments translate to higher air entraining agents' dosage. For this particular reason, careful selection of air entraining agents was coordinated through research and assistance from the SRA manufacturer.

4.4.2 Air Entraining Agent (AEA)

Daravair 1000 and Darex II were the two air-entraining agents (AEA) selected for the research, and W.R. Grace Products supplied them. Correspondence and recommendation from the manufacturer also affected the decision to select Darex II for use with the SRA. The requirements for adjustments in dosage for achieving specified air content with the usage of SRA led to the choice of Darex II as a second AEA to be used specifically with the SRA, while Daravair 1000 was used for all other mix designs not containing the SRA. The use of the SRA in combination with Darex II was also recommended elsewhere as it provided a more stable air-void system, thus enhancing the performance of the concrete under freeze-thaw cycles.[70] Properties of both AEA are summarized in Table 4-8. Both AEA comply with specification requirements from ASTM C 260.[121]

The amount of air content used for all trial batches during this project is based on NDOT 501 specifications[110] for a Modified EA class concrete. NDOT 501 specifications agree with ACI 211 recommendations concerning air-entrained concrete subjected to severe freeze-thaw cycles. Based on these requirements, a target air content of 5.5% was used, with tolerance of ? 1.5%. The addition rates for air entrainment used in each trial mix design were based on manufacturer's recommendations.

4.4.3 Superplasticizer (HRWR)

The selection of a good and efficient HRWR is crucial when making concrete based on a performance grading system. The usage of superplasticizer (HRWR) over plasticizers (WR) was decided because of many advantages HRWR have relative to WR. These advantages include high slump concrete with no reduction in strength, lower W/C ratios, no segregation in comparison to concrete without HRWR, and faster discharge of concrete from truck mixers. It was necessary to carefully select the HRWR that were selected for this project because of the use of SRA in some of the trial batches mixed during the project.

Some HRWR contain calcium salts that may provide additional protection when potentially alkali-reactive aggregates are used to make the concrete. The addition of a HRWR containing calcium salts rather than sodium salts helps lower the alkali content of the mix. The specific HRWRs, which satisfy the requirements of calcium salts content, are generally known as naphthalene HRWRs.[29] Naphthalene HRWR provide the following additional benefits:

?? Higher solids content that enhance workability and become more cost-effective,
?? Slight set delay making it easier to control the rheology of the concrete, and
?? Less expensive because of the availability from different manufacturers.
??

When the larger part of the HRWR is added at the end of the mixing sequence, the slump loss can be significantly reduced, which mean that almost all HRWR molecules are being used to fluidify the cement paste, rather than attempting to combine with the Tricalcium Aluminates (C_3A) and interfere with the hydration and hardening process.[29, 122-123]

For the reasons mentioned above, it was decided to use Daracem 19 as a HRWR for all mix designs that did not contain the SRA. W. R. Grace Products supplied Daracem 19. The second HRWR selected for the project was the Adva 100, also provided by W. R. Grace Products. W.R. Grace recommended that Adva 100 be used with mix designs containing the SRA for better results. Daracem 19 satisfied specifications from ASTM C 494[79] as a HRWR type A and F, while Adva 100 satisfied ASTM C 494 as a type F HRWR. Both HRWR complied with NDOT 702.03.03

specification requirements for admixtures other than air-entraining.[121] The selected HRWRs and AEAs also complied with guidelines from ACI 212.[124, 88]

An investigation was conducted into the effects of HRWR in combination with trial batches containing SRA. Table 4-9 summarizes the properties of the Daracem 19 and Adva 100. Furthermore, another objective of the research was to determine if the Adva 100 could be used in combination with the SRA and Darex II without observing secondary effects, especially the loss of air content as the SRA concrete hardened.

Because of inherent variability during trial batching in a laboratory setting, the rate of addition of HRWR varied for each mix design, but was set based on the final targeted slump (i.e. slump after air content was satisfied, at least 100 mm (4 in.) but not to exceed 200 mm (8 in.)), and upon recommendations from the manufacturer. Thus the final addition rates of HRWR used in the trial batches during this project reflected both the unique properties of each trial mix design as well as an attempt to remain within the range of addition rates provided by the manufacturer.

4.5 Curing Procedures

4.5.1 Curing Compound

A curing compound was used in this project. It was desired to visually observe its effectiveness in reducing the evaporation loss during hydration and curing of the concrete. A white-pigmented curing compound conforming to NDOT 702.03.04 specifications was used. Titan Construction Supply from Reno, NV, supplied the curing compound. The compound is described as a poly-alpha-methyl styrene concrete curing compound. Information pertaining to the physical properties of the curing compound is summarized in Table 4-10. In addition to the curing compound, three curing methods were defined and applied during the research. The first two curing methods were applied during part one (described later), while a third curing method was applied during the second part (described later) of the research. The three curing methods are described below. The first two curing methods were defined in an attempt to replicate actual field conditions. They were submitted to and approved by NDOT.

4.5.2 Method 1

Specimens were stripped from the molds and a white-pigmented curing compound was immediately applied on all specimens. Then they were placed in the basement [see Figure 4-1] of the structural lab under ambient temperature and humidity conditions throughout the rest of phase one of the research. This method will be referred to as “dry cure”.

4.5.3 Method 2

Specimens were stripped from the molds and a white-pigmented curing compound was immediately applied. Then the specimens were placed in a “moist” box [see Figure 4-2] on top of a wet burlap and were cured under another moist burlap for 72 hours. The specimens were then retrieved from the moist box and placed in the basement of the structural lab under ambient temperature and humidity conditions. The burlap met NDOT 702.03.01 curing materials compliance specifications.

The moist box was built to accommodate the amount and size of the specimens placed within. The walls of the moist box consisted of thick plastic sheeting that kept the moisture from evaporating. The box was placed on the floor of the basement of the structural lab and left under ambient temperature and humidity conditions throughout the rest of part one of the research. This method will be referred to as “moist cure”.

4.5.4 Method 3

All specimens were stripped from the molds. Then the specimens were placed in a controlled chamber where temperature and relative humidity were set to ASTM C 157[115] Section 10.1.2 recommendations for air storage. The chamber temperature was set to 23► C ? 1.7► C (73.4► F ? 3► F), and the relative humidity was set to 50 ? 4%. This curing condition was maintained throughout the testing of the specimens. Please note that this method was applied to mix $C2\text{-}SC_B$ immediately after the specimens spent 7 days in the moist box. This method will be referred to as “lab cure”.

4.6 Materials Storage

The cementitious materials and all the admixtures were stored in the basement of the structural laboratory to be protected from weather conditions. This condition also provided a semi-controlled basement climate environment. The cement and SCC/SCA were kept in their original sacks until use; the fly ash was in its sealed bucket until use. The admixtures containers were opened only before each trial mix batching.

The coarse and fine aggregates were each stored in their own covered wooden box and kept outdoors. The aggregates moisture content was determined using ASTM C 566[125], and was taken into account in the proportioning of the concrete ingredients. The moisture content of the aggregates was determined the afternoon before the morning of each trial mix batching.

4.7 Mix Preparation, Procedure, and Mixing Equipment

The materials were weighed in 20-liter (5-gallon) buckets 15 hours before each trial mix batching. The buckets were stored in a shaded area in the structural lab to allow the materials to cool down during the night. All the trial mix designs were batched early in the morning following the cooling period of the materials.

In preparation for mixing, the drum was “buttered”. The buttering process consisted of placing into the drum a 20-liter bucket containing a combination of coarse aggregates, fine aggregates, and cement. The drum was started and water was added at the beginning of the rotations until an acceptable consistency was reached. The drum was stopped after 2 minutes and the content was discharged. The buttering ensured the drum wall was at a saturated surface condition.

For each trial mix design, two-third of the coarse and two-third of the fine aggregates were placed into the drum, followed by a third of the water that contained the air-entraining admixture (AEA). The drum was started and spun for 1½ minute. At the end of the 1½-minute, the drum was stopped and materials that stuck to the drum wall were scraped off. The remaining coarse and fine aggregates were added, followed by the cementitious materials. The drum was started and the remaining water was added immediately as the drum spun. For all trial mix designs containing the shrinkage-reducing admixture (SRA), the remaining water contained the SRA. The materials were mixed for a few spins and the drum was stopped, and materials sticking to the drum wall were removed. Then it was mixed for an additional 2½ minutes, at the start of which the HRWR was added incrementally. At the end of the 2½ -minute mixing, the trial batch was checked for slump while it rested for 1 minute. The checking was set to take no more than one minute. When the slump reached an acceptable value, the drum was stopped and air content was measured. If the targeted range of air content was not achieved, adding AEA to the trial batch made the adjustment. In any case, the mixing time after each AEA adjustment was never to exceed one minute thereafter. At that point, the trial batch was discharged into wheelbarrows and all of the test specimens were fabricated. The digital thermometer was placed into the concrete in one of the wheelbarrows and fresh concrete temperature was read at that point.

The mixing equipment was an electric Whitman model WC-92S. The actual capacity of the mixer was 0.25 m^3 (9 ft^3). However, it was decided not to exceed a batch size of 0.13 m^3 (4.5 ft^3) so the trial batches would provide excellent consistency results.

4.8 Mix Designs and Testing Program

This research investigated a total of 27 different trial concrete mix designs. The study was broken in three Phases. As Table 4-11 shows, 14 mixes were done in Phase 1, 8 mixes in Phase 2, and 5 mixes in Phase 3. The tests performed by the University of Nevada Reno and those performed by W.R. Grace, for each Phase, are shown in Table 4-12. The range of trial mix designs included a variety of combinations of cementitious materials with or without the SRA, and with or without the SCC/SCA. The various combinations reflected the desire of the author to investigate the effects of the fly ash, SRA, and SCC/SCA on early age drying shrinkage and cracking of concrete.

4.8.1 Control Mix Designs and Mix Design Procedure

Three control mix trial batch mix designs were defined for the research. All control mix designs had identical properties, except for cementitious materials content. Cementitious contents ranged from 6.0, 7.0 to 7.5 sacks/yd^3 of concrete. Table 4-13 lists the control mix designs for the project. Table I of NDOT 501 specifies the use of a Modified EA class concrete for roadway deck slabs and approach slabs. Using the properties of the selected materials, defined cement contents, and requirements for Modified EA class concrete, the mix proportions were accomplished using guidelines set forth in ACI 211.1[126], ACI 211.4R[72], and De Larrard.[30]

4.8.2 Mix Designs containing the Fly Ash, SCC/SCA, and SRA

Trial batches containing the SCC/SCA were tested at all levels of cement content. However, the trial batches that contained the SRA were tested only at two levels of cement content. The fly ash was added at rate of 25% of cement content at a 1:1 mass ratio. The SCC/SCA was added at 15% of cement content at a 1:1 mass ratio in all trial batches. The SRA was added at a rate of 1% by mass of cement, and replaced water at 1:1 volume ratio. Table 4-14 lists all the trial mix designs, which made use of the fly ash, the SCC/SCA and the SRA.

The mix designs are identified first by cement level (i.e. control mix reference), following with the “SC” symbol to denote mix containing the SCC/SCA, then with “FA” when fly ash was included. If both the SCC/SCA and the fly ash were added to a mix, “FA” was written before “SC”. Finally “SRA” if the mix design contained the SRA. As an example, if the mix design contained 418 kg/m^3 of cement, without SCC/SCA and no fly ash, but with SRA, the mix would identify as C3-SRA. If for the same cement content, the SCC/SCA, fly ash, and SRA were included, then the mix design would identify as C3-FA-SC-SRA. The remaining ingredients for each trial batch were adjusted accordingly. It must also be noted that the adjustments made for water content were performed to remain consistent in the final adjusted 0.42 W/C ratio targeted for all trial batches of this research project.

4.9 Fresh Concrete Properties

All the trial batches in this research were submitted to slump, air content, and temperature tests. The slump test was performed according to ASTM C 143.[127] The air content was measured by following procedures outlined in ASTM C 173,[128] while the temperature was read following recommendations from ASTM C 1064.[129]

The temperature reading was performed immediately after the end of mixing at the time when both slump and air content targets were satisfied at the mixing stage. The thermometer was left in the concrete for at least three minutes to ensure the accuracy of reading. Fresh concrete properties were assessed for the first two Phases of the study. In the third Phase of the study, air content and slump were obtained from the quality control checks performed by the engineers on site. The results from the fresh concrete tests of the trial batches designed by the University of Nevada-Reno are summarized in Table 4-15. Table 4-15 shows only fresh concrete properties of the first Phase of the research. Results for the second and third Phases of the research are presented in Chapter 5. Table 4-16 identifies the HRWR and AEA used for each trial batch in Phase 1 of the study, and the corresponding final dosage. Results for Phases 2 and 3 are presented in Chapter 5.

4.10 Hardened Concrete Properties

The tests performed consisted of the drying shrinkage using ASTM C 157,[115] compressive strength with ASTM C 39,[130] flexural strength by the modulus of rupture following ASTM C 78,[131] the cracking tendency and the heat of hydration as described in the NCHRP Report 380,[9] the estimation of the resistance of concrete to chloride ion penetration with ASTM C 1202,[132] and the determination of the hardened air-void system with ASTM C 457.[133]

4.10.1 Drying Shrinkage (ASTM C 157)

The drying shrinkage test was performed on all specimens of each trial mix designs prepared throughout the research. For each mix design in Phases 1 and 2, two 75 ? 75 ? 285 mm (3 ? 3 ? 11¼ in.) shrinkage specimens for dry cure and two 75 ? 75 ? 285 mm (3 ? 3 ? 11¼ in.) shrinkage specimens for moist cure were cast. Molds used for the specimens complied with ASTM C 157 [see Figure 4-3(a)]. In contrast, in Phase 3, 3 drying shrinkage specimens were prepared.

Immediately after the cast, the specimens were placed on a table [see Figure 4-1(a)] under a plastic sheet and left to cure for 23-½ ? ½ hrs. Then they were stripped and the reference reading performed within the first half hour as specified by ASTM C 157. Following the initial reading, all four specimens (for each trial mix design) were sprayed with the curing compound, and left to dry for 5 minutes. For the moist cure, two specimens were placed in the moist box for 72 hours, while two other specimens were placed on a custom-built storage shelf as illustrated in Figure 4-3(b) for dry curing. After the 72 hours moist curing, the drying shrinkage specimens were retrieved from the moist box and placed on the storage shelf with the other two specimens to be cured under dry curing condition. After the initial reading for reference, length comparator [see Figure 4-4(a)] readings were done at 3 days, 7 days, 14 days, 28 days, and 56 days on all specimens for all trial mix designs. This test was performed throughout all Phases of the study and tests results are presented in Chapter 5.

4.10.2 Compressive Strength (ASTM C 39)

For each trial mix design, a total of 24 100 ? 200 mm (4 ? 8 in.) cylinders were cast [Figure 4-4(b)]. A lid was placed on each mold about 3 minutes after the cast and all 24 cylinders were left to cure for 23-½ ? ½ hrs. After that time, they were stripped and curing compound was sprayed on all the cylinders, and left to dry for 5 minutes. Twelve cylinders were left to cure in dry condition, while the twelve others were placed in the moist box to be under moist cure for 72 hours. After the 72 hours, the twelve cylinders in the moist box were taken out and joined the other twelve cylinders to cure under dry (or ambient) condition.

The specimens were tested for compressive strength in the concrete laboratory [Figure 4-5(a)] when they reached the age of 3, 7, 28, and 56 days. Figure 4-5(b) shows the box in which the cylinders were placed to be sprayed with the curing compound. The specimens were left in the box for 5 minutes before they were placed either in the moist box for good curing, or on the shelf and under the table for dry cure. Please note that these same boxes were used to spray all other test specimens as well. Compressive strength tests for the UNR mix designs were performed for all three Phases of the study, and results are presented in Chapter 5.

4.10.3 Flexural Strength using the Modulus of Rupture (ASTM C 78)

Four 75 ? 75 ? 285 mm (3 ? 3 ? 11¼ in.) beam specimens were cast for each trial mix design using standard procedures provided by ASTM C 78. After 23-½ ? ½ hrs of initial cure, the beam specimens were stripped form the molds, placed in the spray box [figure 4-5(b)] and curing compound was applied to all beam specimens. A 5 minutes period elapsed before two beam specimens were placed in the moist box to cure under the moist curing condition for 72 hours, while the other two beam specimens were placed on the storage shelf to cure under dry curing condition. After the 72 hours of good cure in the moist box, the specimens were retrieved and placed with the other two beams already under dry (or ambient) cure.

The beam specimens were tested in the concrete laboratory. A suitable apparatus meeting ASTM C 78 requirements was fabricated to perform the flexure test by the third point loading method. The tests were performed when the specimens were at 3, 7, 28, and 56 days of age during Phase 1 of the research, while measurements were taken at 7 and 28 days for beam specimens cast during Phase 2 of the research. Results from the modulus of rupture tests as a measure of the tensile strength of concrete under flexure are presented in Chapter 5.

4.10.4 Cracking Tendency and Heat of Hydration (NCHRP 380)

The cracking tendency and heat of hydration were evaluated by a test that is based on a procedure suggested by the NCHRP Report 380. A 70 mm (2 ¾ in.) thick concrete ring was cast around an inner circular steel ring. The inner circular steel ring had an outside diameter of 319 mm (12 ¾ in.) and a wall thickness of 12.7 mm (½ in.). A non-reusable sonotube with inside diameter of 457 mm (18 in.) served as the outer form for the concrete ring. The height of a test specimen was 152 mm (6 in.). The specimen was cast on a flat wooden base that was covered by a thick polyethylene sheet, as shown on Figure 4-6. A releasing agent was applied on the base and walls of the concrete ring form (i.e. outer face of the steel ring, inner face of the sonotube, and base of the concrete ring form).

Three general foil type strain gages were attached to the inner face of the steel ring, and indirectly monitored restrained drying shrinkage of the concrete ring. As the concrete ring shrunk, it compressed the steel ring and generated strain in the steel ring. A digital automated data acquisition system was used to record the strain, which recorded the steel strain in each gage at intervals of 15 minutes for 21 days. An independent steel ring with strain gages, without a concrete ring, was monitored for temperature compensation.

A thermocouple was placed in each concrete ring and measured the temperature evolution of the concrete, while another thermocouple was placed in the structural lab basement close to the steel ring specimens to monitor the ambient temperature. The thermocouples were connected to the same data acquisition system as the strain gages and the measurements were taken every 15 minutes for 21 days.

All the concrete rings were cast and left to cure for 23-½ ? ½ hrs on the floor of the structural laboratory. During the initial 24-hr cure, one ring was submitted to moist cure, which consisted of spraying the top surface of the concrete ring with water and cover it with a wet burlap and a thick polyethylene sheet over the burlap. The second concrete ring was covered with a polyethylene sheet. After the initial cure, the sonotubes were stripped and the concrete rings were lifted from the base to minimize restraint due to friction between the concrete and the base, and both rings upper and outer wall face were sprayed with curing compound. After 5 minutes, the first concrete ring was covered with a wet burlap and a thick polyethylene sheet over it and moist cured under ambient conditions in the structural lab basement for 72 hours. After the 72 hours, the burlap and polyethylene sheet were removed, and the first ring simply cured under dry (ambient) structural laboratory basement conditions. The second ring was left to dry cure under ambient conditions in the structural lab basement. This procedure was applied to all trial mix designs during Phase 1 of the study.

The concrete rings were visually monitored for apparent cracks, and the monitoring took place daily. Results from the cracking tendency and heat of hydration tests are available in Chapter 5.

4.10.5 Chloride Ion Penetration (ASTM C 1202), and Hardened Air-Void System (ASTM C 457)

A selection process determined the mix designs that were promising in application of bridge deck. This selection process is described in more details in Chapter 5. These selected mix designs were reproduced and submitted to tests described in Phase 2. The University of Nevada-Reno tested all the mix designs reproduced in Phase 2 for resistance to chloride ion penetration; except for the control mix designs. Only selected mix designs from Phase 2 were subjected to the hardened air-void system. Because the laboratory at UNR was not equipped to perform the hardened air-void system test, W. R. Grace Products conducted both the chloride ion penetration and hardened air-void system tests on the selected mix designs.

The hardened air-void system standard test method is carried out to determine the air content of hardened concrete. In addition, the test provides information regarding specific surface, void frequency, spacing factor, and paste-air ratio of the air-void system in hardened concrete. These parameters are related to the susceptibility of the cement paste to damage by freezing and thawing. The test produces data to estimate the likelihood of frost damage. Results of the chloride ion penetration and hardened air-void system tests are presented in Chapter 5.

CHAPTER 5

Experimental Results

5.1 Introduction

As indicated in Chapter 4, laboratory testing was conducted on 27 trial mix designs. The investigation focused on fresh and hardened properties of concrete. The analyses of these properties were consolidated to enable the determination of effective, robust, and construction-friendly concrete mixture design that would appropriately apply to Nevada concrete bridge decks.

Results for Phase 1 of the study (as described in Table 4-11 and Table 4-12) are presented and discussed first, followed by a discussion of the selection process of the most promising trial mix designs, which were thereafter reproduced and tested according to the procedure described in Phase 2 of the study (Table 4-12). The second section of this chapter contains results from the second Phase and a discussion of the results. The last section of this chapter presents results from Phase 3 of the study.

5.2 Test Results: Phase 1

This phase of the study included tests for compressive strength, drying shrinkage, flexural strength as a function of modulus of rupture, cracking tendency, and heat of hydration. Effects of cementitious materials contents, the SCC/SCA, the SRA, the various combinations of SCC/SCA with SRA, and the curing conditions in each of the tests are discussed. A discussion on the effect of the HRWR used in this research is also provided.

5.2.1 Compressive Strength (ASTM C 39)

5.2.1.1 Impact of Cement Content

The impact of the three cement contents used in this research clearly showed that increasing cement content consistently increased the compressive strength of concrete for the control trial mix designs presented in Table 4-13. The compressive strength of each trial mix design was measured at 3, 7, 28, and 56 days. The data points are averages of three specimens. Figure 5-1 shows the effect of cement content on the compressive strength of the control trial mix designs for specimens subjected to dry cure, while Figure 5-2 shows the same effect for the control mix designs subjected to moist cure. Both Figures 5-1 and 5-2 clearly illustrate the increasing gain in compressive strength of the control mix designs as the cementitious content was increased, whether in dry or moist curing conditions. In addition, the implementation of moist cure also increased the compressive strength of concrete as compared to the compressive strengths recorded for the specimens that were dry cured. For this study, typical increases in strength based on curing regime were of the order of 1.5 to 15%.
One other significant result obtained from the effect of cement content is the reliable and consistent level of strength reached by the mix designs with the lowest cement content. All the trial batches containing this cement content tested very well in compressive strength and even exceeded the targeted strength level of 27 MPa (4000 psi). This observation showed that appropriate mixture proportioning for concrete with low cement content could achieve desirable strength level.

5.2.1.2 Impact of the SCC/SCA

To better assess the effect of SCC/SCA on hardened properties of concrete, it was decided to isolate it from other parameters, and test it at the cement content identified as C2 in Table 4-13. Isolating the SCC/SCA from other parameters provided results that illustrated best the effect of the SCC/SCA on concrete compressive strength. The impact of SCC/SCA on the compressive strength of concrete is shown in Figure 5-3. The hydration of the SCC/SCA tends to slightly decrease the compressive strength of concrete during the initial curing when compared to the control mix designs, regardless of the applied curing method. Trial mix design C2-SC_A (see description in Table 4-14) was subjected to both dry and moist cure, while C2-SC_B (see Table 4-14) was subjected to lab cure. It is evident from Figure 5-3 that all three mix designs projected close strengths, except during the first 7 days during the curing period. Thus while the curing regimes slowed the compressive strength gain rate during the imposed curing time, the strength gain rate after the curing period did not differ significantly from the control mix. It must be noted however that as the 56 days were being reached, trial mix C2-SC_B was slightly developing higher strength gain, showing the effectiveness of the lab curing in enhancing the long term compressive strength of concrete made with the SCC/SCA.

Compressive strength test results of the trial mix designs for the two other cement contents C1 and C3 containing both the SCC/SCA with the fly ash are shown in Figure 5-4. The results show that the combination of SCC/SCA and fly ash produce the lowest compressive strengths in both cement levels. This observation correlates with the relatively high air content level recorded for mixes C1-FA-SC and C3-FA-SC as summarized in Table 4-15. Though the compressive strengths were the lowest of all trial mix designs, the strengths exceeded the targeted strength level at 28 days. Figure 5-4 further shows that a 3-day moist cure could provide significant improvement in the rate of compressive strength gain. This important observation confirms the vital need of moist cure to enhance the compressive strength of mix designs prepared with the SCC/SCA alone, and that of mix designs incorporating a combination of SCC/SCA with fly ash.

5.2.1.3 Impact of the SRA

The impact of the SRA on compressive strength is shown in Figures 5-5(a) and 5-5(b). Figure 5-5(a) shows the impact of the SRA and SRA with Fly Ash on compressive strength for the low cement content at both the dry and moist cures. Figure 5-5(b) shows the same effect for the high cement content. It was found that the SRA reduced the compressive strength of concrete at all levels of cement content investigated. This reduction was expected, and confirmed both the test results and the literature study presented in Chapter 2. An overview of the calculated reduction indices is presented in Table 5-1. The indices indicated that the variation in the strength reduction at any age of the concrete is not strictly limited to a narrow range, rather the variation is wide and typical of strength losses observed in mix designs using the SRA.

The data showed that the SRA reduced the compressive strength of concrete subjected to dry curing at 28 days by 7%, and 20% for the SRA and fly ash, regardless of the cement content. In contrast, the moist curing procedure clearly improved strength with the SRA concrete strength at 28 days reduced only by 6%, and 13% for the SRA and fly ash concrete. It is to be noted that the fly ash contribution to strength is not fully developed at 28 days and strength monitoring of any concrete containing fly ash should continue up to 90 days. Though strength losses were noted, all the mix designs showed in Table 5-1 met and exceeded strength requirement at 28 days, regardless of the presence of the SRA and the fly ash. The same effects were also observed on specimens tested at 56 days.

Figure 5-5(a) and Figure 5-5(b) also show that the applied curing method and curing time do not significantly affect the compressive strength of mix designs containing SRA alone. Yet, the same figures indicate that a mere 3-day moist cure enhances the compressive strength. Another observation was that for these mix designs, the one containing fly ash with the SRA and subjected to moist cure showed greater strength gain rate than that subjected to dry cure. The moist cure was sufficient enough to provide the necessary hydration in the presence of fly ash in the mix design.

5.2.1.4 Effect of Various Combinations of SCC/SCA, SRA, and Fly Ash

The effect of the combinations of SCC/SCA, SRA, and fly ash on compressive strength is illustrated in Figures 5-6(a) and 5-6(b). Figure 5-6(a) shows the effect of the combinations on compressive strength of the low cement content subjected to both dry and moist cures, while Figure 5-6(b) shows the same effect for the high cement content. These figures show that the combination of SCC/SCA, SRA, and fly ash yield compressive strength that exceeded the targeted strength. They also show that in the presence of fly ash and moist cure, the compressive strength of mix designs at both levels of cement contents have the lowest strengths but still exceed the targeted strength at 28 days. This reduction of strength demonstrate the need for carefully implementing a combination of the SCC/SCA, the SRA, and the fly ash altogether. The SRA and fly ash both reduce the compressive strength of concrete. The SCC/SCA also has lower compressive strength compared to its control mix design. The sum of the individual effects of each of these admixtures form a compound that is very likely to exhibit a much lower compressive strength, regardless of the curing method applied.

In addition, the data suggests that, for mix designs combining all three components and subjected to moist cure, the strength loss induced by the SRA is compensated by the presence of the SCC/SCA. The initial 3-day moist cure caused this compensation, indicating here again the effectiveness of the moist cure in hydrating the cementitious materials, more specifically the SCC/SCA and fly ash. A complete summary of the compressive strength test results of all trial mix designs batched in this first part of the research is found in the appendix in Table A5-1.

5.2.2 Drying Shrinkage (ASTM C 157)

5.2.2.1 Impact of Cement Content

As anticipated, the drying shrinkage behavior of specimens at the three levels of cement content showed that increasing the cement content also increases the drying shrinkage of concrete, regardless of the applied curing method. As Figure 5-7 shows, the lowest cement content produces the lowest drying shrinkage.

Table A5-2 in the appendix summarizes the results from the drying shrinkage tests of part one of the research. Table 5-2 provides a summary of the drying shrinkage improvements, in percentage terms relative to the control trial batches, of all the mix designs of Phase 1. While a low cement concrete reduces the drying shrinkage, it does not mean that it achieves a desirable shrinkage performance. Using Table 3-11, it can be seen that the control mix designs would achieve very low performance grades. In fact the three mix designs would receive shrinkage performance grades of 2, 2, and 1 respectively. This means that if cement content alone is a parameter for shrinkage performance, low cement content does not necessarily mean good shrinkage performance. Thus very poor shrinkage performance is achieved regardless of the cement content, for the control mixes.

5.2.2.2 Impact of the SCC/SCA

It can be seen from Figures 5-8(a) and 5-8(b) that the drying shrinkage of trial batches containing the SCC/SCA improved only in the presence of the 3-day moist cure and the addition of fly ash. Figure 5-8(a) shows the effect of the SCC/SCA with fly ash on drying shrinkage at the lowest cement content, while Figure 5-8(b) shows the same effect for the highest cement content. It should be noted that at both the low and high cement contents, the drying shrinkage behavior was essentially the same. It was also evident that the fly ash enhanced the long-term drying shrinkage in the presence of moist cure. Thus the SCC/SCA alone did not improve the drying shrinkage behavior of the trial batches at the lowest and highest cement contents even when subjected to a 3-day moist cure.

In contrast, the trial batches that contained the intermediate cement content (C2) showed very good drying shrinkage behavior (this behavior would also be expected at any other level of cement content). It must be noted that for mix designs containing expansive cementitious materials the curing procedure has a major effect on the results. The mix design C2-SC_A was both dry and moist cured, while the C2-SC_B mix was cured in laboratory condition. Figure 5-9 shows a dramatic reduction in the drying shrinkage behavior of the C2-SC_B mix design. The drying shrinkage was reduced by 83% at the end of the 56-day test in this case. Thus the lab cure on mix designs containing the SCC/SCA has resulted in a reduction of drying shrinkage of 106% at 28 days, and 83% at 56 days, as indicated in Table 5-2. The efficiency of SCC/SCA to minimize drying shrinkage is therefore highly sensitive to the applied curing method. By using the performance grading summarized in Table 3-11, it can be seen that the drying shrinkage of concrete containing the SCC/SCA and lab cured may be classified as Grade 3 concrete, i.e. the highest performance level in terms of shrinkage.

5.2.2.3 Impact of the SRA

The impact of the SRA on drying shrinkage is showed in Figures 5-10(a) and 5-10(b). Figure 5-10(a) describes the drying shrinkage behavior of the mix designs at the low cement content subjected to dry and moist cures, and Figure 5-10(b) shows the same effect for the trial batches at high cement content. The figures suggest that the SRA is very effective in reducing the drying shrinkage of concrete for the mix designs in question. Table 5-2 indicates that these mix designs had their drying shrinkage reduced by at least 30%.

Table 5-2 further suggests that, for trial mix designs at higher cement content containing only the SRA, a moist cure (as defined in this study) would further reduce the drying shrinkage by 15%. The SRA is very effective in reducing the drying shrinkage of concrete regardless of the curing method applied, and yet the moist cure contributes effectively in assisting the SRA in further reducing the drying shrinkage of concrete. Thus the SRA added at a rate of only 1% by cementitious mass can achieve high performance level in reducing drying shrinkage, and its effect is further improved by applying moist curing. Using Table 3-11, the use of SRA and moist cure to reduce drying shrinkage, for concrete at both cement levels, indicate that a performance grade of 3 can be achieved, the best performance grade that can be given for shrinkage behavior. Without any curing applied to the SRA concrete, the achieved performance grade would still remain a 3 for low cement level, while the achieved grade for the high level cement would be a 2. This simply means that at the concrete mix design level, parameters can be adjusted to obtain desired performance levels for drying shrinkage.

5.2.2.4 Effect of Various Combinations of SCC/SCA, SRA, and Fly Ash

The combinations of SCC/SCA, SRA, and fly ash and their effects on drying shrinkage are showed in Figures 5-11(a) thru 5-11(b). As previously described, Figure 5-11(a) is for trial batches at low cement content, while Figure 5-11(b) is for trial batches at high cement content. The combination of the SCC/SCA with the SRA yield the most pronounced effect on drying shrinkage. At both cement levels, the drying shrinkage was dramatically reduced and outperformed the drying shrinkage behavior of all other mix designs. As an example, a quick reference to Table 5-2 shows that combining the SCC/SCA with the SRA reduce the drying shrinkage of concrete at 28 days by 50% for C1, and by 82% for C3, when moist cured. Again these excellent results reflect the efficiency of the moist cure to provide sufficient hydration to the SCC/SCA during the crucial first 3 to 7 days of curing.

The reduction of drying shrinkage was even more pronounced for the same mix designs when fly ash was added. Drying shrinkage at 28 days was reduced by 84% for the low cement mixes, and by 134% for the high cement mixes. Therefore, the combinations of SCC/SCA, SRA, and fly ash show outstanding reducing effects in drying shrinkage of concrete, even exceeding those made with the SCC/SCA alone, or those made with the SRA alone. These excellent results indicate the effectiveness of both the SRA and the applied moist cure to enhance the expansion process of the SCC/SCA during the hydration process, and to effectively reduce the heat of hydration in the first three days (as will be shown later). The crucial factor that yielded these outstanding results was the application of the moist cure. With this excellent shrinkage behavior, concrete containing both the SCC/SCA and the SRA can easily be assigned a shrinkage performance grade of 2 when dry cured, and a shrinkage performance grade of 3 when moist cured (see Table 3-11).

5.2.3 Flexural Strength (ASTM C 78)

The flexural strength of all the mix designs is shown in Figures 5-12(a) and 5-12(b). Figure 5-12(a) shows the flexural strength of all the trial mix designs at the 28- and 56-day tests. Figure 5-12(b) shows the flexural strength of all the trial mix designs at the 3- and 7-day tests. This separation of the test data was done for clarity. Table A5-3 in the appendix summarizes the test results for the tensile strength of concrete based on the modulus of rupture.

Tensile strength equations based on modulus of rupture provided by ACI 363[134] were used for comparison purposes. Two equations were plotted against the results from the modulus of rupture tests. One curve represents the typical upper bound of expected tensile strength, while the second curve represents the lower bound of expected tensile strength, both of which are based on the modulus of rupture of concrete.

The modulus of rupture at 28 and 56 days of all trial mix designs is shown in Figure 5-12(a). Equations from ACI 363 were used and plotted on the same figure for comparison purposes. These equations predict the tensile strength of high strength concrete as measured by the modulus of rupture. The expressions for the equations are shown in Figure 5-12. The figure suggests that all the trial mix designs yielded satisfactory tensile strength, and fall within the bounds set by the equations of ACI 363.

As Figure 5-12(b) indicates, the early age flexural strength of concrete for all trial mix designs, based on the modulus of rupture, varied from 2 to 4 MPa (290 to 580 psi) at the 3-day test, while it varied from 2.6 to 4.3 MPa (377 to 624 psi) at the 7-day test. At the 3-day test, the compressive strength of the corresponding trial mix designs ranged from 17 to 36 MPa (2465 to 5221 psi), and from 25 to 44 MPa (3626 to 6382 psi) at the 7-day test. Comparing this figure to Figure 5-12(a) shows that variations in modulus of rupture values are more pronounced at early age than at 28 and 56 days. The data does not clearly show the effects of curing on the modulus of rupture. It is important to note that many materials and ambient parameters affect the modulus of rupture of concrete at very early age.

These results further suggest that the SCC/SCA and the SRA concrete, regardless of the curing procedure applied, produce acceptable flexural strength based on the modulus of rupture, and did not affect in any detrimental way the evolution of the modulus of rupture. It can be deduced from Table 3-11 that there is no recommended performance parameter associated with the modulus of rupture of concrete. While Table 3-11 is based on 28-day tests, it would be relevant here to consider a parameter associated with the modulus of rupture at early age.

5.2.4 Cracking Tendency (NCHRP Report 380)

The cracking tendency test was conducted on two specimens for each mix design in the first part of the research. One specimen experienced a poor curing regime, while the other specimen experienced the good curing regime. The strain in the steel rings was monitored for 21 days. A visual inspection for cracking of the concrete ring was conducted daily for 21 days.

The strain gage data did not indicate any concrete ring cracking; except for the control mix design C3 (see mix description in Table 4-13). A visual inspection of the two C3 concrete rings showed the cracks fully developed across the ring and down its depth. The visual observation was recorded 20 days after the cast of the concrete rings. It must be noted that previous to the 20-day visual inspection, no apparent cracks were recorded. The strain data for the control mix design C3 is shown on Figure 5-13(a). The data is temperature compensated. As indicated earlier, the strain data was monitored for 21 days. However, only the first three days are shown in Figure 5-13. The data showed no significant variations in the steel strain after the initial 3 days after the concrete was cast. The data shows clearly that during the first 24 hours, the specimens experienced as much as 5.9 ?s/hr of total shrinkage (i.e. thermal, drying, plastic, and autogenous shrinkage). This translated in a total shrinkage of at least 142 ?s in the first 24 hours after casting. Though the cracking of these specimens cannot be fully explained in terms of the shrinkage rate experienced in the first 24 hours, it must be noted that if this rate remained constant during the moist curing time, assuming a linear trend for the total shrinkage development coupled with leveling of the shrinkage rate at 4 ?s/hr by increment of 0.5 ?s for every day, the total shrinkage at 7 days would well reach at least 495 ?s, which would be an unacceptable level of shrinkage performance for concrete at that time.
Using Figure 5-13(a), a sharp drop in strain intensity is recorded at 24 hours. This sharp drop indicated that the tensile stresses induced by drying and thermal shrinkage stresses have overcome the tensile capacity of the concrete at 24 hours. This behavior was not recorded with the rest of the mixes. The strain data and visual inspection data suggest that the cracking started at the micro level and propagated over the 20 day period before any visual confirmation of the cracking could be observed. Therefore, the first 24 hours during which the concrete shrinkage is tested through the ring test could reveal suspicious trends that could lead to obvious signs of cracking of concrete at later times.

None of the other trial mixes showed signs of cracking tendency. The strain data did not provide additional useful results associated with cracking tendency. The daily visual inspection for cracks further complemented the strain data result, in that there was no recorded cracking in any of the concrete rings, except the two rings representing the control mix design C3. As an example, the ring strain for the low cement content C1 control mix design steadily increases and remains steady after approximately 24 hours, and experienced no drop in strain and no visual cracking was recorded. This behavior can be seen in Figure 5-13(b). Furthermore, the same figure suggests that the mix designs containing the SRA, and the SCC/SCA induced no significant strain in the steel ring. The variations of up to 30 ?s observed for each mix design are attributed to temperature effects.

It is therefore critical to carefully monitor the steel ring strain in the first three to seven days. The strain data may indicate a tendency for cracking if the strain data shows a sharp drop in strain as Figure 5-13(a) does. This sharp drop in the ring strain may well provide additional information as to the sensitivity of a trial mix design to micro cracking as early as 24 hrs after casting, but the daily visual inspection can only be used to monitor the time-to-cracking of a particular mix design as the micro cracking develops through the full depth of the concrete ring. It is important to note that no other trial mix designs induced similar strain drop behavior, and that no time-to-cracking was recorded from the daily visual inspection of the concrete rings.

Thus, as far as this research is concerned, the time-to-cracking recorded at 20 days for the control mix design C3 is by no means indication of the actual cracking of the concrete. The cracking started well in the early age phase of the concrete cure, i.e. within the first three days when the ring strain drop took place.

5.2.5 Heat of Hydration (NCHRP Report 380)

Each trial mix design for phase one was tested for heat of hydration. A thermocouple was placed in each concrete ring and the temperature was measured every 15 minutes for 21 days. Because of the particular interest in observing the temperature evolution in the concrete for the first 3 days of curing, it was decided to select the first 3-day temperature data of all trial mix designs and produce comparison plots. Temperature data for the first three days was sufficient for this study as the early age concrete temperature was before and after the peak heat of hydration is reached was capture during that time, which was the most important data to capture in this test. Figures 5-14(a) and figure 5-14(b) are plots of the temperature data of all the mixes designed at the low cement level, and subjected to both dry and moist cure respectively. As Figures 5-14(a) and 5-14(b) show, the SRA and the SCC/SCA effectively reduce the peak heat of hydration. The most effective reduction in the peak heat of hydration came from the combination of the SCC/SCA and the fly ash for the low cement content mix designs, while the combination of the SCC/SCA, SRA, and fly ash was most effective in reducing the peak heat of hydration in the trial batches of high cement content as shown on Figures 5-15(a) and 5-15(b).

Additionally, for trial batches at the low cement content, the SRA effectively lengthened the time to reach the peak heat of hydration by at least 3 hours, which suggest a longer time to setting of concrete, and significant reduction of thermally induced strains in the concrete, improving the tensile strain development of the concrete during the critical 3 to 7 days after casting by reducing the rate at which the tensile strength of concrete is developing, yet reaching adequate tensile strength at 3 and 7 days, as Figure 5-12(b) indicates. Note also that it was observed that the mix designs at the high cement content do not show the same effects caused by the SRA, except when fly ash was added to the SRA. This confirms the effectiveness of fly ash in reducing the peak heat of hydration and lengthening the time to reach the peak of hydration. The reasonable explanation of the ineffectiveness of the SRA producing the same effects observed at the low cement content, comes from the cement content itself. Cement content has the effect of increasing the rate of hydration, thus increasing the rate at which the peak of hydration is reached. However, Figures 5-14 and 5-15 show clearly that adding fly ash and the SRA can minimize the cement content effects during the hydration process.

At the low cement content, Figure 5-14 show that the SRA tends to keep the temperature of concrete to a low range, lower than the other trial batches, regardless of the applied curing. After the three days of moist cure, the concrete temperature for all trial batches move towards the ambient temperature. Again, this effect was not observed for trial mixes at the high cement level. (The average ambient temperature in the basement of the structural laboratory during the entire Phase 1 was 22.8➤ C (73➤ F)).

The SCC/SCA and fly ash produced the same effects as the SRA did during the hydration process. Both the heat and peak heat of hydration were reduced when the SCC/SCA was used with the fly ash. Nevertheless, the dry cure and cement content affected the time at which the peak of hydration was reached for these mix designs. As Figures 5-15 suggests, at high cement content and regardless of the curing regimes, the peak of hydration was reached faster than the control mix designs. This was not observed for the low cement specimens. This effect of shortening the time to peak heat of hydration at high cementitious content when using the SCC/SCA can cause practical problems on the construction site during casting of the concrete in hot weather. Combining the SCC/SCA with the SRA produced lower heat of hydration but no significant retardation in the time to peak heat of hydration.

5.3 Selection Process

With the results collected from Phase 1, it can be seen that both the SCC/SCA and the SRA reduce both the drying and thermal shrinkage of concrete. Before proceeding to Phase 2 of the research, the results from Phase 1 were interpreted and the mix designs that provided an excellent overall performance were selected for testing using the procedure described for Phase 2 (see Tables 4-11 and 4-12). Because both the SCC/SCA and the SRA were very efficient in reducing drying and thermal shrinkage, the final selection was based on a single criterion, which was strictly economic.

For the purposes of this research, the results from part one show that both the SCC/SCA and the SRA effectively reduce the thermal and drying shrinkage of concrete. Results also indicate that the SCC/SCA and the SRA produce excellent hardened properties of concrete, namely compressive strength and flexural strength based on the modulus of rupture. The SCC/SCA and the SRA also produce minimum effect in the steel ring strain data, indicating virtually no cracking tendency. However, the SCC/SCA shows sensitivity to ambient temperature, concrete temperature and hydration process. Practically, it implies that additional precautions must be taken to ensure that workability is maintained for a sufficient amount of time during casting of the concrete.

In terms of economic benefits, both the SCC/SCA and the SRA increase the unit cost of concrete. For an SRA or an SCC/SCA concrete to be cost effective, it must show that it will lengthen the life expectancy and reduce the rehabilitation cost of a concrete structure. In addition, it must be shown that the initial cost increase for the concrete is insignificant when compared to the long-term rehabilitation costs. Because of a rather large amount of variables that come into the cost analysis, a comprehensive analysis is not presented. An initial cost analysis of concrete per unit volume was performed. After consulting NDOT, local concrete producers, and manufacturers of the SCC/SCA and the SRA, a rough initial cost analysis was performed.

With the information provided, at the dosage rate investigated in this research, the SRA would cost an additional \$26/m^3 (\$20/yd^3) of concrete. It is anticipated that the unit cost decrease as the SRA is used more and more in the concrete industry, especially for highway superstructures. In addition, the SRA manufacturer recommended that Darex II be used with the SRA for better results. If no significant difference in the AEA dosage is required, the use of the specific AEA would not drive any cost difference, while significant AEA dosage requirements would certainly induce significant additional cost. In contrast, the SCC/SCA would cost an additional \$39/m^3 (\$30/yd^3) of concrete (does not include cost incurred by the concrete plant for providing a separate silo for the SCC/SCA).
Assume that a typical bridge in Nevada has dimensions of 52 ? 12 ? 0.25 meters (170 ft. ? 40 ft. ? 10 in.). That corresponds to a deck volume of 156 m^3 (204 yd^3). Assume also that only 10% of the deck surface has cracked. Common repairs using methyl methacrylate crack healers, and overlay, both rate at an average cost of \$43/m^2 (\$4/ft^2). The core sampling to determine the extent of the crack depth costs on average \$500 per bridge. Bridge deck surface repair costs amount to \$5,490. The additional initial cost for the usage of the SRA in concrete amounts to \$4,056, and the use of the SCC/SCA for concrete would initially cost \$6,084. It should be noted that these

costs do not include factors such as traffic control (can be as high as ½ the cost), engineering time, loss of traffic lanes, cracks that are not repaired but present in other areas of the deck, freight costs, and others.

Using the initial cost analysis presented above, and results of fresh and hardened properties of the trial mix designs, it is evident that trial batches containing the SRA have an economic advantage over the SCC/SCA. However, it must be stressed that beyond the economic advantages, both the SCC/SCA and the SRA provide excellent measures to reduce drying, thermal, and plastic shrinkage of concrete at early-age. Therefore, based on the economic analysis alone, the SRA was selected for Phase 2.

5.4 Test Results: Phase 2

Phase 2 consisted of reproducing selected mix designs showed in Table 5-1 along with their corresponding control mixes. They were tested according to the procedure describing Phase 2 (see Table 4-12). The results for the fresh properties of concrete are provided in Table 5-3. Table 5-3 provides a summary of the air content, slump, and fresh temperature of concrete, as well as the final dosage for the AEA and HRWR.

5.4.1 Compressive Strength

Table A5-4 in the appendix presents the results for the compressive strength test performed in Phase 2. Figure 5-16 shows the results for compressive strength compared with results from Phase 1. Table 5-4 summarizes the reduction in compressive strength when the SRA is added to the mix. When comparing Tables 5-1 and 5-4, the data suggests similar reductions in compressive strength. It shows consistency in the reproduction of the mix designs, and the reduction of concrete compressive strength when the SRA is present in the mix design. Also, lab cured did not significantly improve the compressive strength of concrete containing the SRA. As far as Phase 2 is concerned, typical reduction of 2 to 7% in compressive strength was observed for mix designs with the SRA alone, while reduction of 6 to 10% in compressive strength was observed when the fly ash was added. These results are consistent with results observed in Phase 1.

5.4.2 Drying Shrinkage

The drying shrinkage test results for this phase are summarized in Table A5-5 of the appendix. Figure 5-17 is a plot of the results, and as this figure suggests, the variations in drying shrinkage were similar to that observed in Phase 1. This trend suggests that the SRA, used at 1% as it was in this research, effectively reduces the drying shrinkage of concrete. The reduction in drying shrinkage is summarized in Table 5-5. As Table 5-5 suggests, the use of the SRA at 1% reduces the drying shrinkage by at least 23%, which was also observed when the fly ash was added. Drying shrinkage was reduced by at least 26% at 28 days, while it was reduced by at least 22% at 56 days. Using Table 3-11, the mix designs of Phase 2 receive the highest shrinkage performance ranking, i.e. a grade of 3.

5.4.3 Modulus of Rupture

A summary of the results from the modulus of rupture test is presented in Table A5-6 of the appendix. Figure 5-18 shows the data of the modulus of rupture as a function of the cylinder strength. Figure 5-18 also features the lower bound curve from the ACI 363 Equation. The data clearly suggests, as in Phase 1, that the modulus of rupture is also affected by the addition of the SRA, and also the addition of the fly ash. But as Figure 5-18 also shows that the modulus of rupture reaches acceptable strength at the 28-day test, as seen by the fact that the modulus of rupture of all the mix designs is found within the boundaries set by ACI 363.

5.4.4 Chloride Ion Penetration

As Table 4-12 indicates, this test was performed in Phase 2. The University of Nevada Reno tested the mix design identified as C2-SC_B, while W.R. Grace Analytical and Technical Services Laboratory tested 4 mix designs, as indicated in Table 4-11. The results from the chloride ion penetration test are summarized in Table A5-7. Four sets of 100 ? 200 mm (4 ? 8 in.) concrete cylinders were submitted to W. R. Grace for the Rapid Chloride Permeability Test (ASTM C 1202). Two 100 ? 50 mm (4 ? 2 in.) slices were taken from one cylinder and randomly identified as 1 and 2 and tested. Rapid Chloride Permeability was performed at 56 and 90 days. Figure 5-19 shows the results of the test. As it shows, neither the SRA nor the SCC/SCA contributed to effectively increase the concrete resistance to chloride ion penetration. However, the data shows clearly that the fly ash provides additional resistance to chloride ion penetration, and that this increased resistance is enhanced between the 56 and 90 days after the concrete is cast. The mix designs that were tested for resistance against chloride ion penetration were rated according to ASTM C 1202 standards. Table 5-6 shows those ratings.

Though the research did not focus on methods to increase resistance to chloride ion penetration, a discussion of this issue was presented in Chapter 2. A brief review of the chapter indicates that with proper curing, silica fume dramatically increases the resistance of concrete to chloride ion penetration. Thus an adequate combination of fly ash and silica fume may provide desired performance for chloride ion penetration resistance, provided that moist curing is provided.

5.4.5 Hardened Air-Void System

W. R. Grace performed the hardened air-void system test. In this test, a 100 ? 125 ? 25 mm (4 ? 5 ? 1 in.) concrete slice and submitted to a procedure called the Linear Traverse method. This method consists of determining specific parameters, which are used to calculate the air content of the concrete. It was shown[135] that this procedure produces results that are in good agreement with results obtained from other ASTM Test Methods, namely the ASTM C 138[136], ASTM C 173[128], and ASTM C 231.[137]

Only the four mix designs containing the SRA selected for Phase 2 were submitted to this test. These mix designs are identified as C1-SRA, C1-FA-SRA, C3-SRA, and C3-FA-SRA. The results for this test are presented in Table A5-8 in the appendix. Table 5-7 compares the test results to the fresh air content of the corresponding concrete mix designs and recommendations for hardened air-void system set by ASTM C 457 and ACI 201[138].

It is clear from Table 5-7 that the hardened air content of concrete is affected by the addition of the SRA. In this research, it was found that the typical absolute reduction of air content ranged from 0.9% to 1.55%. Table 5-7 also suggests that the addition of fly ash adds stability to the air void system of the concrete at its hardened stage. Comparing the test results with the ASTM and ACI requirements presented in the last column of Table 5-7 indicate that the mix designs identified as C1-FA-SRA and C3-FA-SRA met all the requirements for hardened air-void system, except in the total air content A_C. This reduction in air content can be adjusted by increasing the AEA dosage to compensate for the 0.9 to 1.55% typical absolute air content reduction when the SRA is used for concrete bridge decks.

As ASTM C 457 indicates, the parameters of the air-void system in hardened concrete depend essentially on air dosage and the type of AEA used, the degree of consolidation of concrete, and the W/C ratio used. Furthermore, in order to obtain suitable air content, it is important to have a high enough plastic air content and low spacing factor so adequate protection against freezing and thawing is provided.

5.5 Test Results: Phase 3

5.5.1 NDOT Mix Designs

At the request of NDOT, an Engineering Consultant prepared three specific mixture designs, which are summarized in Table 5-8. Variables investigated in these mix designs were W/C ratios and cementitious content. The University of Nevada Reno completed specific tests on specimens from each of the three mix designs. The tests included drying shrinkage, compressive strength, and resistance to chloride ion penetration. The results are presented in Table A5-9. A feature of these mix designs, which separate them from all mix designs presented in this research and must be noted here is the use of the Boral Linx. Boral Linx is a liquid admixture that is added to the plastic concrete at the batching phase and functions as an Alkali-Silica reaction inhibitor. Because ASR expansion was not a focus of this research, no tests regarding the effects of Boral Linx on these mix designs is presented in this research.

Based on the performance grading system (Table 3-11), the NDOT mix designs did not yield a high grade both in drying shrinkage and resistance to chloride ion penetration. Table 5-9 shows that NDOT mix designs were assigned at best a grade of 1 in drying shrinkage performance, while the performance in chloride ion penetration received a grade of 2 at 90 days.

The tests data are plotted in Figure 5-20 for resistance to chloride ion penetration, Figure 5-21 for compressive strength, and Figure 5-22 for drying shrinkage. As Figure 5-22 clearly indicates, the drying shrinkage of the NDOT mix designs was unexpectedly high at the 28-day test, while the only improvement came in the increased resistance to chloride ion penetration from the 56-day to the 90-day test.

These interesting results seem to contradict the notion that lowering the W/C (water-cementitious) ratio of concrete would equate to increasing the resistance to chloride ion penetration. In fact there is no direct relationship between chloride permeability and W/C ratio. An extensive study[139] reported that the chloride permeability of concrete is strongly affected by the water content, not the W/C ratio. Because the water content in concrete is influenced by the maximum size aggregate (MSA), both the water content and MSA control the behavior of the resistance to chloride ion penetration.[139]

5.5.2 Steamboat Creek Bridge

Results of the drying shrinkage, compressive strength, and chloride ion penetration tests for the Steamboat bridge deck are summarized in Table A5-9 in the appendix. The results indicate relatively low chloride ion penetration, but high drying shrinkage of the concrete. It should be noted that this deck cracked within the first 56 days after casting of the concrete, as indicated in chapter 3. Figure 5-20 shows the mix design resistance to chloride ion penetration, Figure 5-21 is the plot of the compressive strength, and Figure 5-22 the corresponding drying shrinkage. As Table 5-9 shows, the Steamboat bridge deck exhibited high drying shrinkage (rating of 0) at 28 days and an increased resistance to chloride ion penetration at 90 days (rating of 1).

5.5.3 Sahara Bridge Deck

Results for the Sahara deck are presented in Table A5-9 in the appendix. The results showed moderate chloride ion penetration, while shrinkage performance was not as high as that recorded for the Steamboat creek bridge mix design. Figure 5-20 shows the resistance to chloride ion penetration, and Figure 5-22 represents the drying shrinkage behavior of the concrete used for the Sahara bridge deck. For this bridge deck, Table 5-9 indicates that drying shrinkage was rated at 2 and its resistance to chloride ion penetration was rated at 1. The Sahara deck had better performance in drying shrinkage than the Steamboat deck did.

5.6 Discussion and Impact of the Results

Early-age restrained shrinkage cracking can be prevented. The results from phase one show that the SRA and the SCC/SCA produce substantial reduction in shrinkage and early-age cracking potential. The SRA and the SCC/SCA produced lower heat of hydration, which reduced the early-age thermal shrinkage of concrete. Shah et al[139] pointed out that their experiment also studied the effect of the SRA on drying shrinkage and concluded that significant reduction in drying shrinkage was observed.

When moist-cure is applied to both the SRA and the SCC/SCA concrete, significant improvement in all the hardened properties of concrete are observed. Furthermore, the SCC/SCA concrete shows superior behavior in reducing the drying shrinkage of concrete, provided an adequate moist or wet cure.

The use of the HRWR improved significantly the workability of concrete. The use of HRWR reduced bleeding and potential for segregation in high slump concrete. In this research, the use of HRWR with the SRA produced lower air content at the hardened level of concrete. It must be made clear that no direct correlation was made between the use of HRWR with an SRA. As Aitcin[29] points out, there also exist compatibility issues between cementitious materials and the use of HRWR. The results from the hardened air-void system shall be considered in specifications and requirements for durable concrete using the SRA in concrete exposed to freezing and thawing cycles.

From the results of this research, it was not clear whether the SRA and the SCC/SCA increase the resistance to chloride ion penetration. However, the 25% addition rate of fly ash clearly showed that dramatic increase in resistance to chloride permeability is accomplished between the 56th and 90th

day of the life of concrete. Not only did the fly ash add resistance to chloride ion penetration, it also reduced the peak heat of hydration.

Though the SCC/SCA was rejected in the second phase of the study primarily because of economic considerations, this research shows that the SCC/SCA exhibited superior behavior in drying shrinkage when a 7-day moist cure was provided. The peak heat of hydration was also reduced. Based upon chapter 2, the SCC/SCA in combination with silica fume and a 7-day moist cure could show outstanding performance in drying shrinkage reduction as well as a dramatic increase in the resistance to chloride ion penetration. This research also showed that it is crucial that appropriate curing be provided so that the hydration process during the critical phase of the life of the concrete is not negatively impacted.

As a consequence of the study, it becomes clear that bridge deck cracking associated with concrete shrinkage can be successfully minimized by using appropriate measures at appropriate steps of a bridge deck construction project. The study showed that concrete with high performance characteristics associated with drying and thermal shrinkage could be produced. However, plastic shrinkage of high performance concrete is as equally important to consider in successfully minimizing the overall shrinkage behavior of concrete used for bridge deck. The next chapter discusses why plastic shrinkage must be addressed and how it could be minimized using appropriate curing requirements.

By addressing the most critical parameters (namely drying, thermal and plastic shrinkage, and curing requirements) that affect early age shrinkage of concrete, recommendations are proposed. These recommendations are provided in Chapter 7 of this document.

CHAPTER 6

Considerations of Weather Effects in Nevada

6.1 Introduction

The durability of concrete bridge decks is directly related to deck cracking. It was shown in Chapters 2 and 5 that the quality of curing greatly affects the extent of the deck cracking, and therefore has a pronounced impact on the durability of the deck. The adequate procedures required to cure bridge decks are determined by many parameters. One of the significant ones is the weather condition at the time of cast and during curing. For example, freshly placed concrete exposed to hot, windy conditions is prone to plastic shrinkage cracking. The objective of this chapter is to analyze the weather patterns observed throughout Nevada and evaluate its impact on weather-related curing requirements.

Weather data was obtained from the Western Regional Climate Center (WRCC), located in Reno Nevada, and the National Climatic Data Center (NCDC). The data consisted of surface daily data from four stations representative of weather conditions throughout Nevada. The four stations selected for the analysis were Reno, Ely, Tonopah, and Las Vegas. As advised by the WRCC, these four stations best represent the State's four climatic regions, and have better quality of data.

The procedure for the data analysis that is presented in Section 6.6 is an adaptation of a study presented by Carden et al.[140] The weather data was analyzed, evaluated, and used in conjunction with ACI 305[33] Evaporation Chart. The evaporation rate curves were developed using formulations proposed by the Federal Highway Administration[141] (FHWA) and other studies.[142-143] The curves were categorized with respect to weather exposure conditions specific to the climatic regions. Finally, curing requirements categories for bridge deck placements for Nevada are proposed.

6.2 NDOT Specifications

NDOT Specifications[110] 501.03.09 contain requirements for curing. NDOT Section 501.03.10 contains specifications regarding weather limitations. Both sections specify provisions for limiting concrete temperature at placement in cold and hot weather. The use of curing compound is also addressed in NDOT 501.03.09. Provisions for concreting in cold and hot weather are summarized in NDOT 501.03.10. As an example, under the hot weather concreting (NDOT 501.03.10(d)), the only parameter that must be controlled is the concrete temperature, which is not to exceed 32 ► C (90 ► F) immediately before placement.

The specifications provided by NDOT do not formally address the issue of the evaporation rate of concrete and the preventive measures to minimize the effects of evaporation on plastic shrinkage, as affected both by the concrete temperature and the ambient parameters (i.e. air temperature, relative humidity, and wind speed). The objective of this chapter is to determine if there is a need to take into consideration weather effects and their impact during the first 7 days of the life of a concrete bridge deck.

6.3 Environmental Factors

The environmental factors that present a serious challenge during a concrete bridge deck construction are ambient temperature, wind speed, and relative humidity. These factors constitute the most critical parameters that affect plastic shrinkage during curing of the concrete.

Temperature is easy to measure. In general, this parameter is recorded at all weather stations, thus the data is readily available from the WRCC or the NCDC. Wind speed can be measured or estimated using tools such as an anemometer or a ventimeter. A very important aspect of wind measurement is that it should always be measured as close to the concrete surface as possible, as it varies with height.[144] Menzel[144] states that the wind speed should always be measured at a level approximately 500 mm (20 in.) higher than the evaporating surface. Uno[143] points out that the ACI 305 nomograph does not feature Menzel recommendations for wind measurement adjustment, and proposes a wind speed guide that accounts for this adjustment. This guide is presented in Table 6-1. The standard height of 10 meter (30 ft.) is also shown in the table, as it is the standard height at which wind speed is measured at weather stations. This table can prove to be useful to quickly assign a category to wind before and during casting of a concrete bridge deck. Wind speed and relative humidity readings can also be obtained from the WRCC or the NCDC. Relative humidity can be measured directly on site by means of a handheld psychrometer or a wet bulb-dry bulb thermometer.

6.4 Curing Requirements

Normal or high performance concrete must be cured. Certainly no one has expressed doubts about the vital needs for curing requirements. Unfortunately, it does not mean that concrete is always well cured, or that appropriate curing requirements are followed or enforced by contractors, mainly because of economic considerations or lack of incentives. It has been shown that to obtain the highest degree of hydration and the lowest permeability, water curing must be applied.[35]

The importance of appropriate curing is most appreciated when the hydration process is well understood. It must also be recognized that the best curing is not always practical. Curing should be carried out to maximize hydration and minimize shrinkage.

It has been common to specify a curing compound to apply on the surface of the freshly placed concrete to minimize its evaporation rate and magnitude. The impact of curing compound has been studied in the laboratory but never clearly studied in the field.[95] If the concrete that is protected with the membrane formed by the curing compound has a high W/C ratio, hydration is more likely to be completed with the initial mixing water and shrinkage is minimized as long as the curing membrane is present.[95] In contrast, if the W/C ratio is low and a curing membrane is present, self-desiccation develops rapidly throughout the concrete. The lower the W/C ratio the higher the autogenous or chemical shrinkage. This explains why high performance concrete (i.e. low W/C ratio) always has higher initial shrinkage than high W/C ratio concrete. If the curing membrane disappears then drying and plastic shrinkage are very likely to develop rapidly. Thus curing compounds are effective as long as the membrane has not been altered or has not disappeared.[29]

If durable high performance concrete bridge decks are to be built, precautions must be taken to minimize the effects of evaporation associated with plastic shrinkage. Thermal, drying, and plastic shrinkage are all interconnected, and this interconnection starts at the beginning of hydration. Because of the numerous parameters associated with hydration effects, it was decided to focus on the effects of ambient conditions on the hydration and evaporation of concrete at placement and during curing.

6.5 The ACI 305 Evaporation Rate Chart

A historical background on the development of the ACI 305 Evaporation Chart is provided by Uno.[143] The ACI 305 Report essentially states that there exists a direct correlation between the evaporation rate of concrete and plastic shrinkage. The report further explained the implication of ambient condition parameters in the evaluation of the evaporation rate at a specific site during a concrete bridge deck placement. The parameters were identified as ambient temperature, wind speed, and relative humidity. Figure 6-1 shows the Evaporation chart published by ACI Committee 305. To the left of the chart is found an easy instruction on how to use the chart, provided that ambient temperature, wind speed, and relative humidity are measured.

Another study[145], in confirmation of statements made in ACI 305,[33] showed that in effect weather parameters cause the following to take place:

?? If ambient temperature and relative humidity are constant and wind speed increases from 2 to 9 m/s (5 to 20 mph), the evaporation rate experiences a 300% increase.

?? If the ambient relative humidity and wind speed are constant, and the ambient temperature increases from 15 to 32➤ C (60 to 90➤ F), the evaporation rate experiences a 300% increase.

?? If the ambient temperature and wind speed remain constant, and the ambient relative humidity decreases from 90 to 70%, the evaporation rate increases by 300%.

??

It is clear from these conclusions mentioned above that real ambient conditions are a combination of these statements, and that the evaporation rate is significantly affected by any combination of the weather parameters.

6.6 Weather Patterns in Nevada

In order to appreciate the effects of weather parameters on evaporation rates, surface daily data of ambient temperature, wind speed, and relative humidity were collected from the WRCC, and the NCDC provided hourly data of the same parameters. The stations were Reno, Ely, Tonopah, and Las Vegas as shown on Figure 6-2. These stations were selected at the recommendation of the

WRCC. No stations located further north than Reno or Elko could provide sufficient data to be included in this research. The daily data spanned from 1984 to 2000 (data before 1984 is incomplete), and the hourly data selected were that of the months January and July in order to provide a global overview of both cold and hot weather patterns in Nevada.

6.6.1 Geographical Variations

The geographical variations were analyzed and Figure 6-3(a) shows such variations. As Figure 6-3(a) suggests, except for Las Vegas, there are no significant temperature variations with Reno, Ely, and Tonopah. Data for the wind speed, as shown on Figure 6-3(b), also suggest minimal geographical variations. Unlike ambient temperature and wind speed, the geographical variations for relative humidity are more evident, as Figure 6-3(c) shows. Clearly, ambient relative humidity is lower in Southern Nevada.

6.6.2 Monthly Variations on Annual Basis

Figure 6-4 shows the monthly variations in the ambient parameters on an annual basis. As Figure 6-4 suggests, and contrary to intuitive and empirical bases, the wind speed is more pronounced during months where both the temperature is colder and relative humidity is higher. During hot weather (i.e. in the months of July thru September, see Figure 6-4(a).), the relative humidity is at its lowest and the wind speed is decreasing but still remains at uncomfortable levels if concrete were to be cast.

6.6.3 Daily Variations

Daily variations in the weather parameters were more pronounced than the geographical ones. To appreciate these effects, the months of January and July were selected as representative of cold and hot months in Nevada respectively. After inspection of Figures 6-3 and 6-4, the four stations, which represented each of the four climatic regions for Nevada, were narrowed down to two, namely Reno and Las Vegas. Reno was selected as representing Northern Nevada, and Las Vegas was chosen to represent Southern Nevada. Hourly surface data for the Reno and Las Vegas stations and for the months of January and July were selected. The data spanned a period from 1984 to 2000. Daily variations are shown in Figure 6-5 for Reno and Figure 6-6 for Las Vegas. It can be observed from Figures 6-5 and 6-6 that daily variations in wind speed are more pronounced for both Reno and Las Vegas, and even more important during the hot days, i.e. during the summer time. It is also clear that as temperature and wind speed increase in the afternoon, the relative humidity decreases at the same time.

6.7 Rate of Evaporation Formulations

Unpredictability and uncertainty associated with the ambient parameters and how they would dictate the weather conditions cannot be controlled, but preventive measures can be taken to minimize their effect on concrete durability. The evaporation chart provided by the ACI 305 intends to provide engineers a quantitative measure of what is an acceptable evaporation rate just before and during the casting of a concrete bridge deck. Provided that the engineer is equipped to accurately measure all three ambient parameters, the FHWA[141] provided an equation to formulate the evaporation rate. The evaporation rate formula is provided as follows (in SI units):

$$E = \frac{1 + 0.2374 \cdot w}{2906} \cdot \left[C^2 + 4.762 \cdot C + 220.8 - RH \cdot \left(\frac{T^3 + 127.8 \cdot T^2 + 665.6 \cdot T + 34283}{20415} \right) \right] \qquad (6.1)$$

where:

E = Evaporation Rate in $kg/m^2/hr$,
w = Ambient Wind Speed in km/hr,
C = Concrete Temperature in °C,
RH = Ambient Relative Humidity in %, and
T = Ambient Temperature in °C.

FHWA[3] specified that anytime the evaporation rate E, calculated from Equation 6.1, would be equal or greater than 0.5 $kg/m^2/hr$ (0.1 $lb/ft^2/hr$), preventive measures needed to be taken to minimize plastic shrinkage by constructing windbreaks or enclosures to effectively reduce the wind speed, and/or using fog sprayers upwind of the placement operation to effectively increase the ambient relative humidity, and/or reducing the concrete temperature by shading the material storage areas and production equipment, cooling the aggregate by sprinkling, and/or replacing a portion of the mix water with flaked or crushed ice to the extent that the ice is completely melted during the mixing of the concrete.

On the other hand, ACI 305[33] indicates that when the evaporation rate estimated from the nomograph exceeds 1 $kg/m^2/hr$ (0.2 $lb/ft^2/hr$) precautions against premature plastic shrinkage cracking must be taken. The equation used to prepare the ACI 305 Evaporation chart is provided below in S.I. and English unit systems. Equation 6.2 is for the S.I. unit, and Equation 6.3 is for the English Unit.

$$E = 0.313 \cdot \left[0.61 \cdot e^{\frac{17.3 \cdot C}{237.3 + C}} - RH \cdot 0.61 \cdot e^{\frac{17.3 \cdot T}{237.3 + T}} \right] \cdot (0.06 \cdot w + 0.253) \qquad (6.2)$$

where

E = Evaporation Rate in $kg/m^2/hr$,
w = Ambient Wind Speed in km/hr,
C = Concrete Temperature in °C,
RH = (Ambient Relative Humidity in %)/100, and
T = Ambient Temperature in °C.

And

$$E = 0.44 \cdot \left[0.0885 \cdot e^{\frac{17.3 \cdot (C - 32)}{395.1 + C}} - RH \cdot 0.0885 \cdot e^{\frac{17.3 \cdot (T - 32)}{395.1 + T}} \right] \cdot (0.096 \cdot w + 0.253) \qquad (6.3)$$

where

E = Evaporation Rate in $lb/ft^2/hr$,
w = Ambient Wind Speed in mph,

C = Concrete Temperature in °F,
RH = (Ambient Relative Humidity in %)/100, and
T = Ambient Temperature in °F.

As can be observed, Equations 6.1 thru 6.3 are rather complex and not user-friendly. For the engineer on site, the FHWA equation does not require any more parameters than what can be recorded with adequate instrumentation, but the computation assumes the use of S.I. units only (which would require conversion) and if the formulation is not carefully integrated in the calculator, the task can become somewhat tedious and easily lead to errors in computation. The ACI equations have already been plotted in the form of the Evaporation Rate Chart.[33]

Uno[143] proposed an alternative formula, which is much easier to integrate in the calculator, and which require the same parameters. The evaporation rate formula is provided both in the S.I. and English unit systems. For the S.I. unit system,

$$E = 5\left(\left(C + 18\right)^{2.5} - RH\left(T + 18\right)^{2.5}\right)\left(w + 4\right)\times 10^{-6} \qquad (6.4)$$

where

E = Evaporation Rate in $kg/m^2/hr$,
w = Ambient Wind Speed in km/hr,
C = Concrete Temperature in °C,
RH = (Ambient Relative Humidity in %)/100, and
T = Ambient Temperature in °C.

For the English unit system, Equation 6.4 converts to Equation 6.5 as follows:

$$E = \left(C^{2.5} - RH\,T^{2.5}\right)\left(0.4w + 1\right)\times 10^{-6} \qquad (6.5)$$

where

E = Evaporation Rate in $lb/ft^2/hr$,
w = Ambient Wind Speed in mph,
C = Concrete Temperature in °F,
RH = (Ambient Relative Humidity in %)/100, and
T = Ambient Temperature in °F.

Equations 6.4 and 6.5 clearly simplify the process of quickly calculating the evaporation rate before and during the construction process, especially when casting of the concrete bridge deck is in process. It was further shown[143] that the application of the simplified equations, namely Equations 6.4 and 6.5, for determining the evaporation rate are in good agreement with the nomograph values of the ACI 305 Chart.[33] This is illustrated in Figure 6-7. Figure 6-7(a) shows the evaporation rate curves at varying concrete and air temperature for the ACI, FHWA, and Uno Equations. Figure 6-7(b) shows the same curves at varying relative humidity, and Figure 6-7(c) also shows the evaporation rate curves at varying wind speed.

A comparison of the evaporation rates calculated from Equations 6.1 thru 6.5 is presented in Table 6-2. Table 6-2 becomes a quick reference guide that would provide the engineer with already tabulated evaporation rates. It should be noted again here that it is general guideline to provide preventive measures to minimize plastic shrinkage when the values calculated and presented in Table 6-

2 are greater or equal to 0.5 $kg/m^2/hr$ (0.1 $lb/ft^2/hr$) as per the FHWA recommendation, and greater or equal to 1.0 $kg/m^2/hr$ (0.2 $lb/ft^2/hr$) as per the ACI 305 recommendation.

The variations of the parameters in Table 6-2 were based on the curves produced in Figure 6-7. Based on the ranges of air temperature, wind, and relative humidity found in Nevada, almost all conditions are above the maximums stated by ACI and FHWA. The results suggest that precautions should be provided regardless of the assumed scenario.

The reality is that weather is extremely unpredictable in Nevada, as anywhere else. How useful will Table 6-1 and Table 6-2 be to the engineer on site and the contractors? How should these measures be used in pre-construction meetings? The answer lies in the fact that Nevada weather, regardless of the time of the year, produces weather effects that strongly reflect the need of precautions. Thus during the year, as all conditions produce evaporation rates of as much as 0.5 $kg/m^2/hr$ (0.1 $lb/ft^2/hr$), the agencies involved in a concrete bridge deck cast project should make precautions mandatory.

6.8 Evaporation Rate Curves for Nevada

The parameters that are needed to estimate the Evaporation rate curves are concrete temperature, air temperature, ambient relative humidity and wind speed. It was described how each of the ambient reading may be recorded during casting of a deck. It was also shown in Chapters 4 and 5 that concrete temperature can easily measured as well, provided the engineer has adequate equipment. The data of all four parameters and a quick calculation of the evaporation rate on site will only provide the evaporation rate at that time. It will most likely be continuously changing during placement of the concrete. At times, the evaporation rate may well be below the imposed limits, or at other times exceed the limits, but the recording can be used at a later time for modifying the curing conditions.

The following subsections will investigate the real-time effect of varying concrete temperature, and varying ambient parameters on the evaporation rate of concrete. To accomplish this, the temperature evolution of concrete was provided by the hydration tests performed in this study. The data shown on Figure 5-15(b) for the control mix design identified as C3 represents the concrete temperature evolution during the first three days after casting. The first 24 hours of the temperature data was used for the analysis. The data from the ambient parameters came from the hourly data analyzed and shown on Figures 6-5 and 6-6. With data from these curves as input variables, Equation 6.4 was used to create the evaporation rate curves of concrete for Reno and Las Vegas both in cold and hot weather, and based on the first 24 hours immediately after the concrete is cast.

6.8.1 Evaporation Rate Curves Based on Concrete Placement Time

If preliminary testing of the concrete mix designs selected for a bridge deck project is performed in conditions that are representative of the conditions that would represent the ones during the actual cast, important information can be gathered from the tests. Then the data can be used to estimate the evaporation rates as time progresses during the crucial 24 hours after casting. The evaporation rates are then reviewed and necessary measures are taken before the project starts.

Three different time of concrete placement were arbitrarily selected. The selection of these placement times was based on typical practices in the field. These placement times were midnight (12:00 am), beginning of the day (6:00 am), and at the start of the evening (6:00 pm). The evaporation rate curves were determined for both the Southern and Northern Regions in Nevada for January and July and plotted together. Results are plotted on Figure 6-8. Figure 6-8(a) shows the effect of placing concrete at midnight for a typical mix design with concrete temperature evolution resembling that of C3. Figure 6-8(b) shows the effect of waiting to cast at the start of the day, i.e. concrete placement starts around 6:00 am. Figure 6-8(c) shows the effect of starting the placing of concrete early in the evening. Figure 6-8 clearly demonstrates that starting the placement of concrete at the beginning of the evening attenuates most the evaporation rate of the concrete during its first 24 hours of curing. For example, if the cast would start at midnight (12 AM), it can be seen on Figure 6-8(a) that during the first 9 hours after the start of the cast that the ambient parameters induce minimum effects on the evaporation rate of the concrete but the evaporation rate rises rapidly thereafter and peaks in the middle of the afternoon, regardless of the time of year or location. The fact that the evaporation rate reaches its peak in the middle of the afternoon coincides with the ambient temperature peaking at the same time of day, the relative humidity being at its lowest, and wind speed being at its highest [see Figure 6-5]. In addition, the limits set by ACI and FHWA are exceeded by as much as 500% for the worst case. Truly, this is a situation that should be avoided. The situation with starting the cast at 6 AM is not so different than that of starting the cast at midnight.

While the evaporation rate at any placement time exceeds that of the ACI and FHWA, placing the concrete at 6:00 pm reduces the maximum evaporation rate the most. Thus casting at the beginning of the evening would be recommended.

6.8.2 Controlling Concrete Temperature Evolution

A concrete could be designed so that its peak heat of hydration and rate would be reduced. That is the case of mix design C3-FA-SRA. Its heat of hydration curve (see Figure 5-15(b)) was used to develop a similar curve to that of Figure 6-8(c). The result is plotted on Figure 6-8(d). It can be seen from this figure that lowering the heat of hydration of the concrete further attenuates the evaporation rate of the concrete compared to that of mix design C3 shown on Figure 6-8(c), yet still exceeds the limits imposed by FHWA and ACI 305.

6.8.3 Controlling Ambient Parameters

It can become extremely costly to try to control all the weather parameters during casting of a concrete bridge deck. Ideally, as it is done in France, [20] a box is built to become a temporary shelter for the concrete bridge deck during construction. This shelter remains there as long as the project specifications require. However, not all ambient parameters have equal impact on the rate of evaporation of concrete. To demonstrate this effect, the heat of hydration curve of concrete mix design C3-FA-SRA was used to develop the rate curves. In addition, the placement time selected for the development of the curves was 6:00 pm.

Three curves were developed. Each plot captures the effect of controlling one parameter constant during construction. Figure 6-9(a) shows the effect of maintaining the ambient relative humidity at 30%. Figure 6-9(b) shows the effect of keeping the ambient temperature at 15► C (60► F), and Figure 6-9(c) illustrates the effect of keeping the wind at the 500 mm (20 in.) height above the concrete surface (i.e. having a wind barrier or wind breaker) at 3 km/hr (1.9 mph). It can be seen from Figure 6-9 that controlling the effect of wind shows the most effective attenuations on the evaporation rate curves for Nevada. Precautionary measures should be made available to effectively control the effect of wind on the evaporation rate of the concrete deck. In an ideal situation, these precautionary measures should minimize the effects of all the ambient parameters, namely ambient relative humidity and temperature and wind speed.

6.9 Summary

Weather effects on concrete plastic shrinkage are very pronounced. Regardless of the time of the year and location, the ambient parameters have a much more pronounced control over plastic shrinkage than the concrete temperature. Equations provided by the FHWA, ACI 305, and Uno were used to develop evaporation rate curves typical of Nevada based on availability of data representing concrete temperature evolution.

Time of placement of concrete can lead to attenuations in the magnitude of the rates of evaporation during the critical 24 hours after casting, most critical time for plastic shrinkage development. Finally, wind speed was shown to be the most crucial ambient parameter in reducing the evaporation rates during construction anywhere in Nevada.

CHAPTER 7

Summary, Conclusions, and Recommendations

7.1 Summary

Concrete must be engineered to develop characteristics that provide sufficient protection against shrinkage and aggressive agents. Any micro and/or macrocracks network that will develop because of failure to produce high performance concrete will weaken the concrete and provide a path for steel corrosion, eventually leading to structural weakening.

For this study, the University of Nevada Reno (UNR) developed mix designs that addressed specifically drying and thermal shrinkage of concrete. These mix designs included the usage of commercially available shrinkage-reducing admixture (SRA) and shrinkage-compensating cement/additive (SCC/SCA). Specific ASTM Standard tests with other recommended tests were applied to the mix designs for fresh and hardened properties of concrete. The mix designs and tests are described in detail in Chapter 4. The test results are provided in Chapter 5.

In addition, UNR included tables and developed curves that addressed the evolution of the rate of evaporation of concrete subjected to weather conditions similar to that of Nevada. The Nevada Department of Transportation (NDOT), field engineers, and contractors can use the tables as quick reference guides to assess the evaporation rate of the concrete deck during the cast. The curves were developed to provide NDOT with best estimates of the evaporation rates of a low heat of hydration concrete that is placed at the beginning of the evening (i.e. 6:00 pm). These curves apply both in Northern and Southern Nevada, and account for cold or hot weather concreting.

7.2 Conclusions

A. The SCC/SCA investigated in this study showed superior capability to reduce the drying shrinkage of concrete.

B.

In this study, the SCC/SCA concrete showed excellent performance in both drying and thermal shrinkage. The high performance characteristic in shrinkage behavior was highly dependent on the applied curing method. In this study, it was found that a 7-day moist cure would be necessary to ensure that the SCC/SCA concrete exhibit excellent shrinkage performance.

Slump of SCC/SCA concrete also showed sensitivity to ambient temperature. The higher the ambient temperature, the faster the SCC/SCA concrete looses slump, which means increased dosage in superplasticizers (HRWR). Other measured fresh properties (i.e. air content and temperature) did not show the same sensitivity to higher ambient temperature.

Hardened properties of SCC/SCA concrete were found to be comparable to the control mix designs. SCC/SCA concrete showed lower compressive strength during the time of moist cure but reached similar or slightly higher strengths thereafter. It was also found that the SCC/SCA concrete showed similar resistance to chloride ion penetration to that of the control mixes.

C. The SRA investigated in this study showed excellent capability to reduce both the drying and thermal shrinkage of concrete.

The use of the SRA for application in concrete showed excellent performance both in drying and thermal shrinkage. This shrinkage performance showed to have minimal sensitivity to the applied curing regime. As a consequence of the good thermal behavior of SRA concrete, a longer time to setting was observed. As far as this study is concerned, a 3-day moist cure applied to the SRA concrete improved all the hardened properties.

Air content of the concrete was affected by the incorporation of the SRA during the design process. In the presence of an SRA, the recommended hardened air-void system that provides resistance to freezing and thawing is not stable. In this study, it was found that the SRA reduced the total air content of the concrete during its hardening.

Relative to control mixes, the SRA concrete showed lower compressive strength, but similar performance in resistance to chloride ion penetration. All other hardened properties were not negatively impacted.

D. The addition of fly ash with either the SCC/SCA or the SRA for application in concrete exhibited excellent drying and thermal shrinkage performance.

The addition of fly ash in mix designs incorporating the SCC/SCA or the SRA improved both the drying and thermal shrinkage performance of the concrete. When in addition to the SCC/SCA or the SRA, the fly ash further reduced the peak heat of hydration, and further lengthened the time to setting. In the presence of fly ash, mix designs incorporating either the SRA or the SCC/SCA exhibited excellent workability. The addition of fly ash also proved to significantly improve the resistance of concrete to chloride ion penetration.

E. The moist cure improved all concrete properties, except for mix designs incorporating the SCC/SCA.

The study showed that for all mix designs, with the exception of mixes containing the SCC/SCA, a 3-day moist cure enhanced all hardened properties of concrete. Under such curing condition, drying and thermal shrinkage were reduced. The moist cure improved the hydration process by reducing the peak heat of hydration. The moist cure provided more water to allow a more complete hydration of the cementitious materials present in the mix design. It was also observed that the concrete temperature was more stable after the initial 24 hours when subjected to moist cure. This improved stability in the temperature evolution of concrete after the first 24 hours provides a better resistance to temperature gradients.

F. Ambient weather effects on the evaporation rate of concrete would be minimized if the cast is scheduled to start early in the evening, thus minimizing plastic shrinkage.

The study showed that regardless of the time of year and the location, Nevada weather produces an environment that encourages high levels of evaporation rate of concrete. The effects of ambient conditions in Nevada during the first 24 hours of the concrete life can lead to significant plastic shrinkage. In fact, it was found that the evaporation rate of concrete well exceeds the imposed limits.

The study further showed that the time of placement of concrete is critical in order to minimize the effects of ambient conditions. It was found that starting the placement of concrete early in the evening reduced considerably the evaporation rate of concrete.

Provided that a low heat of hydration concrete is produced and placement starts early in the evening, further reduction in the evaporation rate of concrete is likely to occur and improved if wind is kept as low as possible. The study showed that acceptable levels (i.e. remain below the imposed limits) of evaporation rate could be achieved if wind is kept very low, low heat concrete is produced, and placement starts in the evening.

7.3 Recommendations

Shrinkage of concrete can be properly minimized by the use of different methodologies. These methodologies include the engineering of high performance concrete, the control of ambient conditions, the use of excellent materials, and coordination of efforts between cement producers, field engineers, admixture suppliers, and contractors. The proper combination of these methodologies will ensure that a high performance concrete is delivered on site and that a durable concrete will remain and extend the life span of the bridge deck.

Prior to providing recommendations regarding early age shrinkage and cracking of concrete bridge decks in Nevada, it must be noted that certain guiding principles must be followed. These principles include the selection of well graded aggregates, the adequate selection of cementitious content, the implementation of low water-cementitious (W/C) ratio, the utilization of superplasticizers, adequate curing time, careful use of shrinkage-reducing or -compensating admixtures, project specific challenges and needs, and good communication between cement producers, engineers, admixtures representatives, material suppliers and builders. It is fundamental to understand that a careful evaluation and combination of these principles is crucial for the production of durable concrete.

The preferred method for specifying low shrinkage concrete is to specify the desired performance from Table 3-11. Specifying shrinkage performance levels should be the preferred method for limiting shrinkage, this will allow local concrete producers to use their knowledge of the locally available materials in order to achieve the desired performance. Therefore, based on the literature research, admixture manufacturers and cement producers recommendations, and results from the laboratory experiments of this study the following recommendations are offered.

A. NDOT should consider the implementation of the SRA in concrete mix design.

The implementation of the SRA in environments where moderate and severe freezing and thawing is not a concern is strongly encouraged. The maximum recommended addition rate should not exceed 2.5% by weight of cementitious material. In addition, the SRA concrete should be designed at a minimum targeted compressive strength of 31 MPa (4500 psi) at 28 days, with a maximum W/C ratio not to exceed 0.45.

The implementation of the SRA in environments of moderate to severe freezing and thawing must be done cautiously. In addition to recommendation provided for SRA concrete in non freeze-thaw environments, it is strongly encouraged to specify a minimum of 7% plastic air content. Specific requirements for the usage of air entraining agents (AEA) and HRWR with the SRA must comply with the manufacturers recommendations for compatibility. An SRA manufacturer representative should be consulted and included in pre-concrete construction conferences and at the concrete batch plant.

Furthermore, the hardened air-void system test (ASTM C 457) should be specified when an SRA concrete is considered for application in freeze-thaw environments during the design stage of the concrete mixes. This test will ensure that requirements for adequate total hardened air content and spacing factor are met. Fly ash should also be added to an SRA concrete to achieve desirable performance in resisting chloride ion penetration.

B. NDOT should also consider the implementation of the SCC/SCA in concrete mix design.

The implementation of the SCC/SCA for concrete applications in either freeze-thaw or non freeze-thaw environments is encouraged. The SCC/SCA should be used with either Type I or II cement. Recommended addition rates of the SCC/SCA should be selected in coordination with the manufacturer after appropriate compatibility test methods between the selected cement and the SCC/SCA have been performed by the manufacturer. In any case, the SCC/SCA should replace the cement at a 1:1 mass ratio. The maximum W/C ratio should be limited to 0.45. The use of HRWR may be required to achieve a workable concrete within the specified slump range and to meet the maximum W/C ratio.

No SCC/SCA concrete should be considered for application in bridge deck without ensuring that a 7-day moist or wet cure is provided. Fly ash should also be used with SCC/SCA concrete to ensure desired level of performance in resisting chloride ion penetration.

C. NDOT should alter subsections 501.03.10(b) and 501.03.10(d) of the Standard Specifications for concrete and ambient temperature requirements.

For cold weather concreting, NDOT subsection 501.03.10(b) currently states that concrete temperature be maintained above 10► C (50► F) for the first 3 days after placement and above 4► C (39► F) in the following 4 days. It also specifies that concrete should be delivered with a temperature above 10► C (50► F) without exceeding 32► C (90► F). Based on the literature research and results from studies presented in Chapters 1 and 2, concrete temperature should be delivered with a much smaller range. Delivery concrete temperature should be greater or equal to 18► C (64► F) but never to exceed 25► C (77► F).

For hot weather concreting, NDOT subsection 501.03.10(d) currently allows for concrete temperature at delivery to reach as much as 32► C (90► F). Concrete temperature in hot weather should not exceed 20► C (68► F). Exceptions may allow delivery concrete temperature to reach as much as 25► C (77► F) but not more. Results from the heat of hydration tests performed in this study show typical change in temperature of as much as 12► C (21.5► F) in conditions that were neither hot nor cold but in between. This measure would easily diminish thermal gradients.

D. NDOT should consider placing all concrete for bridge deck at the beginning of the evening and should provide measures to maintain evaporation rate as low as possible, regardless of the time of the year and location throughout Nevada.

Results from Chapter 6 clearly indicated that placement of concrete should start early in the evening. In this study, the starting time corresponded to 6:00 pm. Benefits of doing so include avoidance of congested traffic, smoother delivery schedule, higher quality concrete from the batch plant, reduced evaporation, less heat and therefore reduced thermal gradient.

Whether in hot or cold weather concreting, NDOT should consider using anti-evaporative agents rather than curing compounds immediately after the concrete has been placed. Anti evaporative agents are more efficient in retaining water in concrete than curing compounds. In addition to the use of an anti-evaporative agent, moist cure should be provided according to existing specifications found in NDOT subsection 501.03.09. Moist cure should last 7 days, but not less than 7 days when SCC/SCA concrete is placed. If cold weather concreting is scheduled, NDOT should consider the erection of shelter to protect the concrete. The benefits of erecting a shelter include protection against sunrays, unexpected bad weather, wind, and improvement of the curing of the concrete.

E. NDOT should adopt the performance specification as shown in Table 3-11. It is recommended that any trial mix designs NDOT would consider for a bridge deck project be initially tested for compressive strength, drying shrinkage, chloride ion penetration, and alkali-silica reactivity.

Though relatively high compressive strength cannot be an indicator of durability, the increase in strength variability at early age and the decrease in strength variability at later ages are attributed in most part to the nature of concrete than to the degree of control exercised during the production and testing of the concrete. Thus when a compressive strength performance criteria is specified for a concrete at a certain age, there should be a better control on the engineering of the concrete for this performance parameter.

As it was shown in this study, a better control of the shrinkage performance of concrete reduces dramatically the probability of cracking and increases the life of the concrete. In order to achieve concrete that would be acceptable for use in bridge decks, it is recommended that a performance based shrinkage behavior be adopted by using Table 3-11. Concrete should be tested for drying shrinkage and adjustments made to the trial mixes accordingly.

Chloride ion penetration can be a devastating phenomenon. It is known that the water content is the most crucial parameter that influences chloride ion penetration within concrete. A brief discussion on the lowering the water content, including fly ash or silica fume in a mix design highlighted the benefits in increasing the concrete resistance to the phenomenon. To ensure that the highest resistance to this problem can be achieved in the field, concrete should be tested for chloride ion penetration and adjustments made to the trial mixes accordingly by following performance guidelines outlined in Table 3-11.

The problem of alkali-silica reactivity (ASR) was described and discussed in detail by Will and Sanders.[111] In addition, it should be noted that the basic assumption is that for ASR to develop water must be present, and if there is little or no free water in the concrete no reaction can occur, as one of the three conditions necessary for the development of such reaction is missing. However, it was recently found[29] that a substantial rise in the early temperature of concrete may cause some aggregates to become reactive to alkali, thus the importance of selecting aggregates with minimum reactivity potential as recommended in the footnotes of Table 3-11.

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TABLES

Table 2-1 Variation of Coefficients of Thermal Expansion ?$_e$ and Thermal Contraction ?$_c$ at Early Age for Different Water Cement Ratios in Tests Reported by Emborg[48] and Hedlund[49]

Cement Type	W/C	?$_e$ [10^{-6}/?C]	?$_c$ [10^{-6}/?C]
ASTM Type I	0.40-0.50	9.5-12.5	7.0-8.5
Standard Portland	0.50-0.70	9.5-10.5	7.0-8.0
	0.70-0.80	9.5-10.0	7.0-8.0
Type II Standard Portland	0.25-0.35	9.0-12.5	6.0-8.3
	0.40-0.80	9.5-11.0	7.0-9.0

Table 2-2 Resulting Tensile Stresses* in the Deck Slab at an Internal Support for the Continuous Method of Casting for ? values of 0.04 and 0.12 adapted from Ducret and Lebet[55]**

	Tensile Stress ?$_c$, MPa (psi)	
	? = 0.04	**? = 0.12**
Without ?T	1.8 (261)	2.7 (392)
With ?T	2.3 (334)	4.2 (609)

*Includes movement of the traveling formwork

**Low value of ? indicates wide bridge. As ? increases, it is an indication of a narrower bridge deck.

Table 2-3 Resulting Tensile Stresses in the Deck Slab* at an Internal Support for the Piano Casting Method for ? values of 0.04 and 0.12 adapted from Ducret and Lebet[55]

	Tensile Stress ? $_c$, MPa (psi)	
	? = 0.04	**? = 0.12**
Without ? T	-0.2 (-29)	-0.5 (-73)
With ? T	0.2 (29)	1.0 (145)

*See Table 2-2

Table 2-4 Guidelines for Prestressing Techniques adapted from Ducret and Lebet[59]

	Prestressing Technique			
?	**Straight Cables prestressed before connection**	**External Prestressing**	**Straight Cables prestressed after connection**	**Lowering of support**
0.04	Not recommended	Expensive	Recommended	Possible
0.12	Recommended	Possible	Expensive	Not recommended

Table 2-5 Specified Probability of Cracking and Limitation Method adapted from Ducret and Lebet[59]

?$_c$[*]	Probability[**]	Limitation Method for 30 m (80 ft.)	Limitation Method for 80 m (240 ft.)
?$_c$ > ?$_{ct}$[***]	High	None	No Method
0 < ?$_c$ < ?$_{ct}$	Limited	No Method	Two methods or prestressing
?$_c$ < 0	Very low	Prestressing	Prestressing before connection

[*]Tensile stress in the concrete deck after construction
[**]Probability of cracking with time
[***]Tensile strength of the concrete deck

Table 2-6 Default Values[*] for Parameters used in Equation 2.1 adapted from Ducret et al[60]

E_{c1}, MPa (ksi)	E_{c2}, MPa (ksi)	?	? T
6000 (871)	25000 (3626)	10^{-5} K^{-1}	25?C

*These values may be used in the absence of other information.

Table 2-7 Influence of ? on the Effects of Concrete Hydration adapted from Ducret et al[60] and Lebet and Ducret[61]

? ? 0.05	Limited influence on early cracking
0.05 ? ? ? 0.08	Reduce tensile strength ?$_{ct}$, limited risk of early cracking
0.08 ? ? ? 0.12	Reduce tensile strength ?$_{ct}$, probable early cracking, reduction of ?$_c$ should be considered

Table 3-1 Steamboat Creek Crossing Concrete Bridge Deck Mix Design

Project Information			
Project Identification	Geiger Grade and Toll Road Bridge		
NDOT Concrete Class	EA Modified, 28 MPa (4000 psi)		
Cement Type, Supplier	Type I/II Low-Alkali, Calaveras		
Fly Ash Supplier, % (addition rate)	ISG Resources off Bridger Plant, 20		
Material Description and Strength Requirements			
Mixture Ingredients, kg/m^3 (lbs/yd^3)		**Admixtures and Other Properties**	
Cement	312 (526)	AEA[???], ml/100 kg (fl. oz/100 lbs)[?]	91 (1.4)
Fly Ash, Type F	78 (132)	WR[??], ml/100 kg (fl. oz/100 lbs)[?]	320 (4.9)
Coarse Aggregates, #57	867 (1460)	Coarse Aggregate Source or Pit	All-Lite
Fine Aggregates	705 (1188)	Fine Aggregate Source or Pit	Paiute
Water	154 (260)	Target Air Content, %	6
Theoretical Weight	2117 (3568)		

[?] Per weight of cementitious materials.
[??] Master Builders MB AE-90.
[???] Master Builders 300N.

Table 3-2 NDOT Deck Inspection Report for Selected Bridges in Northern Nevada

Location	Type	Date & Time of Placement	Curing Method	Observations
Mt. Rose, I-580	Concrete	10/94 6:00 am	Chlorinated Rubber	Tight Cracks[4]
Brown School Bridge	Concrete	Spring 95	Chlorinated Rubber	Tight Cracks
Old Virginia Bridge	Concrete	Spring 95	Chlorinated Rubber	Tight Cracks
Zolezzi	Box Girder	Spring 95	Chlorinated Rubber	Tight Cracks
So. Meadow PKWY	Concrete	10/94	Wax Based[1]	Tight Cracks
So. Virginia	Steel	09/94	Chlorinated Rubber	Tight Cracks
Robb Dr.	Steel	03/93	Wet[2]	Tight Cracks
Pyramid Interchange	Steel	10/98	PAM[3]	Tight Cracks
Patrick	Steel	08/96	Chlorinated Rubber	Tight Cracks
Clear Acre Slip Ramp	Concrete	06/99	PAM	No Cracks
So. Virginia Off Ramp	Concrete	09/94	Wax Based	No Cracks

[1]Type 2 Class A curing compound meeting NDOT 702.03.01(d) specifications.
[2]Wet burlap sack applied for 3-5 days meeting NDOT 702.03.01(a) specifications.
[3]Poly-Aplha-Methyl Styrene resin based curing compound that meets NDOT 702.03.04 specifications.
[4]Equivalent to hairline cracks.

Table 3-3 Sahara Concrete Bridge Deck Mix Design

Project Information			
Project Identification	Sahara Deck, Las Vegas		
NDOT Concrete Class	D Modified, 31 MPa (4500 psi)		
Cement Type, Supplier	Type V, Ash Grove Cement		
Fly Ash Supplier, % (addition rate)	Not Applicable		
Material Description and Strength Requirements			
Mixture Ingredients, kg/m^3 (lbs/yd^3)		**Admixtures and Other Properties**	
Cement	390 (658)	AEA, ml/100 kg (fl. oz/100 lbs)	-
Fly Ash, Type F	-	WR[??], ml/100 kg (fl. oz/100 lbs)	NA[?]
Coarse Aggregates, #67	1089 (1835)	Coarse Aggregate Source or Pit	Buffalo Rd
Fine Aggregates	712 (1200)	Fine Aggregate Source or Pit	Gorno
Water	176 (296)	Target Air Content, %	-
Theoretical Weight	2367 (3989)		

[?] Information not available.
[??] W. R. Grace WRDA-79.

Table 3-4 U.S. 95 Highway Concrete Deck Mix Design

Project Information			
Project Identification	Not Available		
NDOT Concrete Class	Not Available, 28 MPa (4000 psi)		
Cement Type, Supplier	Mitsubishi Type V, California		
Fly Ash Supplier, % (addition rate)	Western Ash, 20		
Material Description and Strength Requirements			
Mixture Ingredients, kg/m^3 (lbs/yd^3)		**Admixtures and Other Properties**	
Cement	307 (517)	AEA, ml/100 kg (fl. oz/100 lbs)	-
Fly Ash, Type F	61 (103)	WR[??], ml/100 kg (fl. oz/100 lbs)	131-327 (2 -5)
Coarse Aggregates, #67	1110 (1870)	Coarse Aggregate Source or Pit	Lone Mt. Road
Fine Aggregates	828 (1395)	Fine Aggregate Source or Pit	NRM
Water	180 (303)	Target Air Content, %	-
Theoretical Weight	2485 (4188)		

?? Master Builders Pozzolith 220N

Table 3-5 Desert Inn Concrete Bridge Deck Mix Design

Project Information			
Project Identification	Not Available		
NDOT Concrete Class	D Modified, 28 MPa (4000 psi)		
Cement Type, Supplier	Type V, Ash Grove Cement		
Fly Ash Supplier, % (addition rate)	Pozzolanic International (Ipsc Plant), 15		
Material Description and Strength Requirements			
Mixture Ingredients, kg/m^3 (lbs/yd^3)		**Admixtures and Other Properties**	
Cement	308 (519)	AEA, ml/100 kg (fl. oz/100 lbs)	-
Fly Ash, Type F	65 (110)	HRWR[? ?], ml/100 kg (fl. oz/100 lbs)	320 (4.9)
Coarse Aggregates, #67	1080 (1820)	Coarse Aggregate Source or Pit	Buffalo Rd.
Fine Aggregates	736 (1240)	Fine Aggregate Source or Pit	Gorno
Water	168 (283)	Target Air Content, %	-
Theoretical Weight	2357 (3972)		

[? ?] W. R. Grace Daracem 55.

Table 3-6 Reinforcement Considerations of Selected U.S. States

	State Department of Transportation									
	UT	**TX**	**WA**	**WY**	**NM**	**MO**	**ID**	**CO**	**CA**	**NV**
Is Rebar Size limited to No. 5[1]?	Yes	No	No	No	Yes	Yes	No	No	No	No
Min. Top Cover (in.)	2	NA[2]	2 ½	2 ½	2	2 ?	2 ½	2 ½	2	2

[1]Rebar Size in English Units
[2]Not Answered

Table 3-7 Girder Types for Support of Concrete Decks

	Percentage of Total Bridges, %									
Girder Type	**UT**	**TX**	**WA**	**WY**	**NM**	**MO**	**ID**	**CO**	**CA**	**NV**
Steel	36	15	20	95	12	35	20	10	15	35
RC Box	-	3	10	-	2	-	-	-	40	25
Precast, Prestressed	45	80	50	5	85	60	75	40	8	-
Post-Tensioned Box	1	2	10	-	1	-	5	10	25	35
Other	14	-	10	-	-	5	-	40	12	5

Table 3-8 Aggregate Type and Size Considerations

	State Department of Transportation									
Type	**UT**	**TX**	**WA**	**WY**	**NM**	**MO**	**ID**	**CO**	**CA**	**NV**
Limestone	-	X	[a]	X	-	-	-	-	-	-
River Gravel	-	-		X	-	X	X	X	-	-
Slate, Dolomite, etc…	-	-		X	-	X	-	X	-	-
Size, mm (in.)										
19 (0.75)	X	-	X	-	X	X	-	X	-	X
25 (1)	-	-	-	-	-	-	-	-	-	-
38 (1 ½)	-	X	-	X	-	-	X	-	X	-

[a]From glacier activity deposits

Table 3-9 Mineral Admixture Considerations

	State Department of Transportation									
Type	**UT**	**TX**	**WA**	**WY**	**NM**	**MO**	**ID**	**CO**	**CA**	**NV**
Fly Ash	X	X	X	X	X	X	X	X	X	X
Silica Fume	X	X	-	X	-	-	X	X	X	-
Slag, Other	X	X	-	X	-	-	X	X	X	-

Table 3-10 Chemical Admixture Considerations

	State Department of Transportation									
	UT	**TX**	**WA**	**WY**	**NM**	**MO**	**ID**	**CO**	**CA**	**NV**
Plasticizers	X	X	X	X	X	X	X	X	-	X
Superplasticizers	-	X	-	-	X	X	-	X	-	X
Air Entrainers	X	X	X	X	X	X	X	X	X	X
Accelerators	-	-	-	-	X	-	-	-	-	-
Retarders	-	X	-	-	X	-	-	-	-	-
Other	-	-	-	-	X	-	-	-	-	-

Table 3-11 Suggested Concrete Performance Grades for NDOT[3]

Performance Parameter	Test Method	Concrete Performance Grade			
		0	1	2	3
Freeze-Thaw Durability (x = relative dynamic modulus of elasticity)	ASTM C 666	NA	60% ? x ? 80%	80% ? x ? 90%	x ? 90%
Scaling Resistance (x = visual rating of surface after 50 cycles)	ASTM C 672	NA	x = 2, 3	x =1	x = 0
Abrasion Resistance (x = avg. depth of wear in mm)	ASTM C 944	NA	2.0 ? x ? 1.0	1.0 ? x ? 0.5	0.5 ? x
Chloride Penetration (x = coulombs)	ASTM C 1202	NA	4000 ? x ? 2000	2000 ? x ? 1000	1000 ? x
Elasticity (x = modulus of elasticity, x10^6), in psi	ASTM C 469	NA	2.5 ? x ? 4.5	4.5 ? x ? 6.5	x ? 6.5
Sulfate Resistance, x (x = expansion at given age, %), age in months	ASTM C 1012	NA	x ? 0.10 6	x ? 0.10 12	x ? 0.10 18
Shrinkage (x = microstrain)	ASTM C 157	NA	600 ? x? 800	400 ? x ? 600	x ? 400
Creep (x = microstrain/pressure unit), pressure unit = psi	ASTM C 512	NA	0.5 ? x ?0.3	0.3 ? x ? 0.2	x ? 0.2

1. Compressive strength should be specified based on the desired value at a given age. Strengths lower than 2500 psi at 28 days shall not be specified.
Desired fresh concrete properties such as slump and unit weight should be specified.
Maximum aggregate size should be specified to ensure that concrete can flow into forms without segregation or void formation.
All aggregates proposed for use shall be tested for alkali silica reactivity according to ASTM C 1260. Aggregates yielding expansion less than 0.10 % at 16 days shall be considered to have acceptable ASR performance. Aggregates yielding expansion greater than 0.10% shall only be used in combination with materials, which have been shown to mitigate the alkali silica reaction according to ASTM C 227. Combination of materials yielding expansions less than 0.05 % at 3 months and 0.10 % at 6 months shall be considered acceptable.

Table 4-1 Nevada Cement Analysis Report

Information Resource			
Supplier	Nevada Cement Laboratory Test Report		
Report Date	July 26, 2000		
Cement Type	Type I/II Low-Alkali		
Shipping Date	June 2000		
Chemical Analyis and Physical Test Reports			
Chemical Analysis (ASTM C 114), %		Specific Surface (ASTM C 204), m^2/kg	366
Silicon Dioxide (SiO_2)	21.05	Autoclave Expansion (ASTM C 151), %	0.00
Aluminum Oxide (Al_2O_3)	4.40	Set Times Vicat Needles (ASTM C 191)	
Ferric Oxide (Fe_2O_3)	2.38	Initial Set Time, min.	100
Magnesium Oxide (MgO)	3.04	Final Set Time, min.	251
Sulfur Trioxide (SO_3)	2.93	Air Content (ASTM C 185), %	6
Sodium Oxide (Na_2O)	0.24	Compounds (ASTM C 150), %	
Potassium Oxide (K_2O)	0.25	Tricalcium Silicate	56
Titanium Oxide (TiO_2)	0.24	Dicalcium Silicate	18
Loss on Ignition	1.50	Tricalcium Aluminate	7.6
Insoluble Residue	0.44	Tetracalcium Aluminoferrite	7
Total equivalent alkali as Na_2O	0.38	Compressive Strength (ASTM C 109)	
Note: Density of the cement was rated at a value of 3.15.		3-Day, MPa (psi)	26.1 (3783)
		7-Day, MPa (psi)	32.2 (4675)

Table 4-2 Fly Ash Chemical Analysis Report

<table>
<tr><th colspan="4">Information Resource</th></tr>
<tr><td>Supplier</td><td colspan="3">Industrial Service Group Resources, Inc</td></tr>
<tr><td>Report Date</td><td colspan="3">April 6, 2000</td></tr>
<tr><td>Fly Ash Type</td><td colspan="3">Type F</td></tr>
<tr><td>Shipping Date</td><td colspan="3">May 2000</td></tr>
<tr><td>Plant</td><td colspan="3">Bridger</td></tr>
<tr><td colspan="4">Chemical Analyis and Physical Test Reports</td></tr>
<tr><td colspan="2">Chemical Analysis (ASTM C 618-97), %</td><td>Density</td><td>2.34</td></tr>
<tr><td>Silicon Dioxide (SiO_2)</td><td>58.93</td><td>Fineness Retained on #325 Sieve, %</td><td>28.62</td></tr>
<tr><td>Aluminum Oxide (Al_2O_3)</td><td>18.01</td><td>Water Requirement, % of Control</td><td>96.6</td></tr>
<tr><td>Iron Oxide (Fe_2O_3)</td><td>5.48</td><td>Soundness (Autoclave Expansion), %.</td><td>0.06</td></tr>
<tr><td>Sulfur Trioxide (SO_3)</td><td>0.45</td><td>Strength Activity Index with Portland Cement, %</td><td></td></tr>
<tr><td>Calcium Oxide (CaO)</td><td>7.93</td><td>Ratio to Control at 7 days</td><td>86.9</td></tr>
<tr><td>Loss on Ignition</td><td>0.20</td><td>Ratio to Control at 28 days</td><td>96.3</td></tr>
</table>

Table 4-3 Expansive Additive Chemical Properties Test Report from CTS Cement Manufacturing Laboratory using ASTM C 806

Information Resource	
Supplier	CTS Cement Manufacturing Company Test Report
Report Date	July 13, 2000
Cement Type, Supplier	Type I/II Low-Alkali, Nevada Cement
Expansive Additive	Chem Comp III
Chem Comp III Addition Rate, %	16.5
Nevada Cement Addition Rate, %	83.5

Chemical Analyis and Physical Test Reports			
X-ray Fluorescence Analysis, %		Restrained Expansion Data	
Silicon Dioxide (SiO_2)	9.57	Specimen Age	Bar Expansion[?], %
Aluminum Oxide (Al_2O_3)	10.21	3	0.038
Ferric Oxide (Fe_2O_3)	0.96	4	0.045
Magnesium Oxide (MgO)	1.78	5.	0.051
Sulfur Trioxide (SO_3)	30.7	6	0.057
Sodium Oxide (Na_2O)	< 1	7[??]	0.062
Potassium Oxide (K_2O)	0.34	Note: The density of the Chem Comp III was rated at a value of 2.95.	
Titanium Oxide (TiO_2)	0.27		

[?] Expansion results are average of two bars.
[??] A desired minimum expansion of 0.04% 7 days in accordance with Table 3 of ASTM C 845 designations. If the minimum 0.04% expansion is reached at 7 days, the expansive cement shows good compatibility with the cement.

Table 4-4 Maximum Limit Requirements for Expansive Hydraulic Cement from ASTM C 845-96

Standard Chemical Requirements	**Limit, %**	**Actual, %**
Magnesium Oxide (MgO)	6.0	1.77
Insoluble Residue, max	1.0	0.36
Loss on Ignition, max	4.0	2.82
Alkalies (Na_2O + 0.658 ? K_2O) max, %	0.60[#]	0.37

[#] The purchaser has the option of specifying that the alkalies shall not exceed 0.60%. This limit should be specified when the cement is to be used in concrete with aggregates that may produce deleterious reactivity.

Table 4-5 California Highway 58 and 15 Bridges Mix Design

Information Resource	
Supplier	CTS Cement Manufacturing Company Test Report
Report Date	September 15, 1995
Cement Type, Supplier	Type I/II Low-Alkali, Riverside
Expansive Additive	Chem Comp III
Chem Comp III Addition Rate, %	12.5
Nevada Cement Addition Rate, %	87.5

Material# Description and Strength Requirements			
Mixture Ingredients, kg/m^3 (lbs/yd^3)		Requirements	
Cement	417 (702)	Age of Specimen, days	Strength, MPa (psi)
Chem Comp III (SCC/SCA)	59 (100)	10	24 (3500)
Coarse Aggregates	954 (1608)	28	35 (5000)
Fine Aggregates	698 (1177)	Air Content, %	1.5
Water	209 (352)	Note that the maximum water-cementitious ratio for the project was specified at 0.53	
Theoretical Weight	2338 (3939)		

#Design Weight in SSD

Table 4-6 Test Results for the Mix Design presented in Table 4-5

Chemical Analysis[1] Results		Age of Specimens	Expansion[2], %	Strength, MPa (psi)
Magnesium Oxide (MgO), %	1.73	7-day	0.046	4845
Insoluble Residue, max (%)	0.37	28-day	0.060	6558[4]
Loss on Ignition, max (%)	2.87	28-day, % of 7-day	130[3]	-
Alkalies (Na_2O + 0.658 ? K_2O) max, %	0.38			

[1]Results are compared to Table 4-4 or Tables 1 thru 3 of ASTM C 845.
[2]Restrained expansion results according to ASTM C 878. Results are average of three specimens.
[3]Table 3 of ASTM C 845 specifies a maximum of 115%.
[4]Strength recorded at 21 days.

Table 4-7 Rocky Ridge Aggregate Test Results as Determined by NDOT

Information Resource				
Result Information	Materials Division, Nevada Department of Transportation			
Aggregate Supplier	Rocky Ridge			
Date of Report	June 30, 2000			
Specification Reference	NDOT Section 706			
Coarse Aggregates Pit	Rocky Ridge			
Fine Aggregates Pit	Stormy Canyon			
Coarse Aggregate			**Fine Aggregate**	
Size, mm (in.)	# 4 % Pass.	# 67 % Pass.	Size, mm (in.)	% Pass.
75 (3)	-	-	9.5 (3/8)	100
63 (2½)	-	-	4.75 (# 4)	100
50 (2)	100	-	2.36 (# 8)	87
37.5 (1½)	99	-	1.18 (# 16)	65
25 (1)	47	100	0.6 (# 30)	42
19 (¾)	5	90	0.3 (# 50)	22
12.5 (½)	1	62	0.15 (# 100)	8
9.5 (3/8)	1	39	0.075 (# 200)	3
4.75 (# 4)	1	3		
Sodium Sulfate Soundness, % Loss		1.0	Sodium Sulfate Soundness, % Loss	8.0
Abrasion, %		18.0	Organic Impurities	Passed
Absorption, %		0.60	Absorption, %	1.88
Bulk Specific Gravity		2.68	Bulk Specific Gravity	2.55
Bulk Specific Gravity (SSD)		2.69	Bulk Specific Gravity (SSD)	2.60
Cleanness Value (#4 and #67), %		89	Fineness Modulus	2.76
Clay Lumps (#4 and #67), %		0.1, 0.0	Clay Lumps, %	0.3

Table 4-8 Properties[*] and Specification Requirements of the Air Entraining Agents (AEA)

Identification	Specific Gravity	Solids Content, %	PH	ASTM C 260
Daravair 1000	1.00-1.04	4.50-6.00	10.0-12.0	Comply
Darex II	1.005-1.015	9.50-11.50	10.5-12.5	Comply

[*]Information provided by W. R. Grace Products.

Table 4-9 Properties[*] of the Superplasticizers (HRWR)

Identification	Specific Gravity	Solids Content, %	PH	ASTM C 494
Daracem 19	1.10-1.20	38.0-41.0	7.0-11.5	A & F
Adva 100	1.02-1.12	30.00-34.00	7.0-9.0	F

[*]Information provided by W. R. Grace Products.

Table 4-10 Typical Properties[#] of the Curing Compound

Information Source	**Material Safety Data Sheet and Technical Data Sheet.**
Composition	Poly-Alpha-Methyl Styrene, plasticizers, and pigment
Color	White
Reflectance, %	65 (typical)
Flash Point, ► C (► F)	57.2 (135)
Weight, kg/l (lb/gal.)	1.03 (8.6)
Moisture Retention, g/cm^2	0.03 (tested by ASTM C 309)

[#]The manufacturer W. R. Meadows of N. CA provided the information.

Table 4-11 Mix Designs Tested by UNR and W.R. Grace

Mix Designs Tested by the University of Nevada-Reno				Mix Designs Tested By W.R. Grace[2]
Mix Designs	**Phase 1**	**Phase 2[1]**	**Phase 3**	
C1	X	X		
C1-FA-SC	X			
C1-SRA	X	X		X
C1-SC-SRA	X			
C1-FA-SRA	X	X		X
C1-FA-SC-SRA	X			
C2	X	X		
C2-SC$_A$	X			
C2-SC$_B$[3]	X			
C3	X	X		
C3-FA-SC	X			
C3-SRA	X	X		X
C3-SC-SRA	X			
C3-FA-SRA	X	X		X
C3-FA-SC-SRA	X			
NDOT-1			X	
NDOT-2			X	
NDOT-3			X	
Sahara			X	
Steamboat			X	
TOTAL	**14**	**8**	**5**	**4**

[1]Phase 2 consisted in reproducing mix designs that exhibited bets overall shrinkage behavior.
[2]Mix Designs reproduced in Phase 2 that were specifically tested for Chloride ion penetration and Hardened Air-Void System by W.R. Grace.
[3]Except for all other mix designs that were both dry and moist cured in Phase 1, C2-SC$_B$ was subjected to Lab Cure only. This mix design was also tested in Phase 2 for Chloride ion penetration.

Table 4-12 Testing Program and Applied Curing Methods

Tests Performed	By the University of Nevada-Reno			
	Phase 1	Phase 2	Phase 3	
Fresh Properties				
Slump	X	X	X	
Air Content	X	X	X	
Fresh Temperature	X	X	X	
Hardened Properties				
Compressive Strength	X	X	X	
Drying Shrinkage	X	X	X	
Modulus of Rupture	X	X		
Heat of Hydration	X			
Cracking Tendency	X			
Chloride Ion Penetration		X[1]	X	
Hardened Air System		X[2]		
Curing Method Applied				
Dry	X			
Moist	X			
Lab		X	X	

[1] Mix Design identified as C2-SC_B tested by the University of Nevada Reno, while W.R. Grace Laboratory tested the mix designs containing the SRA.
[2] W.R. Grace Laboratory performed this test.

Table 4-13 UNR Control Mix Designs Identification and Properties

Materials and Air Property	Mix Identification		
	C1	C2	C3
Cement Content, kg/m^3 (sacks/yd^3)	339 (6.0)	390 (7.0)	418 (7.5)
Water Content*, kg/m^3 (lbs/yd^3)	142 (239)	164 (276)	176 (296)
Coarse Aggregate*, kg/m^3 (lbs/yd^3)	1095 (1845)	1095 (1845)	1095 (1845)
Fine Aggregate*, kg/m^3 (lbs/yd^3)	877 (1478)	780 (1314)	726 (1223)
Target Air Content, %	5.5	5.5	5.5

? All proportions in saturated surface dry (SSD) conditions.

Table 4-14 UNR Mix Designs Incorporating the Fly Ash, SCC/SCA, and SRA

Mix Identification	Water Content* kg/m^3 (lbs/yd^3)	Coarse Aggregate Content? ? kg/m^3 (lbs/yd^3)	Fine Aggregate Content? ? kg/m^3 (lbs/yd^3)
C1-FA-SC	142 (239)	1095 (1845)	850 (1432)
C1-SRA	139 (234)	1095 (1845)	877 (1478)
C1-SC-SRA	139 (234)	1095 (1845)	874 (1472)
C1-FA-SRA	139 (234))	1095 (1845)	853 (1437)
C1-FA-SC-SRA	139 (234)	1095 (1845)	850 (1432)
C2-SC$_A$#	164 (276)	1095 (1845)	776 (1307)
C2-SC$_B$##	164 (276)	1095 (1845)	776 (1307)
C3-FA-SC	176 (296)	1095 (1845)	692 (1166)
C3-SRA	171 (288)	1095 (1845)	726 (1223)
C3-SC-SRA	171 (288)	1095 (1845)	722 (1216)
C3-FA-SRA	171 (288)	1095 (1845)	696 (1173)
C3-FA-SC-SRA	171 (288)	1095 (1845)	692 (1166)

*Adjusted water content in saturated surface dry (SSD) condition, and SRA water replacement.
? ? Proportions in saturated surface dry condition (SSD).
#Specimens cured under both dry and moist curing methods
##Specimens were subjected to lab cure

Table 4-15 Fresh Concrete Properties of UNR Trial Mix Designs: Phase 1

Mix Identification	Slump, mm (in.) {Target 150 ? 50 mm}	Air Content, (%) {Target 5.5 ? 1.5%}	Temperature, ➤ C (➤ F)
C1	158.8 (6.25)	5.5	26.1 (79)
C1-FA-SC	177.8 (7)	5.75	23.3 (74)
C1-SRA	133.4 (5.25)	4.25	19.4 (69)
C1-SC-SRA	114.3 (4.5)	4.25	25.5 (78)
C1-FA-SRA	198.1 (7.8)	4.25	23.3 (74)
C1-FA-SC-SRA	158.8 (6.25)	4.5	25 (77)
C2	203 (8)	6	22.2 (72)
C2-SC$_A$	101.6 (4)	4.5	22.7 (73)
C2-SC$_B$[#]	101.6 (4)	5	14.4 (58)
C3	108 (4.25)	4.75	27.7 (82)
C3-FA-SC	158.8 (6.25)	6.75	15 (59)
C3-SRA	133.4 (5.25)	5	22.2 (72)
C3-SC-SRA	101.6 (4)	4.5	25 (77)
C3-FA-SRA	141.5 (5.75)	4.5	22.2 (72)
C3-FA-SC-SRA	184.1 (7.25)	4.5	23.3 (74)

[#]This mix design was included here for comparison purposes. More detailed results for this mix design are available in Chapter 5.

Table 4-16 Final Addition Rates for High Range Water Reducers (HRWR), and Air-Entraining Agents (AEA) for the UNR Trial Mix Designs: Phase 1

Mix Identification	HRWR (Superplasticizer)		AEA (Air Entraining Agent)	
	Name	Dosage?, ml/100 kg (fl oz/100 lbs)??	Name	Dosage?, ml/100 kg (fl oz/100 lbs)??
C1	Daracem 19	1248 (19.09)	Daravair 1000	121 (1.85)
C1-FA-SC	Daracem 19	1654 (25.30)	Daravair 1000	87 (1.33)
C1-SRA	Adva 100	926 (14.17)	Darex II	89 (1.36)
C1-SC-SRA	Adva 100	653 (9.99)	Darex II	102 (1.56)
C1-FA-SRA	Adva 100	882 (13.49)	Darex II	101 (1.54)
C1-FA-SC-SRA	Adva 100	1125 (17.21)	Darex II	217 (3.32)
C2	Daracem 19	773 (11.82)	Daravair 1000	95 (1.45)
C2-SC$_A$	Daracem 19	567 (8.67)	Daravair 1000	82 (1.25)
C2-SC$_B$	Daracem 19	649 (9.93)	Daravair 1000	87 (1.33)
C3	Daracem 19	487 (7.45)	Daravair 1000	152 (2.33)
C3-FA-SC	Daracem 19	643 (9.84)	Daravair 1000	233 (3.56)
C3-SRA	Adva 100	582 (8.91)	Darex II	183 (2.80)
C3-SC-SRA	Adva 100	603 (9.23)	Darex II	175 (2.67)
C3-FA-SRA	Adva 100	436 (6.67)	Darex II	183 (2.80)
C3-FA-SC-SRA	Adva 100	570 (8.72)	Darex II	216 (3.31)

? Indicated dosage of HRWR and AEA per weight of cementitious materials.
?? 1.0 ml/100 kg ? 0.0153 fl oz/100 lbs (of cementitious materials).

Table 5-1 Summary of the Compressive Strength Reduction Indices Induced by the Incorporation of SRA in Concrete Mix Designs Subjected to both Dry and Moist Cures: Phase 1

Mix Identification	Compressive Strength, MPa (psi)			
	Specimens under Dry Cure (Method 1)			
	3-day	7-day	28-day	56-day
C1	27 (3943)	35 (5125)	43 (6175)	44 (6327)
C1-SRA	24 (3484)	32 (4667)	39 (5722)	42 (6273)
C1-FA-SRA	23 (3275)	27 (3912)	34 (4878)	37 (5373)
Indices of Reduction*, %	***11.6, 16.9***	***8.9, 23.6***	***7.3, 21.0***	***0.9, 15.1***
C3	33 (4755)	40 (5765)	45 (6521)	48 (6949)
C3-SRA	33 (4688)	38 (5550)	43 (6257)	47 (6854)
C3-FA-SRA	25 (3628)	30 (4342)	36 (5229)	40 (5829)
Indices of Reduction*, %	***1.4, 23.7***	***3.7, 24.7***	***4.1, 19.8***	***1.4, 16.1***
	Specimens under Moist Cure (Method 2)			
C1	29 (4181)	37 (5331)	43 (6269)	45 (6545)
C1-SRA	27 (3939)	33 (4773)	41 (5904)	44 (6400)
C1-FA-SRA	25 (3625)	30 (4421)	39 (5722)	43 (6305)
Indices of Reduction*, %	***5.8, 13.3***	***10.5, 17.1***	***5.8, 8.7***	***2.2, 3.7***
C3	36 (5255)	44 (6400)	48 (7000)	51 (7403)
C3-SRA	33 (4803)	38 (5580)	45 (6577)	47 (6887)
C3-FA-SRA	27 (3887)	33 (4841)	42 (6074)	44 (6450)
Index of Reduction*, %	***8.6, 26***	***12.8, 24.4***	***6.1, 13.2***	***6.9, 12.8***

*The first index corresponds to the reduction incurred in mix designs with the SRA only, while the second index indicated the reduction incurred in mix designs with the SRA and fly ash.

Table 5-2 Effects of SCC/SCA and SRA on Drying Shrinkage: Phase 1

Mix Identification	Drying Shrinkage Reduction* @ 28 Days, %		Drying Shrinkage Reduction* @ 56 Days, %	
	Dry Cure	Moist Cure	Dry Cure	Moist Cure
C1-FA-SC	-20	-8	-29	-10
C1-SRA	37	38	22	25
C1-SC-SRA	8	50	3	40
C1-FA-SRA	24	20	15	13
C1-FA-SC-SRA	30	84	22	66
C2-SC_A	NA**	39	NA**	-13
C2-SC_B	NA**	106***	NA**	83***
C3-FA-SC	3	17	-4	9
C3-SRA	22	36	12	27
C3-SC-SRA	41	82	23	57
C3-FA-SRA	28	35	17	24
C3-FA-SC-SRA	47	134	34	103

*The drying shrinkage reduction is calculated as the reduction or increase in drying shrinkage of the trial batches relative to the control mix designs C1, C2, and C3. A negative reduction value indicates a drying shrinkage higher than that of the control mix design, while a positive reduction value shows an effective lowering of the drying shrinkage relative to the control mix design.
**Not available.
***Results are not related to the good curing method, but to the controlled curing method.

Table 5-3 Fresh Concrete Properties and Final Dosage of the Air Entraining Agent (AEA) and the Superplasticizer (HRWR): Phase 1

Mix Identification	Slump, mm (in.) {Target 150 ? 50 mm}	Air Content, (%) {Target 5.5 ? 1.5%}	Temperature, ► C (► F)
C1-SRA	152.4 (6)	4.5	13.9 (57)
C1-FA-SRA	127 (5)	5	14.4 (58)
C3-SRA	139.7 (5.5)	4.25	17.2 (63)
C3-FA-SRA	139.7 (5.5)	4.75	18.3 (65)

Mix Identification	HRWR		AEA	
	Name	ml/100 kg (fl oz/100 lbs)	Name	ml/100 kg (fl oz/100 lbs)
C1-SRA	Adva 100	1032 (15.79)	Darex II	93 (1.42)
C1-FA-SRA	Adva 100	664 (10.75)	Darex II	119 (1.62)
C3-SRA	Adva 100	556 (8.53)	Darex II	176 (2.69)
C3-FA-SRA	Adva 100	398 (6.09)	Darex II	179 (2.73)

Table 5-4 Summary of the Compressive Strength Reduction Indices Induced by the Incorporation of SRA in Selected Concrete Mix Designs: Phase 2

Mix Identification	Compressive Strength, MPa (psi)			
	3-day	7-day	28-day	56-day
C1	NA	36 (5296)	41 (6018)	44 (6387)
C1-SRA	NA	33 (4773)	40 (5833)	43 (6220)
C1-FA-SRA	NA	30 (4371)	38 (5570)	41 (5971)
Indices of Reduction*, %	NA	***9.8, 17.4***	***6.9, 11.1***	***2.6, 6.5***
C3	NA	41 (5976)	48 (6758)	51 (7296)
C3-SRA	NA	38 (5499)	45 (6518)	47 (6786)
C3-FA-SRA	NA	33 (4789)	42 (6066)	44 (6470)
Indices of Reduction*, %	NA	***7.9, 19.8***	***3.6, 10.2***	***6.9,11.3***

*The first index corresponds to the reduction incurred in mix designs with the SRA only, while the second index indicated the reduction incurred in mix designs with the SRA and fly ash.

Table 5-5 Reduction in Drying Shrinkage: Phase 2

Test Day	C1-SRA	C1-FA-SRA	C3-SRA	C3-FA-SRA
@ **28 Days, %**	28.9	33.7	25.9	28.6
@ **56 Days, %**	23.9	25.6	23.4	22.7

Table 5-6 Resistance to Chloride Ion Penetration Rating: Phase 2

Test Day	**C1-SRA**	**C1-FA-SRA**	**C3-SRA**	**C3-FA-SRA**	***C2-SC$_B$***
	Coulombs Passed and ASTM C 1202 Rating				
@ **56 Days**	4262	6040	5296	7757	*5957*
Rating	**High**	**High**	**High**	**High**	***High***
@ **90 Days**	3212	2085	3960	1637	*3623*
Rating	**Moderate**	**Moderate**	**Moderat e**	**Low**	***Moderate***

Refer to Table 3-11 to assign rating using the Performance Grading System.
Italics in last column indicate mix design tested by the University of Nevada Reno.
First four columns are mix designs that were tested for chloride ion penetration by W.R. Grace.

Table 5-7 Hardened Air-Void System and Requirements: Phase 2

	C1-SRA	C1-FA-SRA	C3-SRA	C3-FA-SRA	Requirement[a]
Air Content A_C, %	3.6 (4.5)[c]	3.7 (5.0)[c]	2.7 (4.25)[c]	3.3 (4.75)[c]	5.5 ► 1.5%[b]
Air Content Reduction, %	0.9	1.3	1.55	1.45	NA
Air Loss, %	20	26	36	30	NA
Spacing Factor, mm (in.)	0.279 (0.0110)	0.180 (0.0071)	0.228 (0.0090)	0.185 (0.0073)	? 0.2 mm (0.008 in.)
Specific Surface S, mm^{-1} ($in.^{-1}$)	19.48 (495)	29.68 (754)	29.56 (751)	33.89 (861)	23.6 ? S ? 43.3 600 ? S ? 1100
Voids V, mm^{-1} ($in.^{-1}$)	4.4	6.9	5.0	7.0	? 1.5?A_C to ? 2?A_C

[a] See ASTM C 457 and ACI 201.2R-92.
[b] Requirements set for this research.
[c] Actual Air Content measured at the fresh stage, according to ASTM C 173.

Table 5-8 Nevada Mix Designs: Phase 3

Materials	Mix Identification and Properties		
	NDOT-1	NDOT-2	NDOT-3
Cement? Content, kg/m^3 ($sacks/yd^3$)	308 (6.5)	332 (7.0)	356 (7.5)
Fly Ash?? Content, kg/m^3 ($sacks/yd^3$)	55 (92)	59 (99)	63 (106)
Water-Cementitious Ratio (W/C)	0.44	0.41	0.38
Boral Linx Content, kg/m^3 ($sacks/yd^3$)	6.4 (10.8)	6.9 (11.7)	7.4 (12.5)
Water Content, kg/m^3 (lbs/yd^3)	158.4 (267)	158.4 (267)	157.8 (266)
Coarse Aggregate???, kg/m^3 (lbs/yd^3)	935 (1575)	935 (1575)	935 (1575)
Fine Aggregate???, kg/m^3 (lbs/yd^3)	765 (1289)	739 (1246)	716 (1206)
Estimated Air Content, %	5.7	5.5	5.5

Note: Admixtures used were Master Builders MB AE-90, and MB 300N.
? Type I/II Low Alkali from Calaveras Cement.
?? Fly Ash Type F, ISG Resources Bridger Plant was added at a replacement rate of 15%.
??? No. 67 coarse aggregate. Both fine and coarse aggregates from Bing Dayton Pit.

Table 5-9 Drying Shrinkage and Chloride Ion Penetration Performance Rating: Phase 3

Test Day	Mix Identification and Rating*				
	NDOT-1	**NDOT-2**	**NDOT-3**	**Steamboat**	**Sahara**
	Drying Shrinkage (ASTM C 157)				
@ 28 days	1	1	1	0	2
@ 56 days	0	0	0	0	2
	Chloride Ion Penetration (ASTM C 1202)				
@ 56 days	High (0)	High (0)	High (0)	High (0)	High (0)
@ 90 days	Moderate (1)	Moderate (1)	Low (2)	Moderate (1)	Moderate (1)

*Drying shrinkage rated using Table 3-12, while chloride ion penetration was rated using ASTM C 1202 and the other rating in ( ) is from Table 3-12. When referring to Table 3-12, please note that the lower the rating number, the lower the performance.

Table 6-1 Wind Speed Guide by Importance Number by Uno[143]

No.	Description	Wind Speed				Comments
		At 30 ft	Equivalent at 20 in.	At 10 m.	Equivalent at 500 mm	
0	Calm	? 1	? 1	? 1	? 1	Smoke rises vertically
1	Light Air	1-3	1-2	1-5	1-3	Smoke drift shows wind direction but not wind vanes
2	Light Breeze	4-7	3-4	6-11	4-7	Leaves rustle
3	Gentle Breeze	8-12	5-8	12-19	8-12	Wind extends light flag
4	Moderate Breeze	13-18	9-12	20-28	13-18	Small branches move
5	Fresh Breeze	19-24	13-16	29-38	19-25	Crested wavelets on inland waters
6	Strong Breeze	25-31	17-20	39-49	26-32	Umbrellas hard to use

Table 6-2 Evaporation Rates Comparison

Variable	Parameters[a]				Evaporation Rates[b]					
	C ➤ C	T ➤ C	RH %	W Kph[c]	ACI		FHWA		Uno	
Wind Speed and cold Temperature	21	21	70	0	0.071	0.015	0.059	0.012	0.057	0.012
				8	0.192	0.038	0.171	0.035	0.171	0.035
				16	0.302	0.062	0.283	0.058	0.285	0.058
				24	0.423	0.085	0.395	0.081	0.399	0.082
				32	0.544	0.112	0.507	0.104	0.513	0.105
				40	0.663	0.135	0.619	0.127	0.627	0.128
Relative Humidity	21	21	90	16	0.104	0.023	0.099	0.020	0.095	0.019
			70		0.302	0.062	0.283	0.058	0.285	0.058
			50		0.491	0.102	0.467	0.096	0.475	0.097
			30		0.662	0.135	0.651	0.133	0.665	0.136
			10		0.862	0.175	0.836	0.171	0.855	0.175
Changing Air and Concrete Temperature	10	10	20	24	0.521	0.107	0.506	0.104	0.465	0.095
	16	16			0.771	0.158	0.739	0.151	0.755	0.155
	21	21			1.055	0.216	1.038	0.213	1.064	0.218
	27	27			1.514	0.310	1.520	0.311	1.521	0.312
	32	32			2.020	0.414	2.023	0.414	1.980	0.406
	38	38			2.816	0.577	2.745	0.562	2.628	0.538
Air Temperature	21	38	20	24	0.615	0.126	0.620	0.127	0.673	0.138
		27			0.941	0.193	0.922	0.189	0.949	0.194
		10			1.189	0.244	1.171	0.240	1.214	0.249
		0			1.255	0.257	1.217	0.249	1.291	0.264
Concrete Temperature	32	0	50	32	3.034	0.621	2.983	0.611	3.058	0.626
	21				1.486	0.304	1.414	0.290	1.586	0.325
	16				1.030	0.211	0.937	0.192	1.090	0.223
Wind Speed and hot Temperature	32	32	10	0	0.340	0.070	0.339	0.069	0.318	0.065
				24	2.273	0.465	2.270	0.465	2.227	0.456
				40	3.562	0.729	3.558	0.729	3.500	0.717

[a]Parameters are as follows: C for Concrete temperature; T for air temperature; RH for relative humidity; and W for wind speed.
[b]ACI values from Equations 6.2 and 6.3; FHWA values using Equation 6.1; Uno values using Equations 6.4 and 6.5.
[c]1 kph = 0.62137 mph.
FHWA Evaporation Rate should not exceed 0.5 kg/m^2/hr (0.1 lb/ft^2/hr).
ACI Evaporation Rate should not exceed 1.0 kg/m^2/hr (0.2 lb/ft^2/hr).

FIGURES

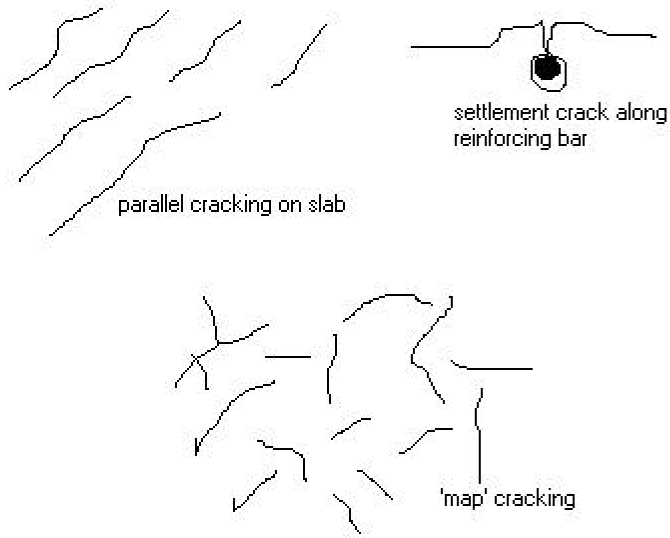


Figure 2-1 Typical Forms of Plastic Shrinkage Cracking

(a)

(b)

Figure 3-1. (a) Typical application of curing compound on the fresh concrete bridge deck; (b) fresh concrete quality control and sampling.

(a)

(b)

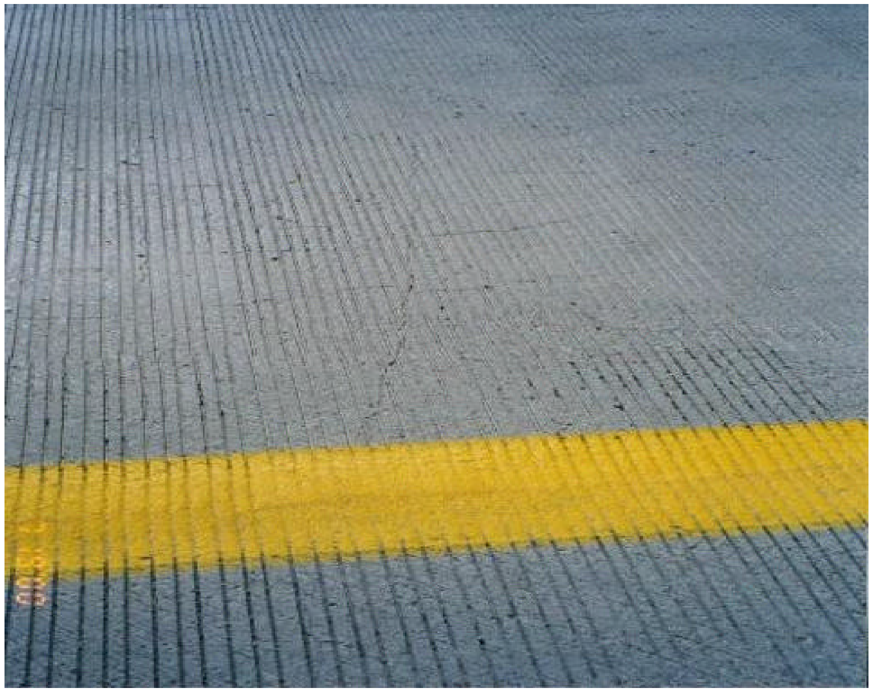
Figure 3-2. **(a) Mechanical screeding and concrete placement with a pump; (b) completed Steamboat Creek bridge on August 15, 2000.**

(a)

(b)

Figure 3-3. **(a) and (b) Longitudinal and diagonal cracks on the Steamboat Creek bridge deck.**

(a)



(b)

Figure 3-4. (a) Transverse and longitudinal shrinkage cracking following the pier caps of the Mt. Rose and I-580 Bridge; (b) transverse cracks over the pier caps.

Figure 3-5. Longitudinal cracks under wheel paths on the North bound lanes of the I-1951 Zolezzi structures. Mild shrinkage cracks were observed over the entire deck.

Figure 3-6. Shrinkage cracks of the I-1305R Clear Acre slip ramp. Entire deck was in fair to good condition. Deck and approach slab are monolithic, and approach slab had more

cracks.

(a)

(b)

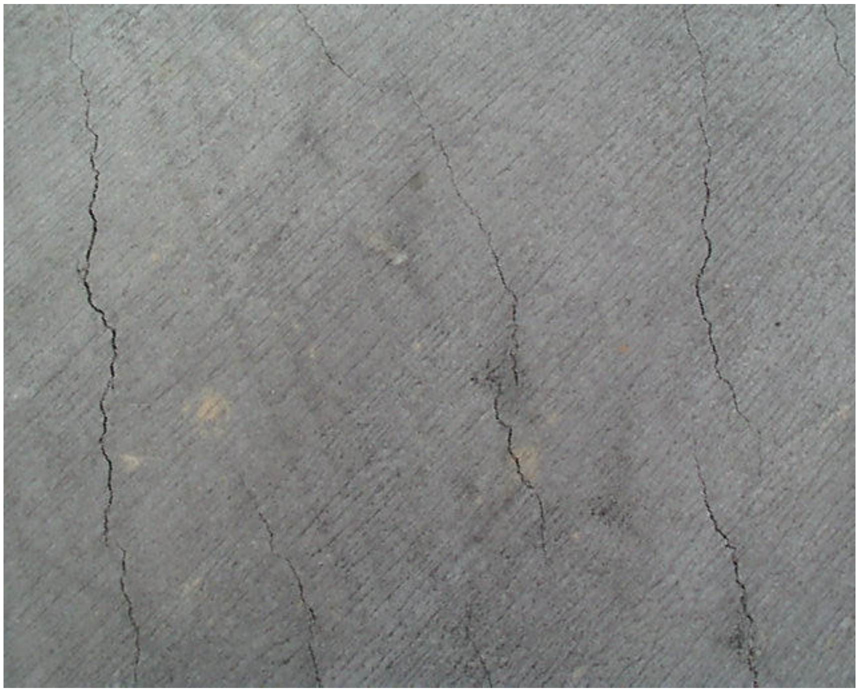
Figure 3-7. (a) Sahara deck crack photographed eight hours after deck pour; (b) curing compound being applied eight hours after deck pour.

(a)

(b)

Figure 3-8. (a) and (b) show combination of longitudinal and transverse cracking on the U.S. 95 Highway bridge deck project.

(a)



(b)

Figure 3-9. (a) Extensive cracks in the mid-section of the deck; (b) finished deck two months after deck pour. A difference in surface color indicates sections separation during the three-section deck pour process from July 5 to July 6, 2000.

(a)

(b)

Figure 3-10. (a) Overall view of the Desert Inn Bridge; (b) the only visible surface crack on the whole deck.

(a)

(b)

Figure 4-1. (a) Basement of the structural lab where a table supported the specimens during cast and poor cure; (b) Specimens in poor curing condition.

(a)

(b)

Figure 4-2. (a) and (b) Overview of the moist box used for the good curing method, in which specimens were moist cured for three days.

(a)

(b)


10A-1
10A-2
10B-1
10B-2
11A-2
11A-1
11B-2
11B-1
12A-1
12A-2
12B-1
12B-2
7A 1
7A 2
7B 1
7B 2
8A-1
8A-2
8B-2
8B-1
9A-1
9A-2
9B-1
9B-2

Figure 4-3. (a) Drying shrinkage specimen molds; (b) Custom built storage shelf for drying shrinkage specimens.

(a)

(b)

Figure 4-4. (a) Length comparator reading of a drying shrinkage specimen; (b) 24 cylinders cast for each trial mix design.

(a)

(b)

Figure 4-5. (a) Cylinder test for compressive strength in the concrete lab; (b) Box where all specimens were placed 24 hours after cast to be sprayed with curing compound.

(a)

(b)

Figure 4-6. (a) Steel ring with the strain gages and sonotube mold ready for the concrete ring cast; (b) wooden platform supporting the setup for the cracking tendency and hydration tests. Note the thermocouple on the upper right going into the concrete.

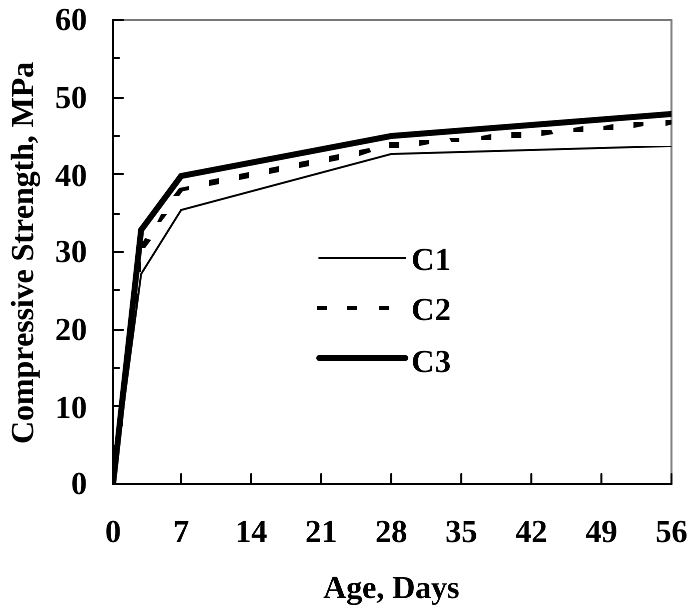


Figure 5-1. Effect of Cement Content on the Compressive Strengths of the Control Mix Designs in Dry Cure: Phase 1

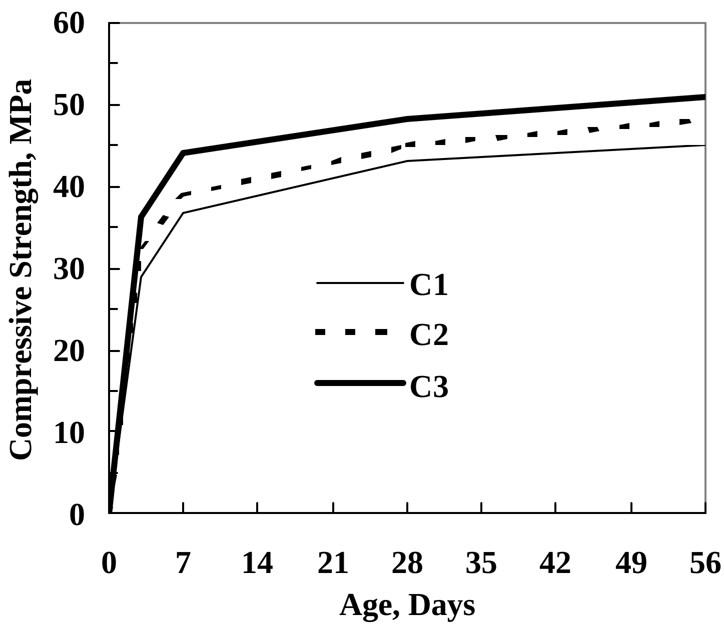


Figure 5-2. Effect of Cement Content on the Compressive Strengths of the Control Mix Designs in Moist Cure: Phase 1

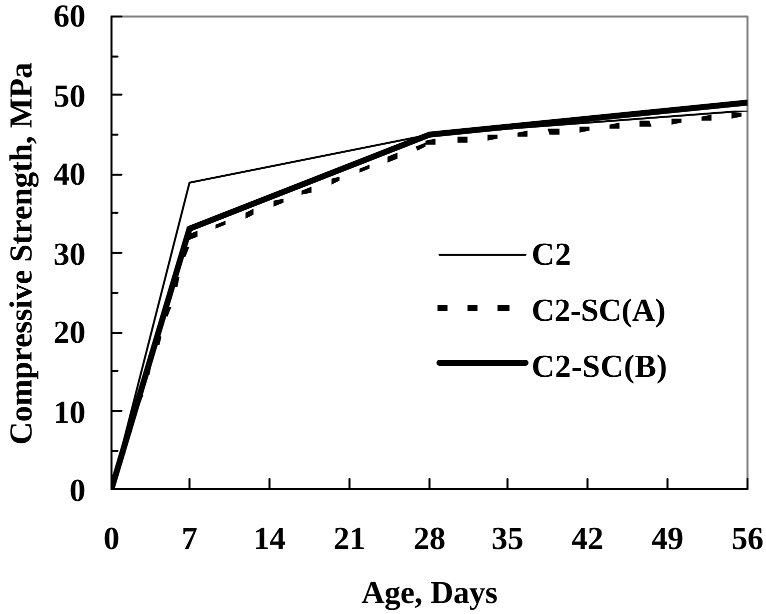


Figure 5-3. Effect of the SCC/SCA on Compressive Strength: Phase 1

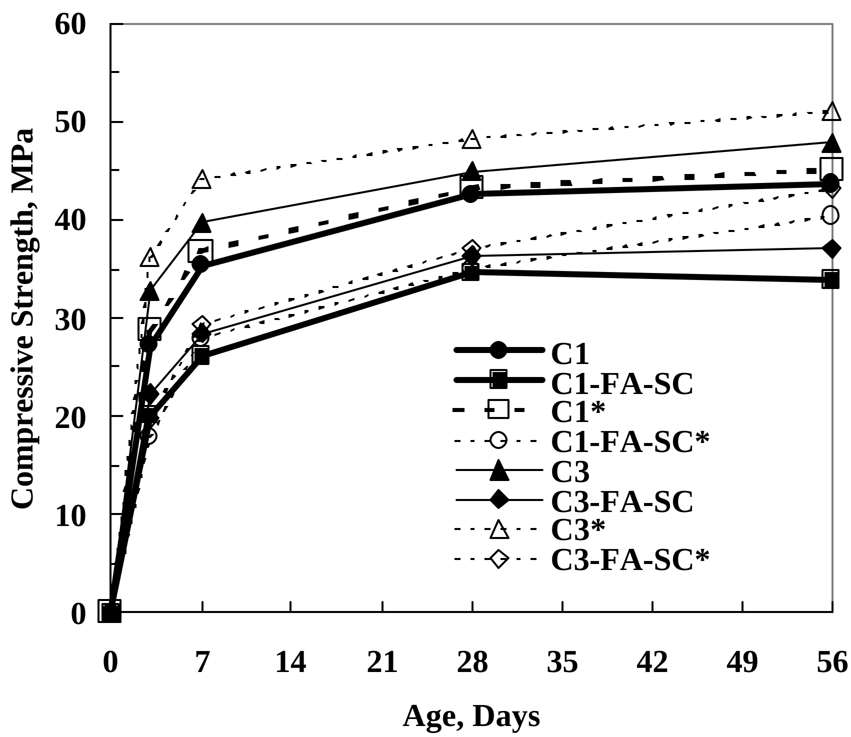


Figure 5-4. Effect of the SCC/SCA with Fly Ash on Compressive Strength: Phase 1 (? Denotes Data of Specimens Subjected to Moist Cure.)

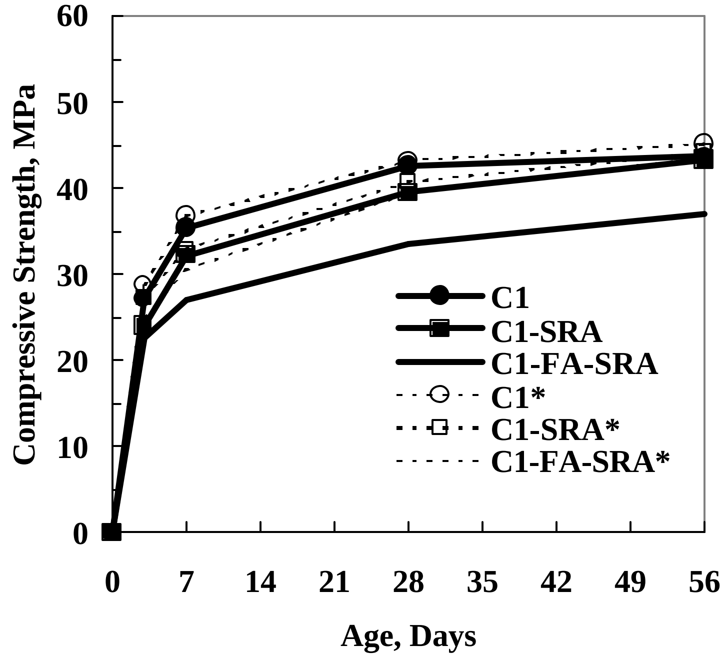


Figure 5-5(a). **Effect of the SRA on Compressive Strength: Phase 1**

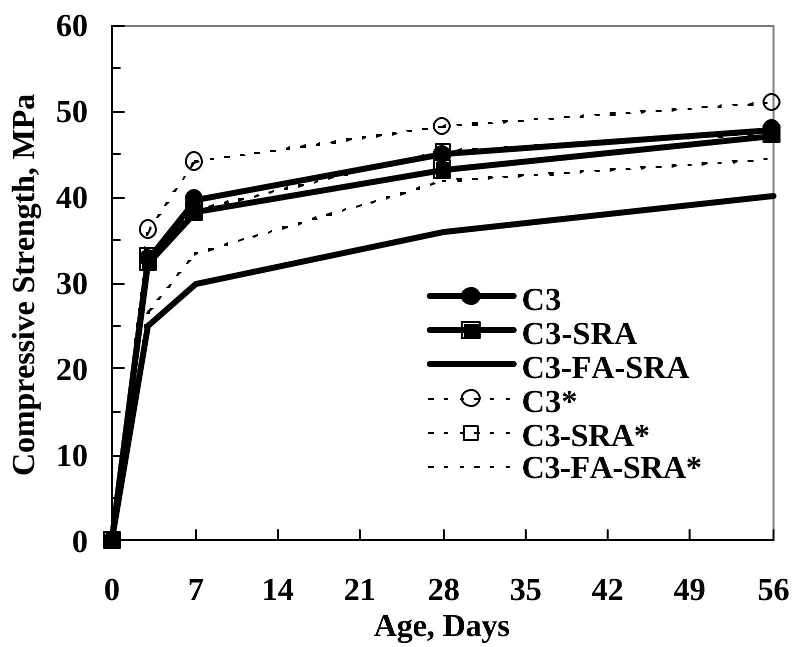


Figure 5-5(b). **Effect of the SRA on Compressive Strength: Phase 1**

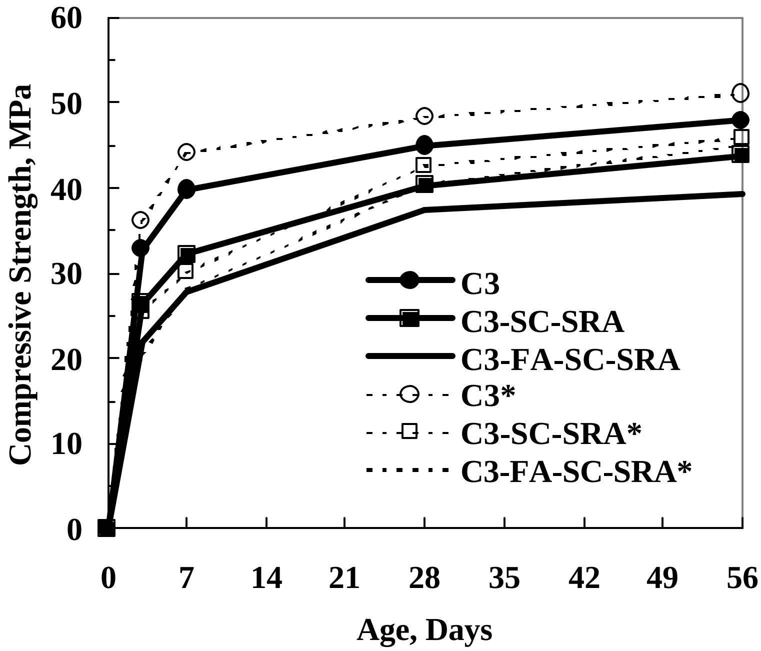


Figure 5-6(a). Effect of Combinations of SCC/SCA, SRA, and Fly Ash on Compressive Strength: Phase 1

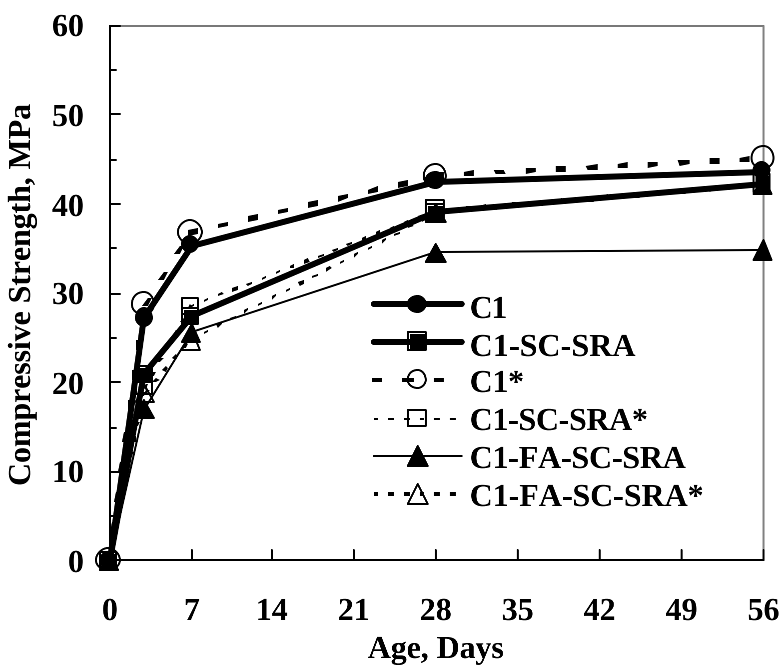


Figure 5-6(b). Effect of Combinations of SCC/SCA, SRA, and Fly Ash on Compressive Strength: Phase 1

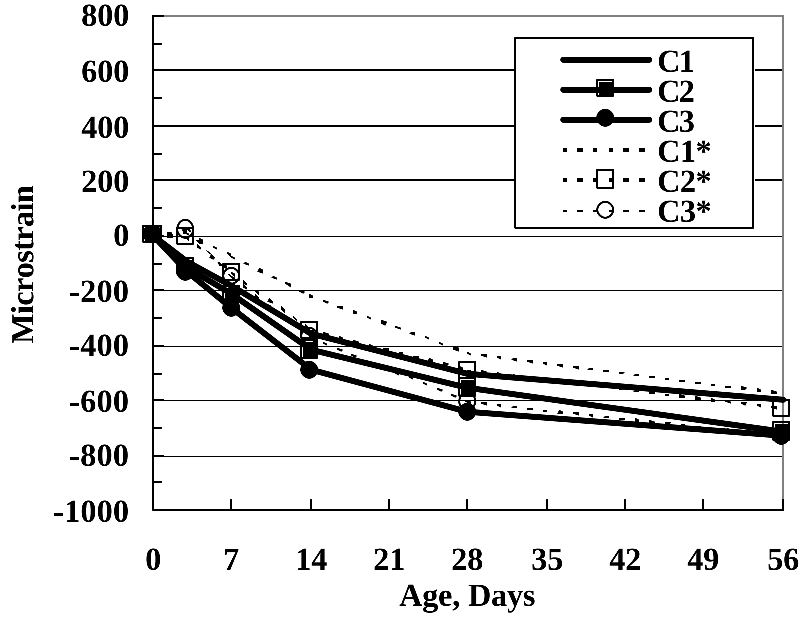


Figure 5-7. Effect of Cement Content on Drying Shrinkage of Concrete: Phase 1

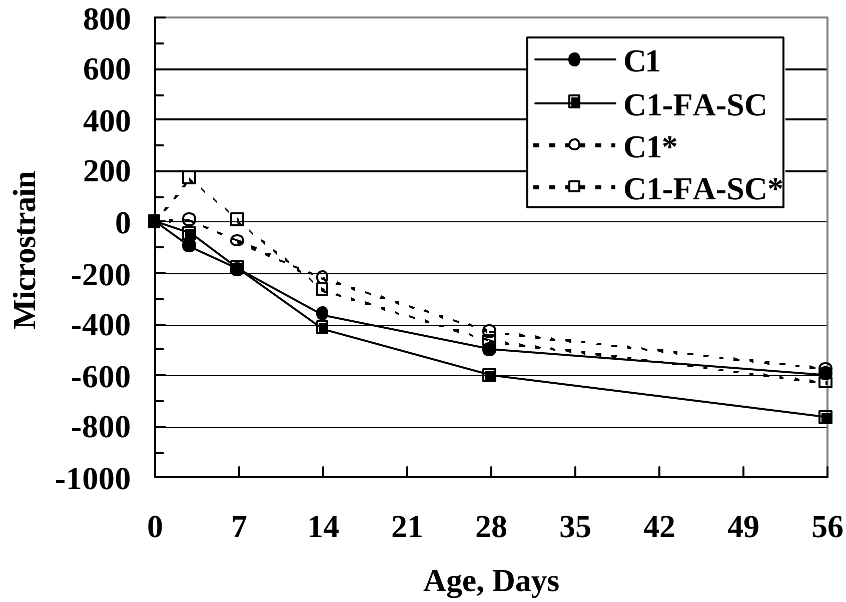


Figure 5-8(a). Effect of the SCC/SCA and Fly Ash on Drying Shrinkage of Concrete: Phase 1

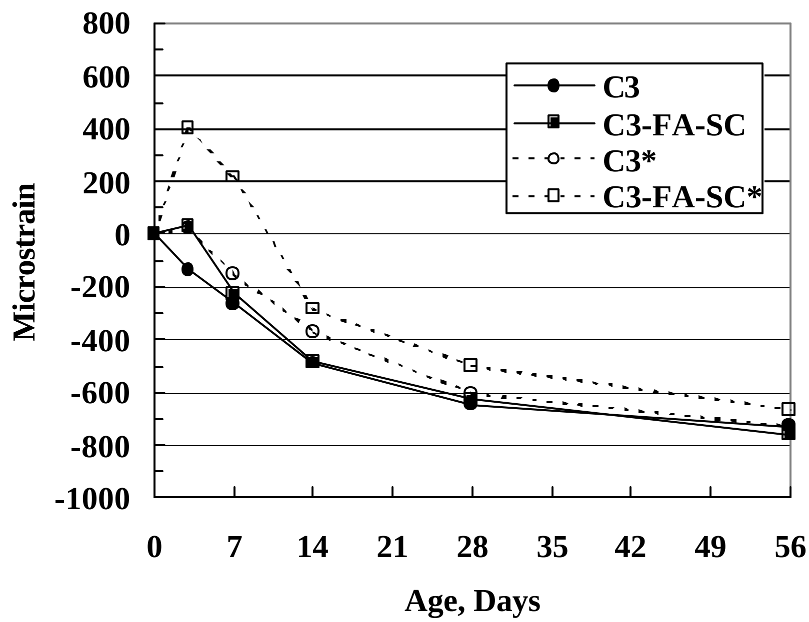


Figure 5-8(b). Effect of SCC/SCA and Fly Ash on Drying Shrinkage of Concrete: Phase 1

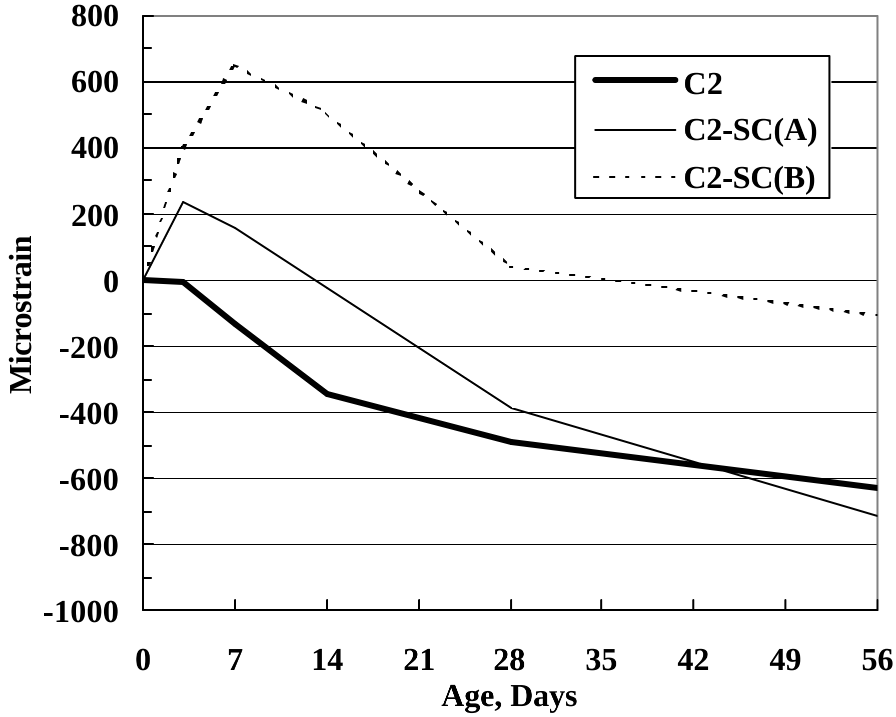


Figure 5-9. Effect of SCC/SCA on Drying Shrinkage of Concrete. Data Shown for the C2 Cement Content and Applied Curing Regimes: Phase 1

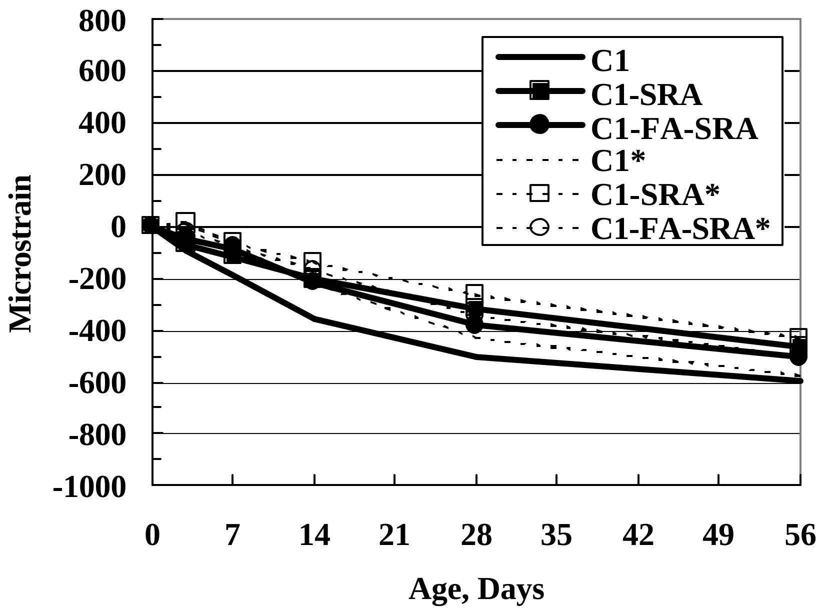


Figure 5-10(a). **Effect of the SRA on Drying Shrinkage of Concrete: Phase 1**

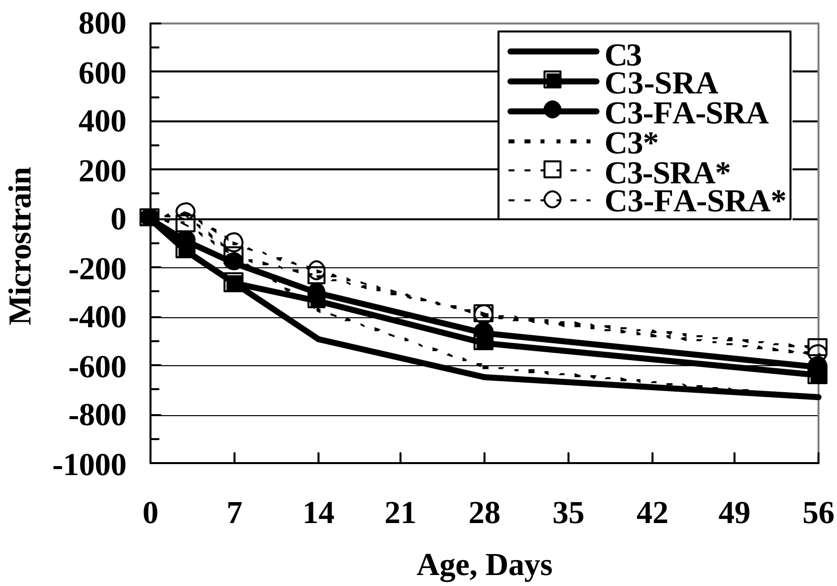


Figure 5-10(b). **Effect of the SRA on Drying Shrinkage of Concrete: Phase 1**

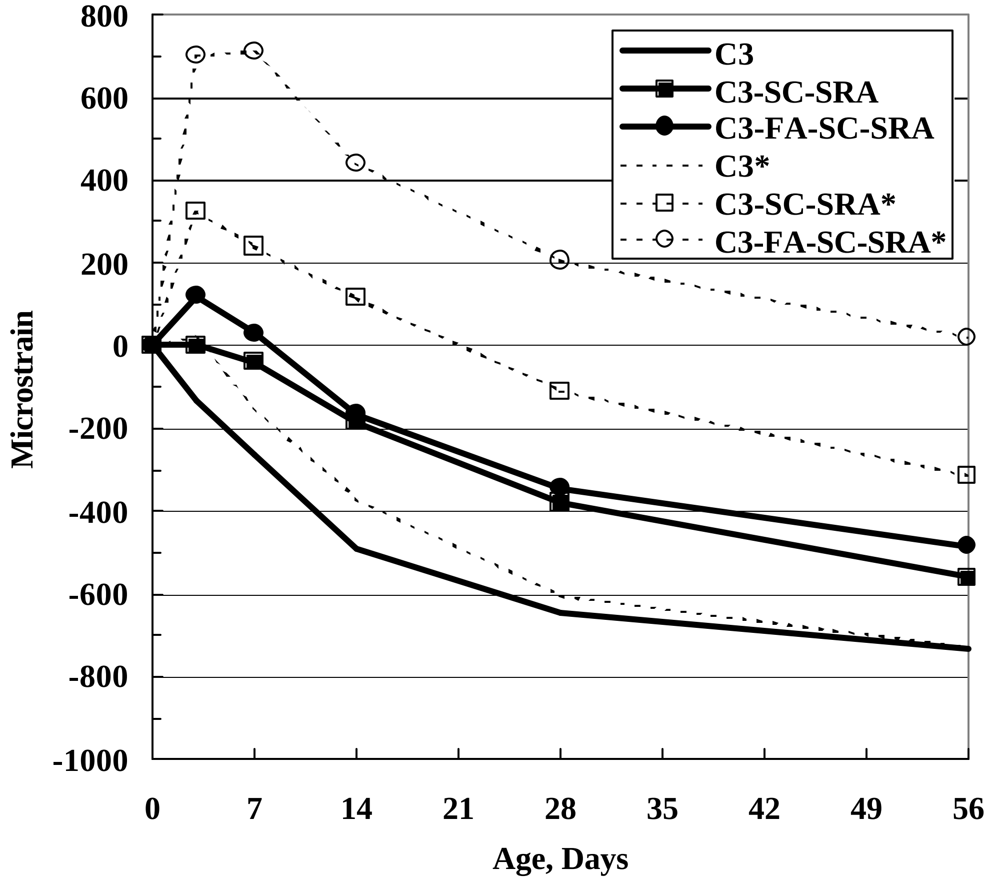


Figure 5-11(a). **Effect of Combining the SCC/SCA, SRA, and Fly Ash on the Drying Shrinkage of Concrete: Phase 1**

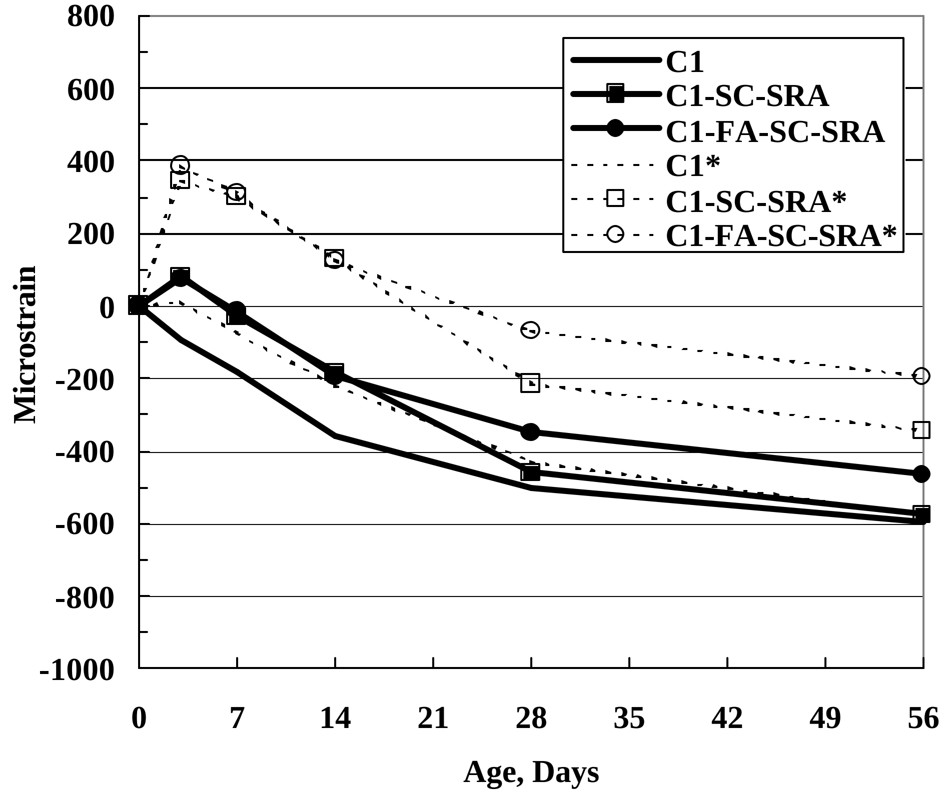


Figure 5-11(b). **Effect of Combining the SCC/SCA, SRA, and Fly Ash on the Drying Shrinkage of Concrete: Phase 1**

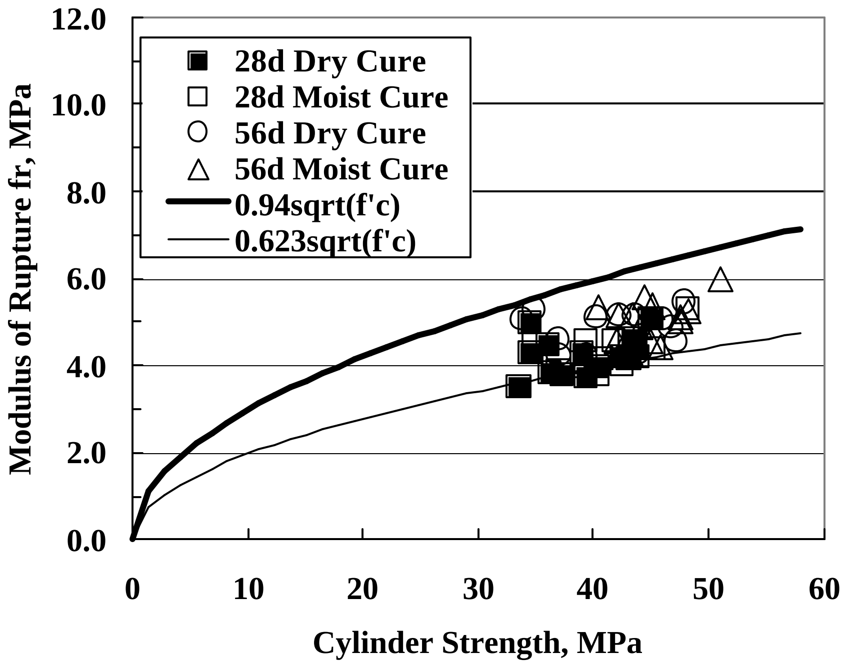


Figure 5-12(a). **28- and 56-Day Concrete Modulus of Rupture as a function of Cylinder Strength: Phase 1**

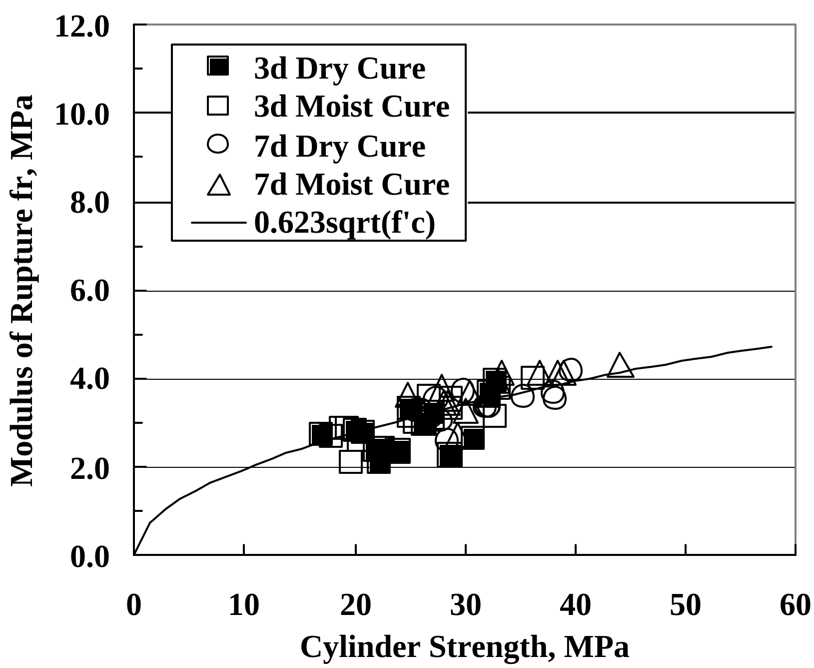


Figure 5-12(b). **Early-Age (3- and 7-Day) Modulus of Rupture as a Function of Cylinder Strength: Phase 1**

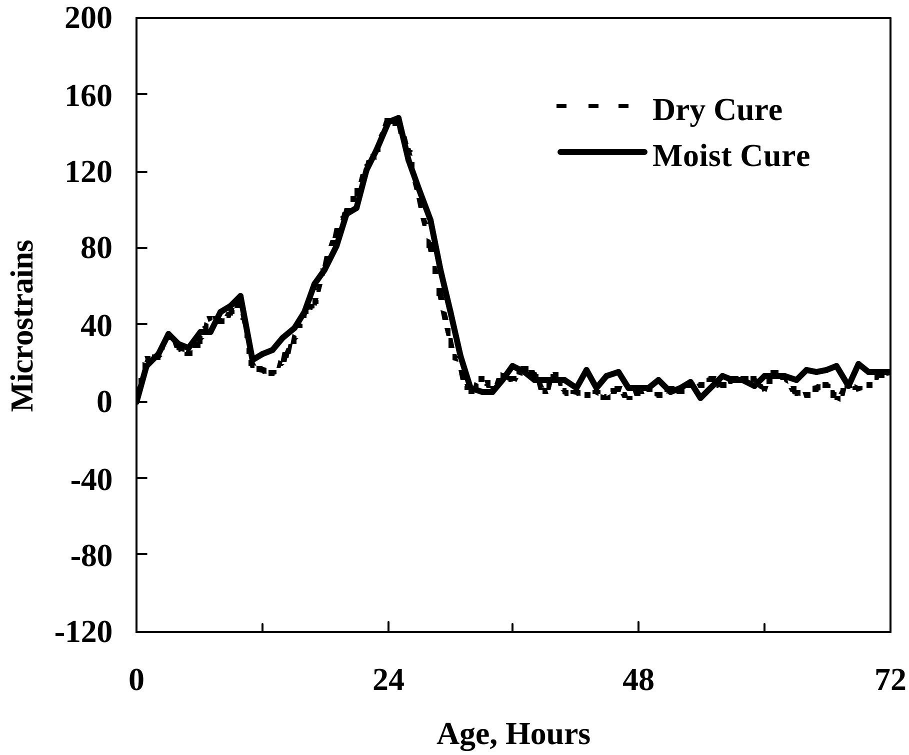


Figure 5-13(a). **Steel Ring Strain of the C3 Control Mix Design for the Two Curing Methods: Phase 1**

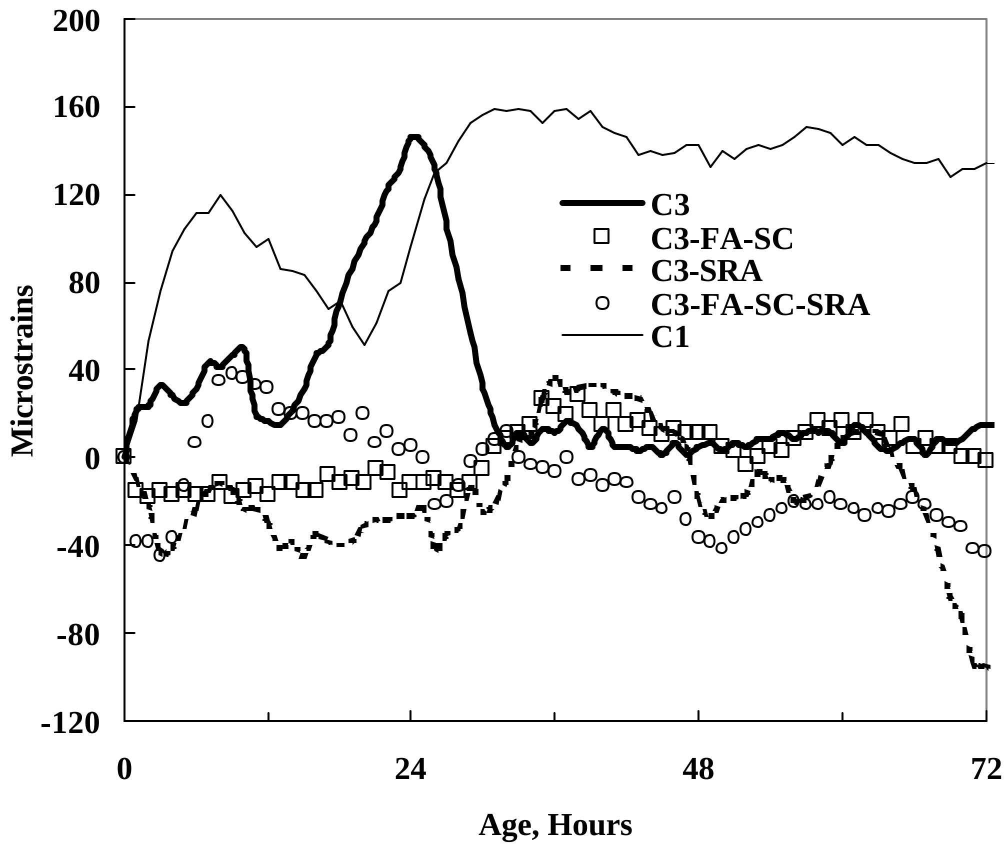


Figure 5-13(b). **Effect of the SRA and SCC/SCA on the Steel Ring Strain of Trial Mix Designs: Phase 1**

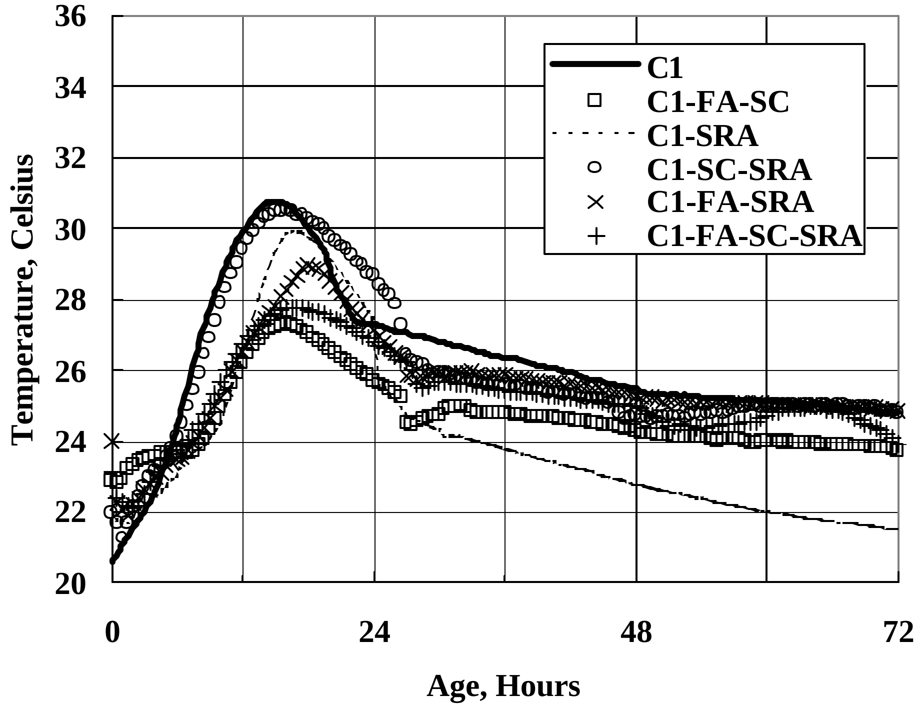


Figure 5-14(a). **Temperature Evolution of all Trial Mix Designs at the C1 Cement Content, Subjected to Moist Cure: Phase 1**

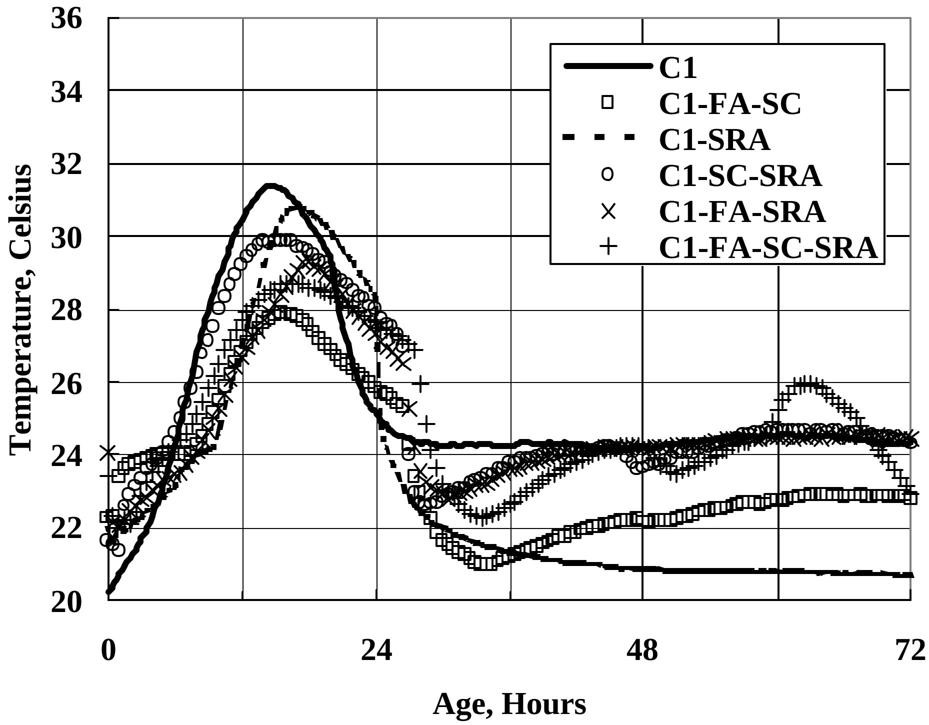


Figure 5-14(b). **Temperature Evolution of Trial Mix Designs at the C1 Cement Content, Subjected to Dry Cure: Phase 1**

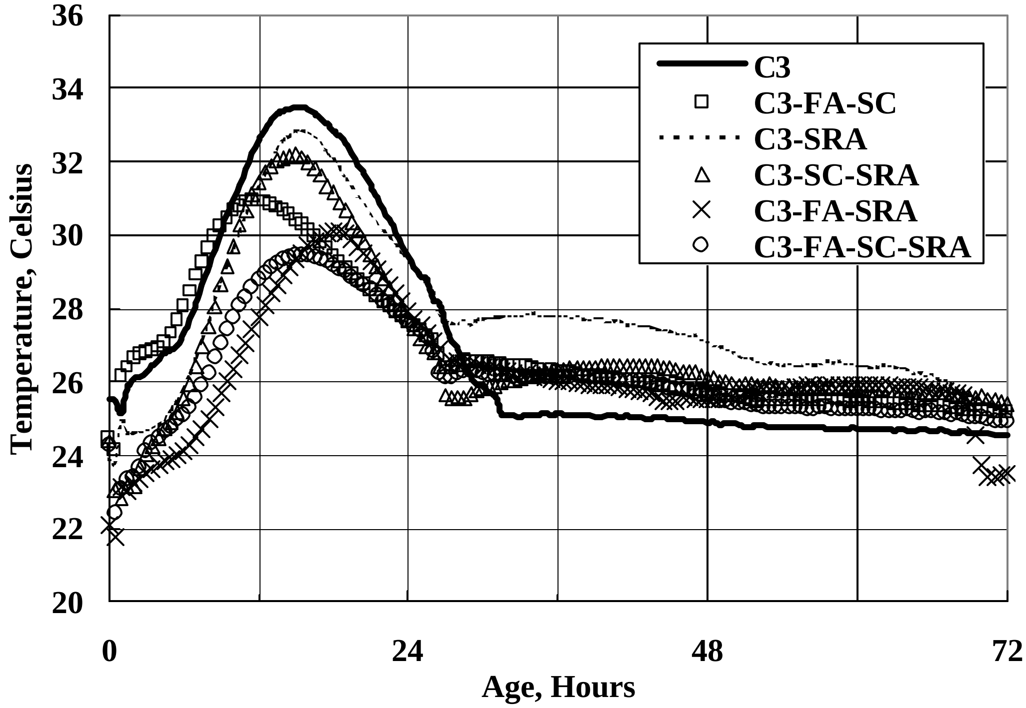


Figure 5-15(a). **Temperature Evolution of Trial Mix Designs of High Cement Content (C3) Subjected to Moist Cure: Phase 1**

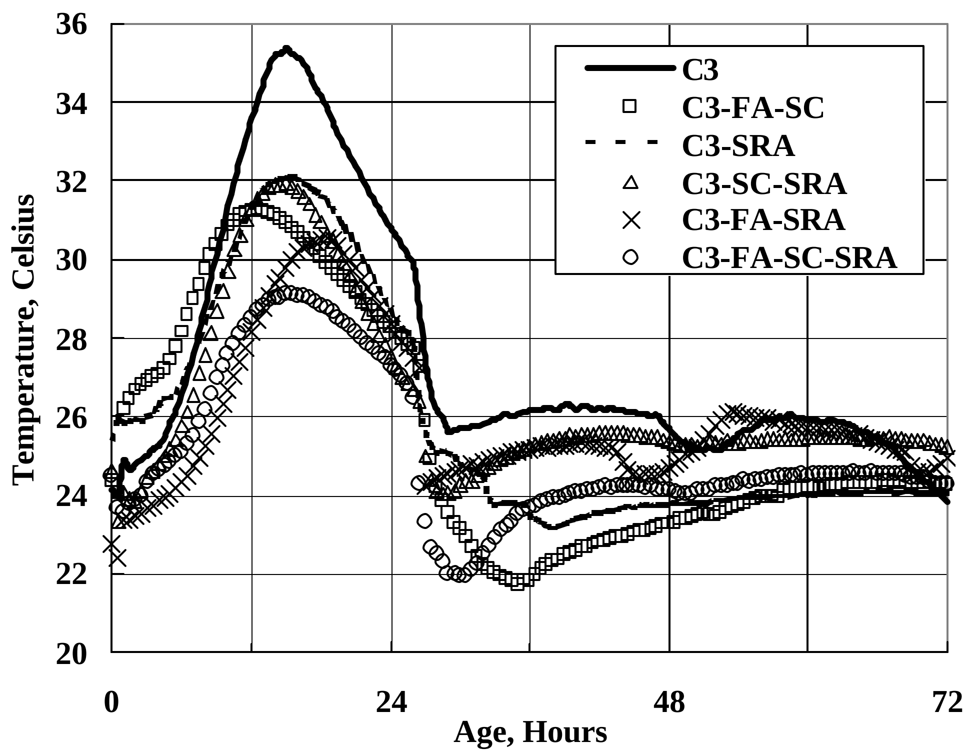


Figure 5-15(b). Temperature Evolution of Trial Mix Designs at High Cement Content (C3) Subjected to Dry Cure: Phase 1

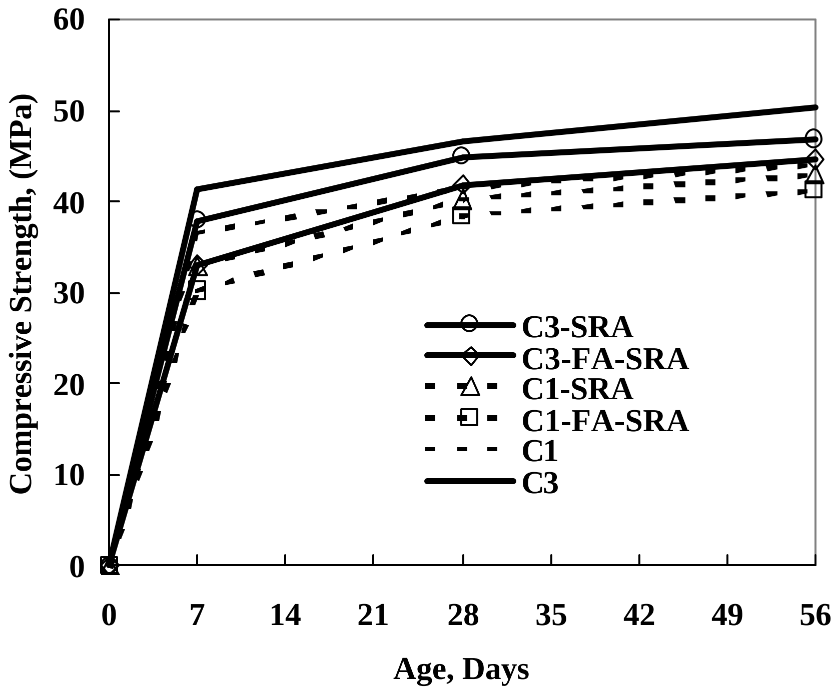


Figure 5-16. Compressive Strength of Mix Designs with SRA: Phase 2

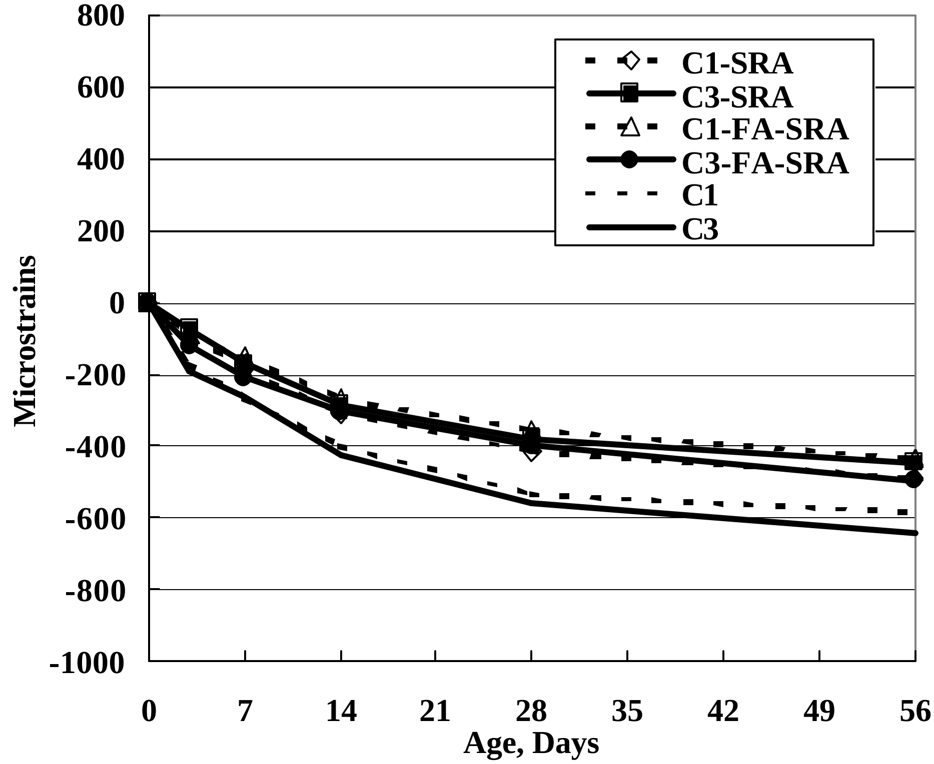


Figure 5-17. Drying Shrinkage of Mix Designs with SRA: Phase 2

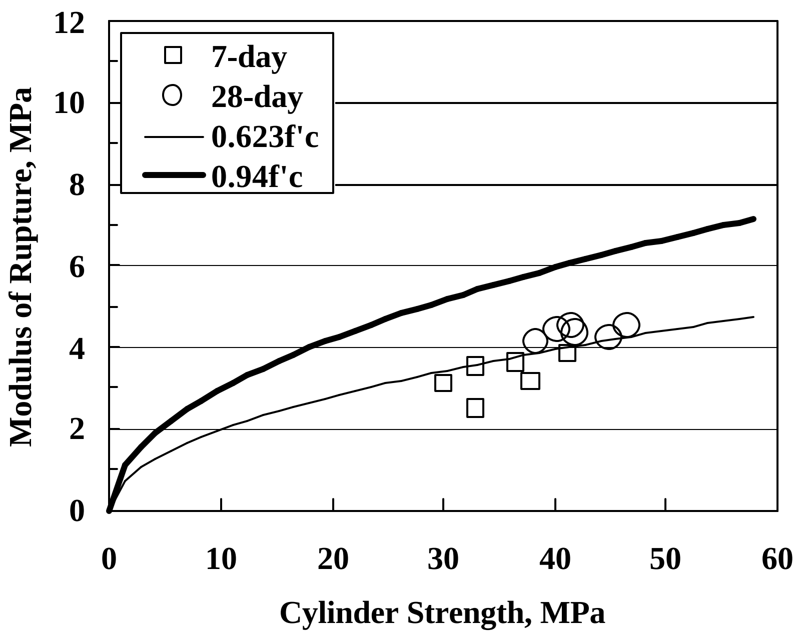


Figure 5-18. **Modulus of Rupture: Phase 2**

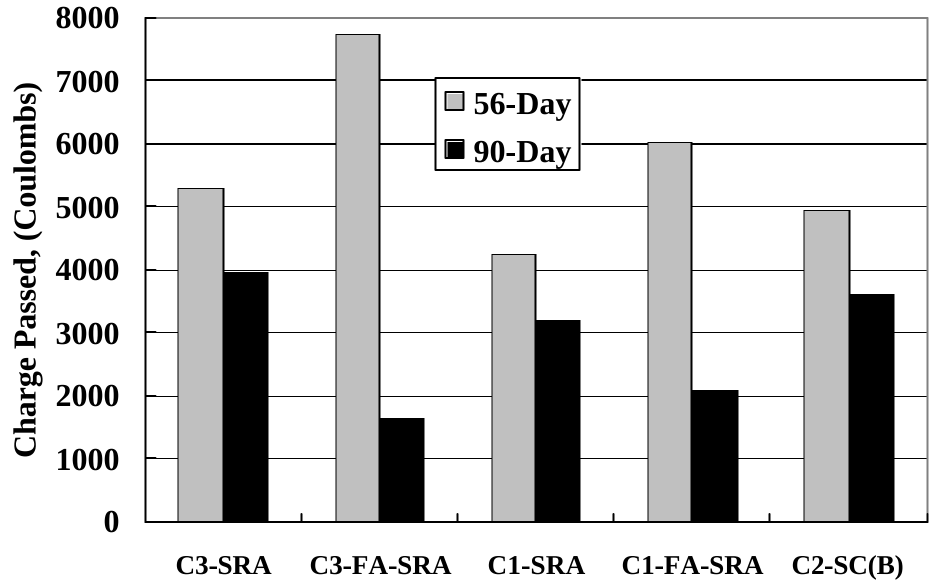


Figure 5-19. **Resistance to Chloride Ion Penetration: Phase 2**

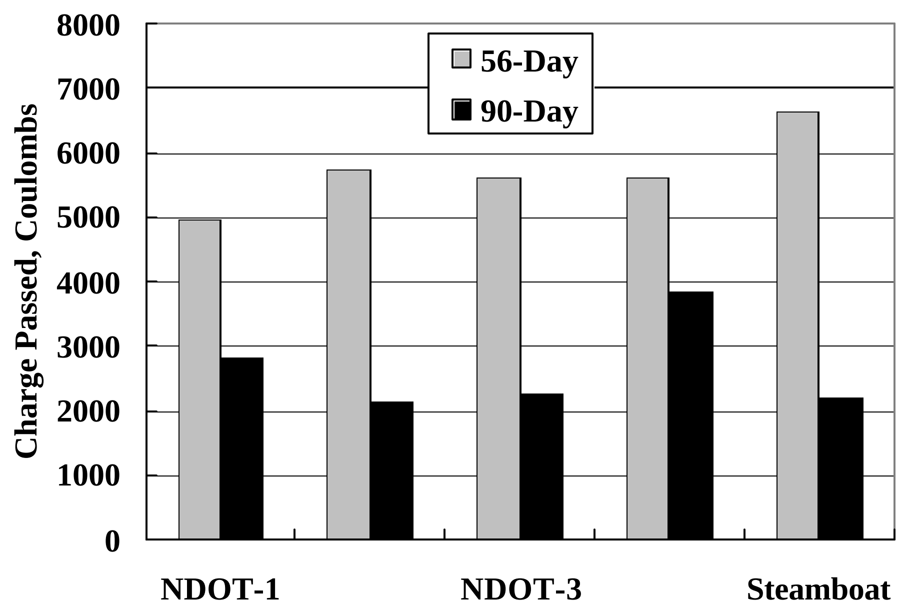


Figure 5-20. **Resistance to Chloride Ion Penetration: Phase 3**

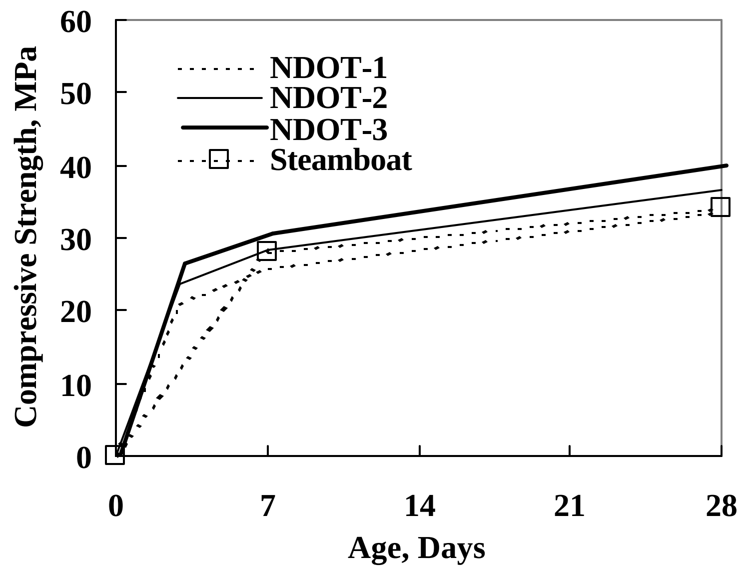


Figure 5-21. **Compressive Strength: Phase 3**

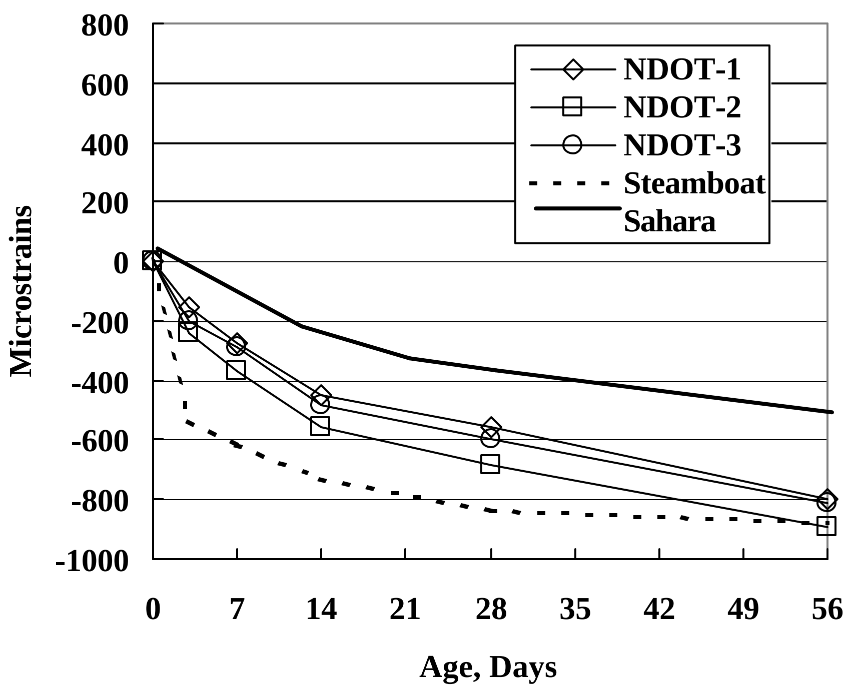


Figure 5-22. **Drying Shrinkage: Phase 3**

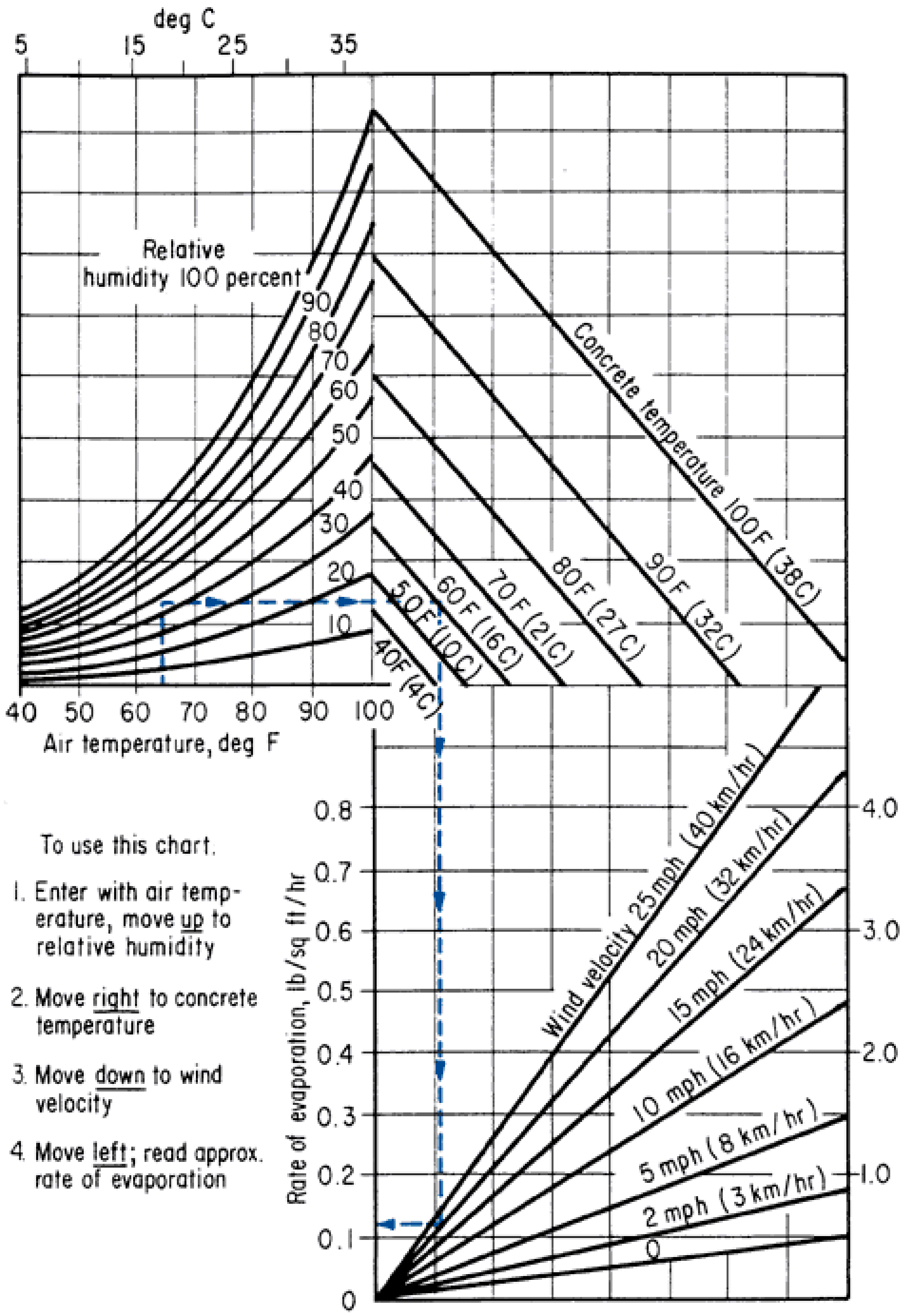


Figure 6-1. **Evaporation Rate Chart From ACI 305.**

Figure 6-2. Location of the Selected Weather Stations in Nevada.

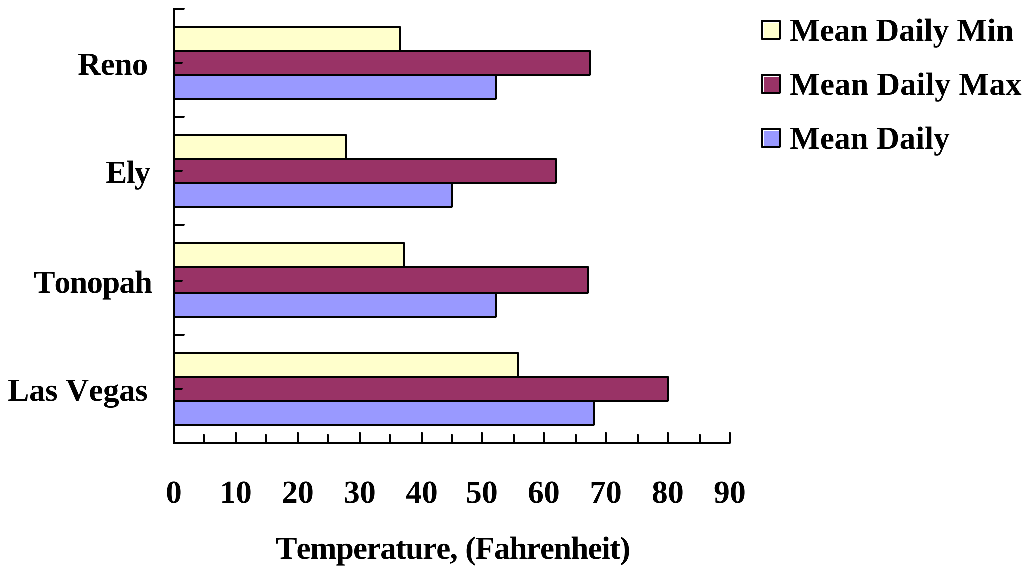


Figure 6-3(a). Annual Mean Daily Temperature.

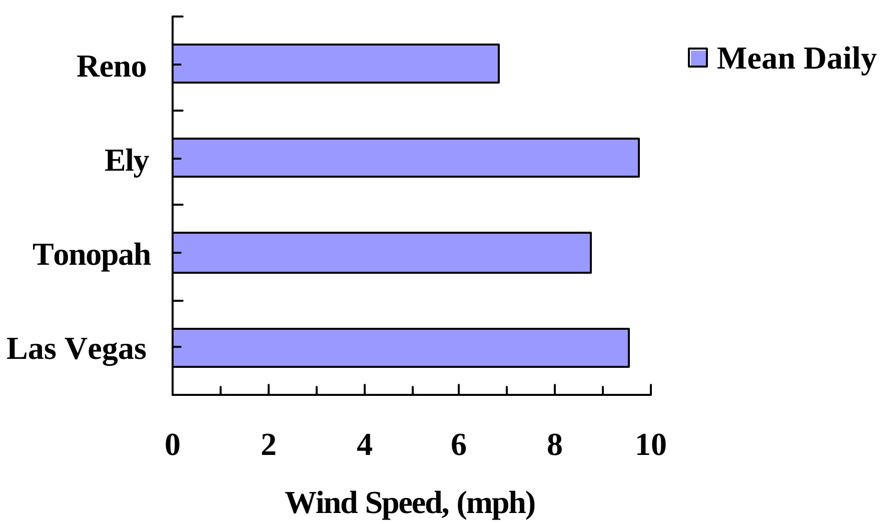


Figure 6-3(b). Annual Mean Daily Wind Speed.

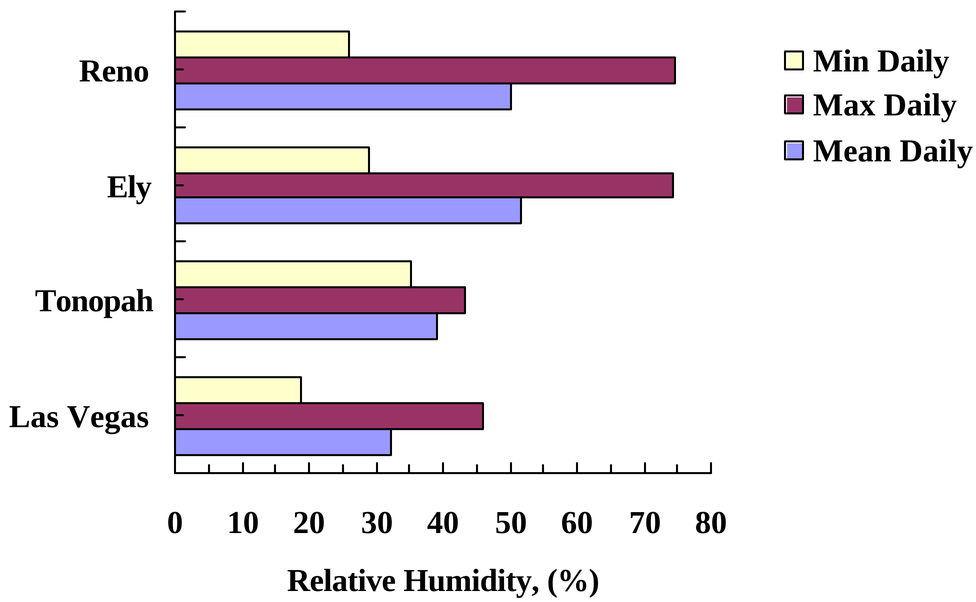


Figure 6-3(c). **Annual Mean Daily Relative Humidity.**

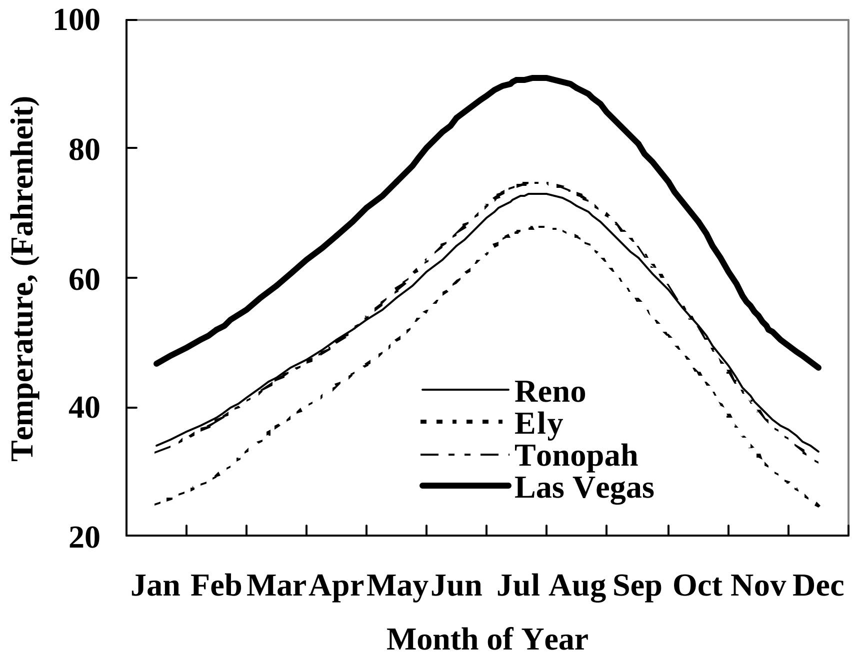


Figure 6-4(a). **Annual Mean Temperature**

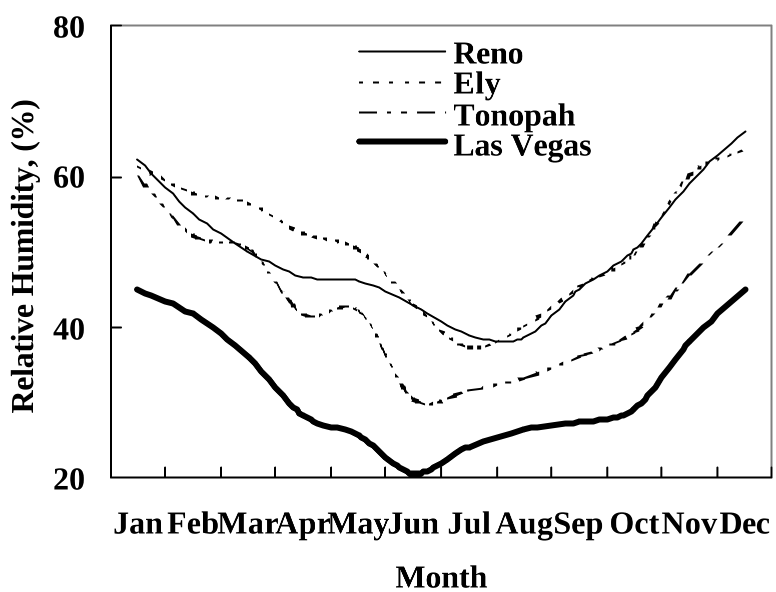


Figure 6-4(b). **Annual Mean Relative Humidity**

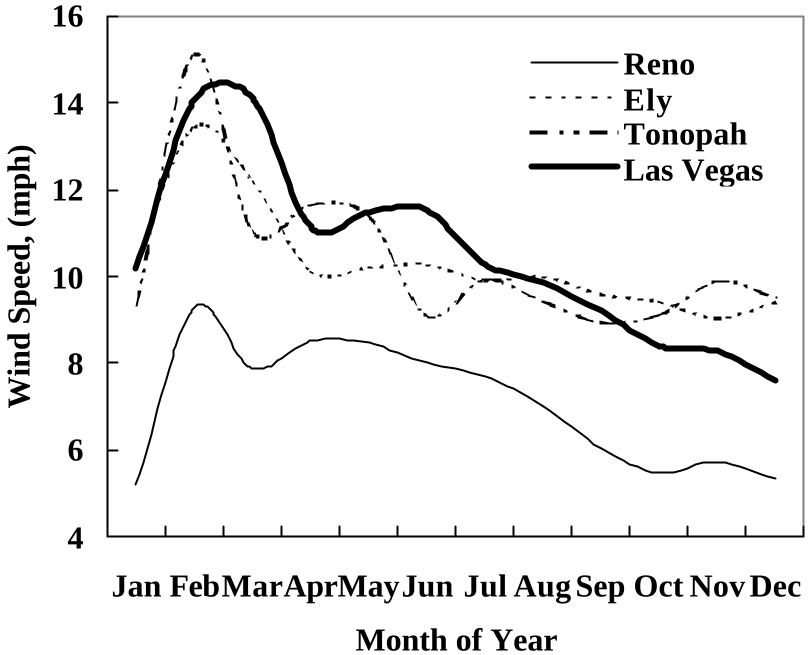


Figure 6-4(c). **Annual Mean Wind Speed**

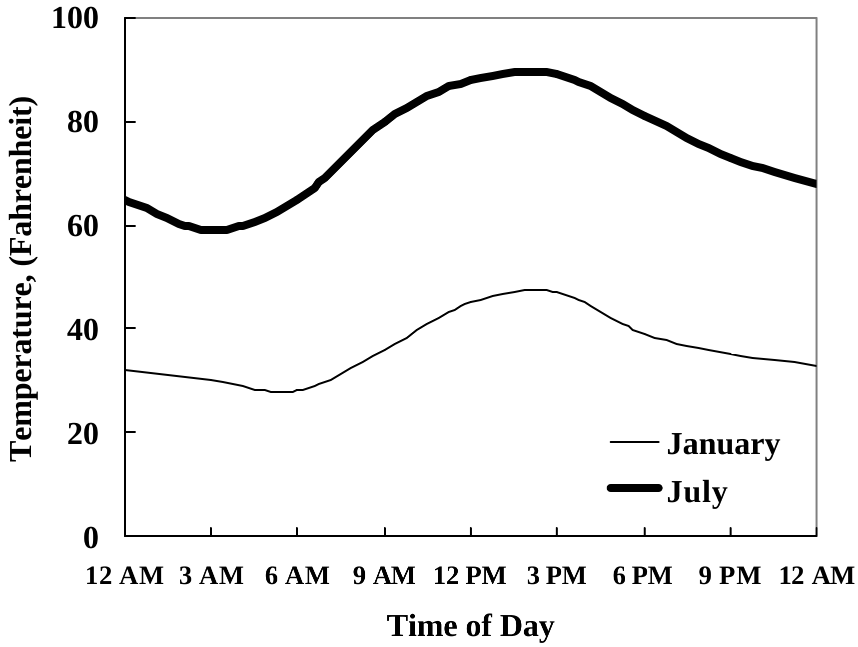


Figure 6-5(a). **Mean Daily Temperature in Reno.**

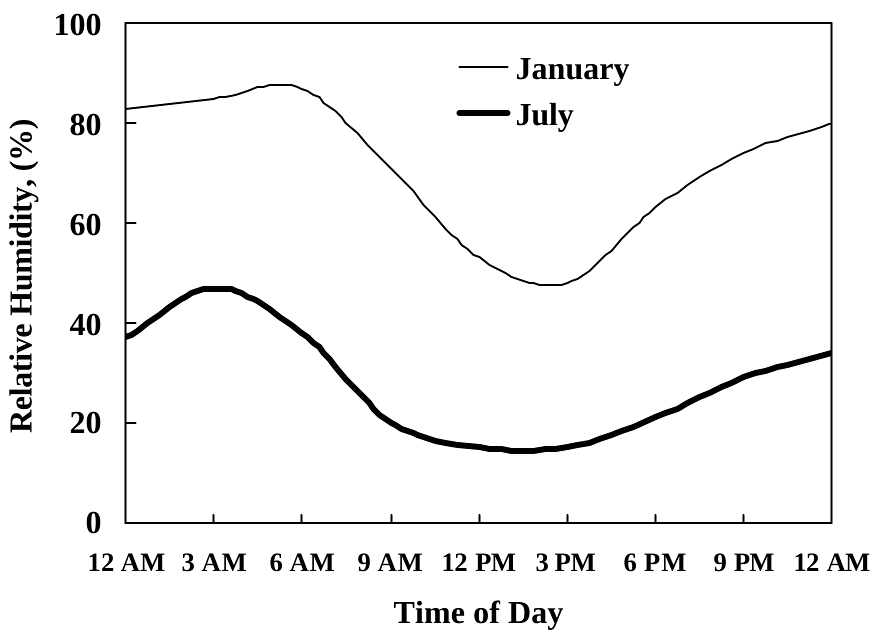


Figure 6-5(b). **Mean Daily Relative Humidity in Reno.**

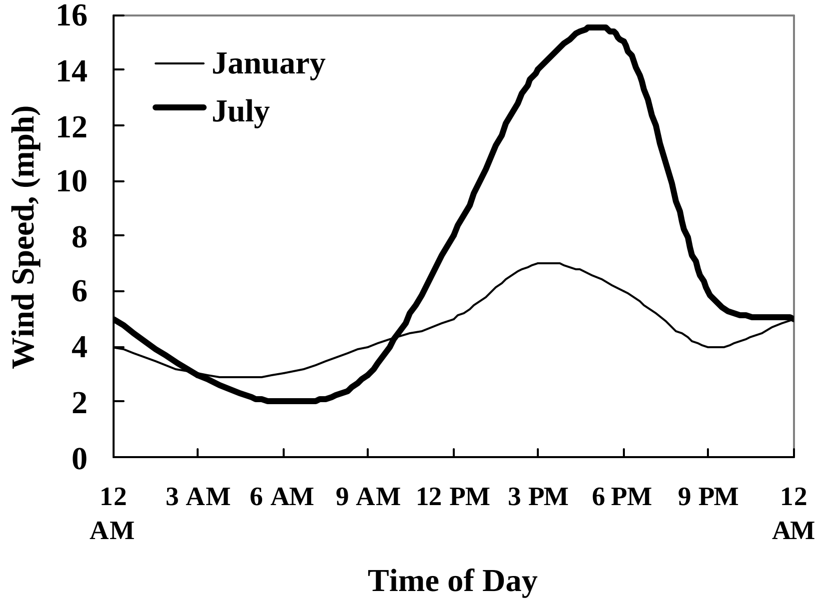


Figure 6-5(c). **Mean Daily Wind Speed in Reno.**

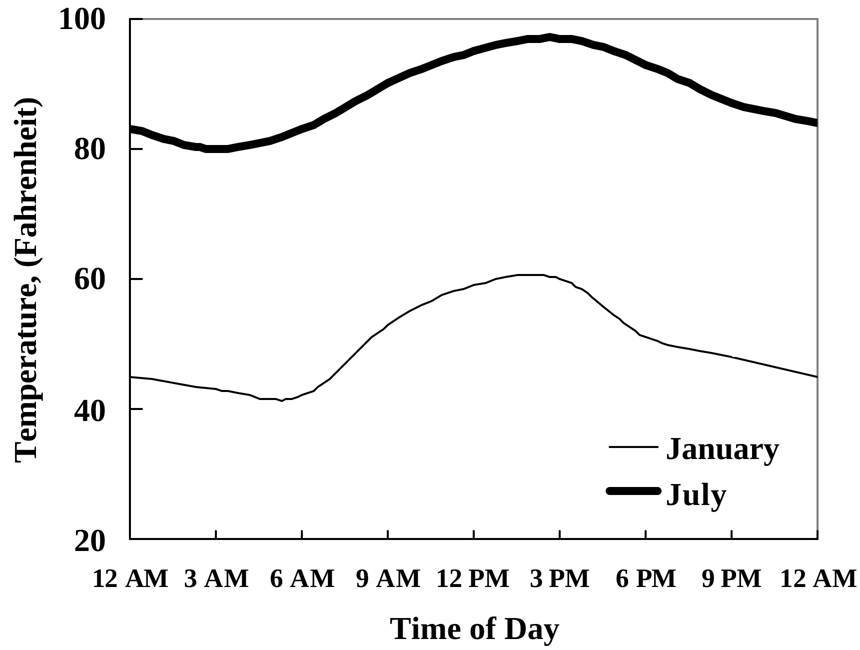


Figure 6-6(a). **Mean Daily Temperature in Las Vegas.**

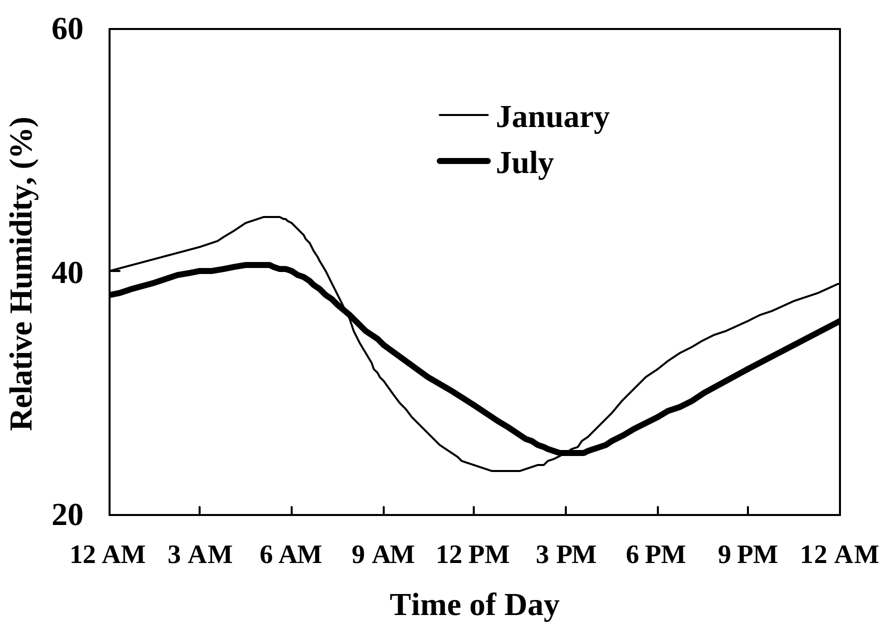


Figure 6-6(b). **Mean Daily Relative Humidity in Las Vegas.**

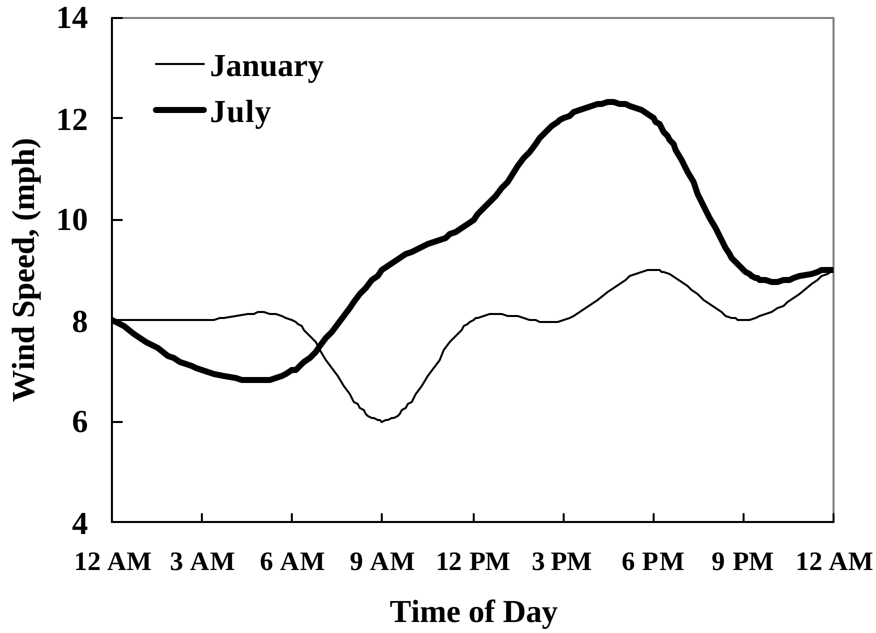


Figure 6-6(c). **Mean Daily Wind Speed in Las Vegas.**

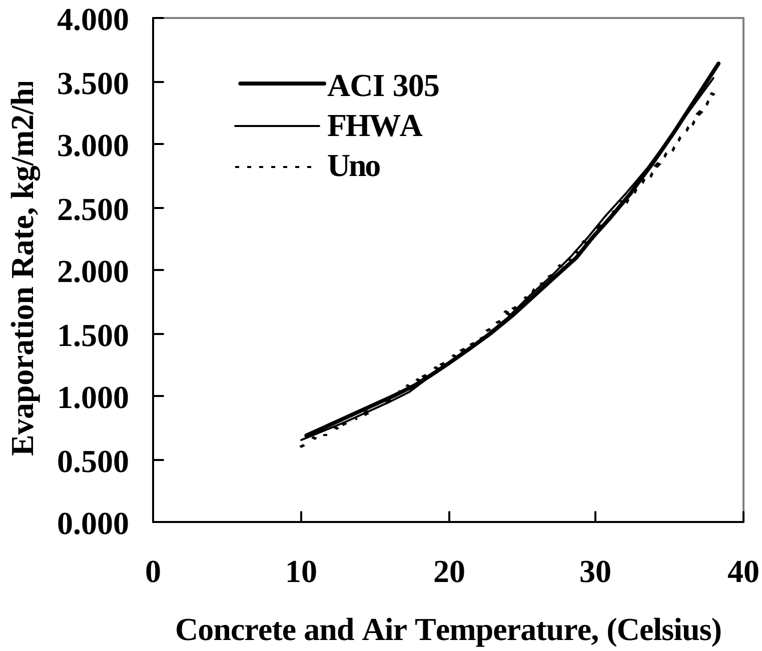


Figure 6-7(a). **Evaporation Rate at Varying Concrete and Air Temperature.**

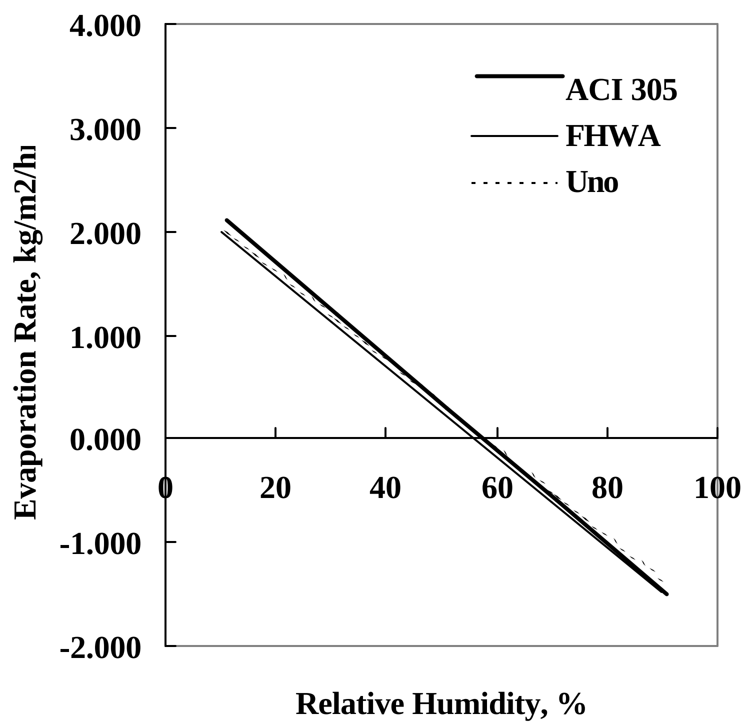


Figure 6-7(b). Evaporation Rate at Varying Relative Humidity.

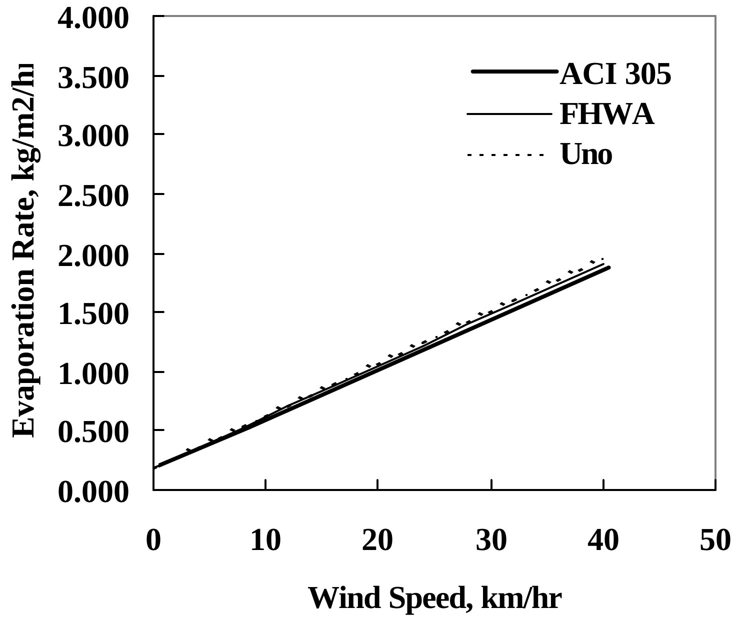


Figure 6-7(c). Evaporation Rate at Varying Wind Speed.

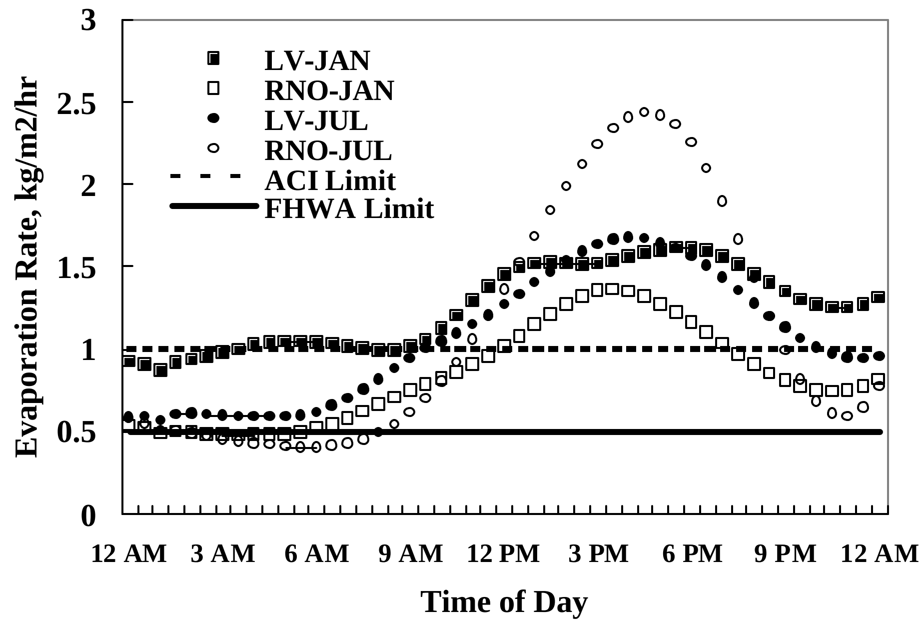


Figure 6-8(a). Evaporation Rate Curves for Nevada Based on Concrete Placement Time of 12:00 AM.

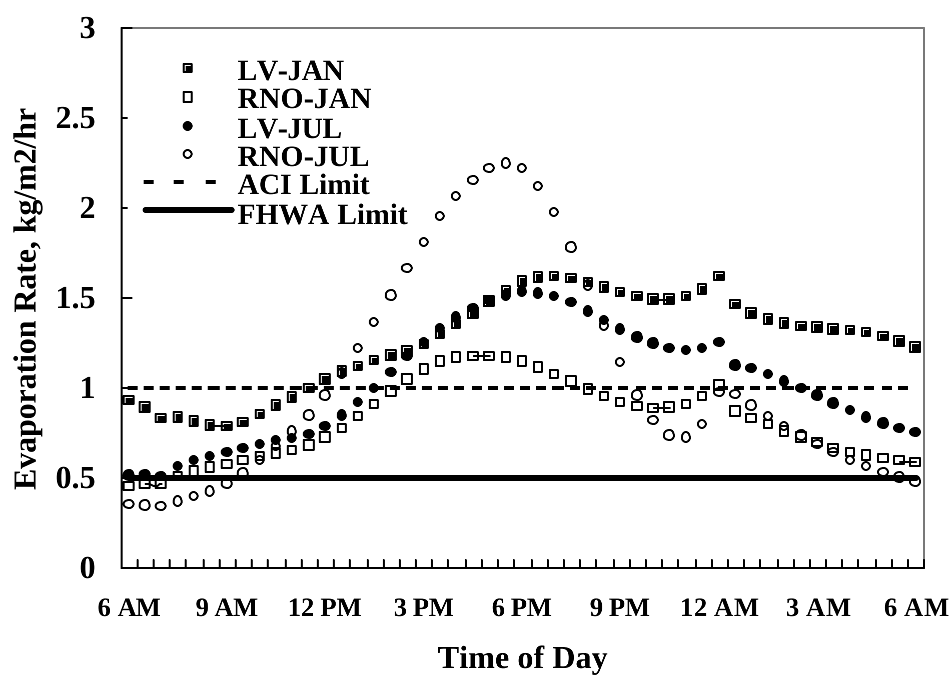


Figure 6-8(b). Evaporation Rate Curves for Nevada Based on Concrete Placement Time of 6:00 AM.

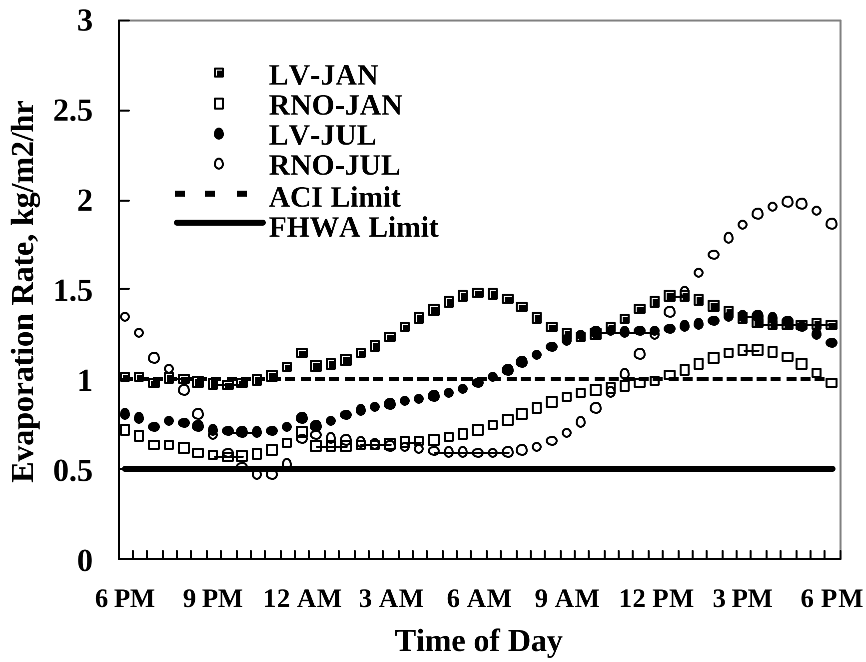


Figure 6-8(c). Evaporation Rate Curves for Nevada Based on Concrete Placement Time of 6:00 PM.

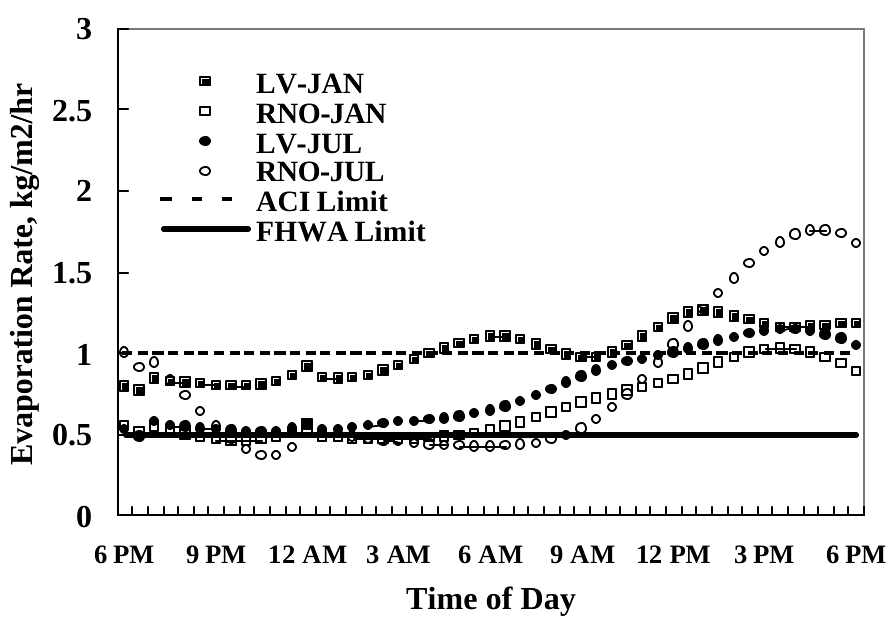


Figure 6-8(d). Evaporation Rate Curves for Mix Design with Reduced Heat of Hydration and Placement Time of 6:00 PM.

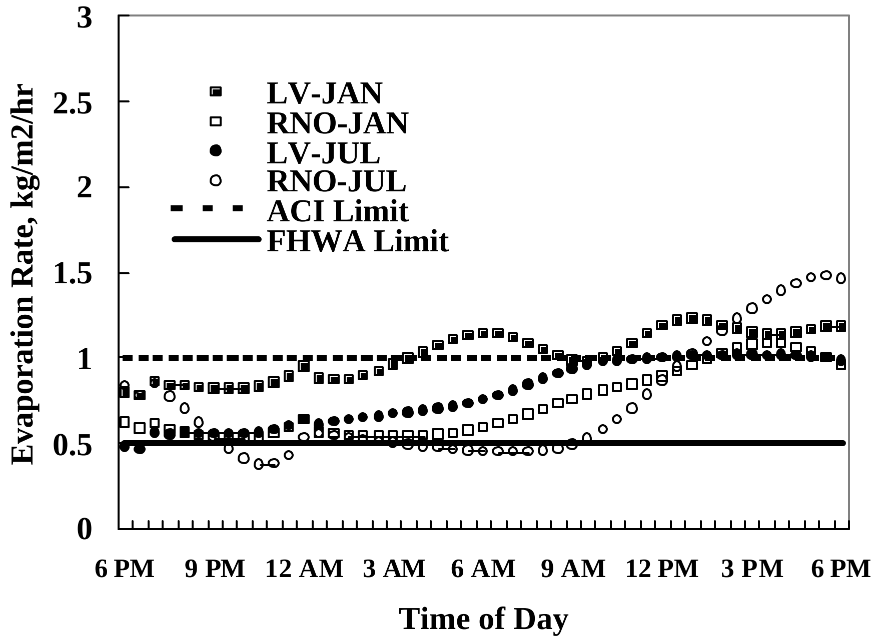


Figure 6-9(a). Evaporation Rate Curves for Nevada with Controlled Relative Humidity at 30%.

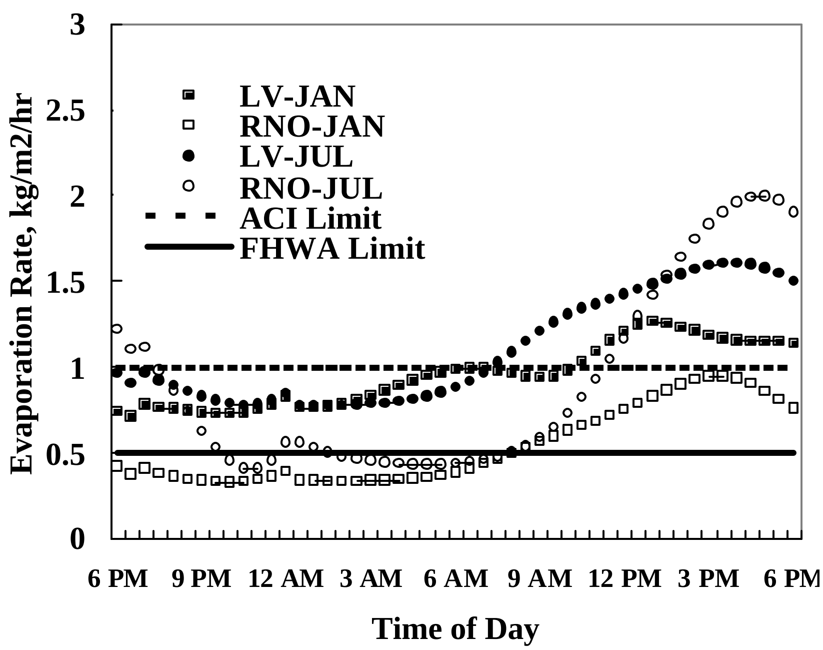


Figure 6-9(b). Evaluation Rate Curves for Nevada with Controlled Air Temperature at 15➤ C (60➤ F).

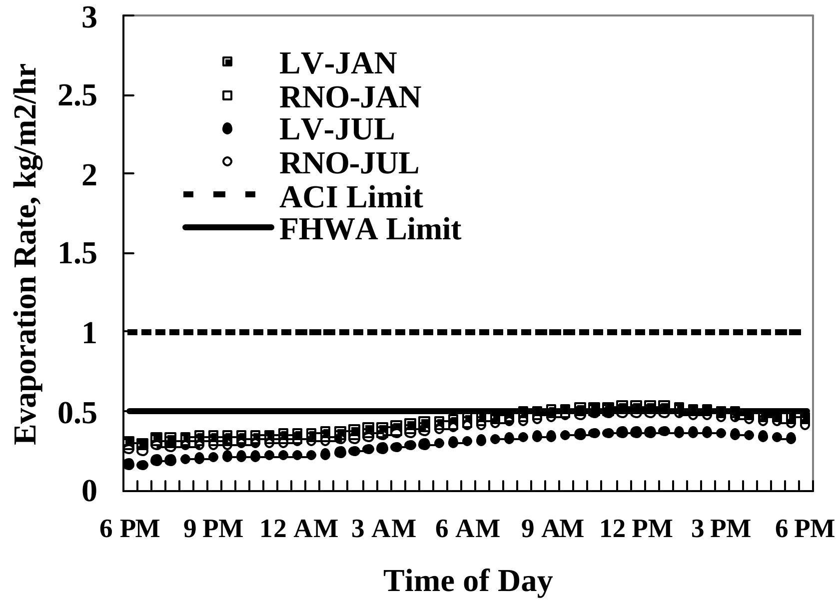


Figure 6-9(c). Evaporation Rate Curves for Nevada with Controlled Wind Speed at 3 km/hr (1.9 mph).

APPENDIX

Table A3-1 Survey: Transverse Cracking in Newly Constructed Bridge Decks

	Survey Questions
1	Do you consider transverse cracking in your newly constructed bridge decks to be a problem? a. Yes b. No
2	What degree of transverse cracking do your bridge decks have? a. Severe b. Moderate c. Mild
3	Do you use any preventative measures such as windshields, sunshades, or fog mists during hot weather deck casts? a. Yes b. No
4	If yes to #3, what measures do you use and under what conditions?
5	Do you have a strict curing specification for bridge deck construction that aims to prevent plastic shrinkage cracking? a. Yes b. No
6	If yes to #5, please attach a copy of this specification
7	How long do you cure the deck? a. < 7days b. 7 days c. > 7 days
8	What type of curing procedure do you generally use for your bridge decks (i.e. continuous moist curing, curing compound and wet burlap combined, pigmented curing compound etc…)?
9	Do you typically limit the size of transverse reinforcement to #5 bars (English units)? a. Yes b. No
10	What is the minimum top cover used for transverse reinforcement in your bridge decks? a. 2 in. b. 2 ½ in. c. 3 in. d. Other
11	What types of girders do you use to support your deck (Circle all applicable and provide a % of the total bridges)? a. Steel ____% b. R/C Box _____% c. Precast, Prestressed I ____% d. Post-Tensioned Box ____% e. Other ____________; ____%
12	Do you consider concrete materials properties such as the modulus of elasticity, heat of hydration, aggregate type, cement type and content to be the crucial factors affecting transverse cracking? If yes, how? a. Yes b. No How: ______________________
13	What type of cement do you typically use in your bridge decks? a. I b. II c. III d. IV e. V f. Other _______
14	Do you provide a minimum or maximum cement content specification for bridge decks? If so, please specify? a. Minimum: ____________ b. Maximum: _____________
15	What type of coarse aggregate do you typically use in your bridge decks (i.e. limestone, river gravel, slate, dolomite, etc…)?
16	What nominal maximum size aggregate do you use in your bridge decks? a. ½ in. b. ¾ in. c. 1 in. d. 1 ½ in. e. Other ________
17	Do you consider the coarse aggregate's modulus of elasticity and coefficient of thermal expansion when selecting the type and source of your coarse aggregate? a. Yes b. No
18	Do you use mineral admixtures such as fly ash, silica fume, or slag as partial replacement in your bridge deck mix designs? a. Yes b. No
19	If yes to #18, how much cement do you typically replace? a. 5-10% b. 11-15% c. 16-20% d. 21-25% e. >25% f. Other: ____________; _____%
20	Do you use chemical admixtures such as water reducers, superplasticizers, and air entrainment etc… in your bridge deck mix designs? a. Yes b. No
21	If yes to #20, indicate what chemical admixtures are typically used. Circle all applicable answers. a. Water Reducers b. Air Entrainers c. Superplasticizers d. Accelerators e. Retarders f. Other __________________________
22	Do you use high or normal strength concrete in your bridge decks? Please specify typical design strength used. a. High _____________ b. Normal _____________
23	Are you familiar with the NCHRP Report 380 on transverse cracking in newly constructed bridge decks? a.

	Yes b. No
24	Have you conducted any research pertaining to transverse cracking in your newly constructed bridge decks? a. Yes b. No
25	If you are familiar with NCHRP Report 380, then have you used the cracking tendency test developed by the NCHRP Report 380 researchers? a. Yes b. No
26	If yes to #25, did you find the results of this testing helpful? a. Yes b. No
27	Would you like a summary of the results of this survey? a. Yes b. No

Table A5-1 Compressive Strength Test Results: Phase 1

Mix Identification	Compressive Strength*, MPa**			
	3-day	7-day	28-day	56-day
C1	27, 29	35, 37	43, 43	44, 45
C1-FA-SC	20, 18	26, 28	35, 35	34, 40
C1-SRA	24, 27	32, 33	39, 41	43, 44
C1-SC-SRA	21, 19	27, 28	39, 39	42, 42
C1-FA-SRA	23, 25	27, 30	34, 39	37, 43
C1-FA-SC-SRA	17, 19	26, 25	35, 39	35, 42
C2	31, 33	38, 39	44, 45	47, 48
C2-SC$_A$*	29, 29	32, 32	44, 44	46, 48
C2-SC$_B$**	NA	33	45	49
C3	33, 36	40, 44	45, 48	48, 51
C3-FA-SC	22, 20	28, 29	36, 37	37, 43
C3-SRA	32, 33	38, 38	43, 45	47, 47
C3-SC-SRA	26, 26	32, 30	40, 43	44, 46
C3-FA-SRA	25, 27	30, 33	36, 42	40, 44
C3-FA-SC-SRA	22, 20	28, 28	37, 40	39, 45

*All data is the average of three specimens. The first value corresponds to Dry Cure, and the second value corresponds to Moist Cure.

**10 MPa ? 1450.39 psi.

***This mix design was submitted to Dry and Moist cure (Methods 1 and 2).

****Mix C2-SC$_B$ was Lab cured (Method 3).

Table A5-2 Drying Shrinkage Test Results: Phase 1

Mix Identification	Drying Shrinkage*, Microstrain			
	3-day	7-day	28-day	56-day
C1	-95, 10	-185, -75	-500, -430	-595, -575
C1-FA-SC	-45, 170	-180, 5	-600, -465	-765, -575
C1-SRA	-70, 15	-115, -65	-315, -265	-465, -430
C1-SC-SRA	80, 345	-30, 300	-460, -215	-575, -345
C1-FA-SRA	-45, -20	-85, -75	-380, -345	-505, -500
C1-FA-SC-SRA	75, 385	-15, 310	-350, -70	-465, -195
C2	-115, -5	-215, -135	-555, -490	-715, -630
C2-SC_A	85, 235	25, 155	-435, -385	-730, -715
C2-SC_B	400	650	40	-108
C3	-135, 20	-265, -150	-645, -605	-730, -730
C3-FA-SC	30, 400	-225, 215	-625, -500	-760, -665
C3-SRA	-130, -20	-265, -155	-505, -390	-640, -530
C3-SC-SRA	0, 325	-40, 240	-380, -110	-560, -315
C3-FA-SRA	-90, 25	-180, -100	-465, -395	-605, -555
C3-FA-SC-SRA	120, 700	30, 710	-345, 205	-485, 20

? All data is the average of two specimens. Negative value means shrinkage, positive value means expansion. The first value is for dry cure; the second value is for moist cure.

Table A5-3 Modulus of Rupture Test Results: Phase 1

Mix Identification	Modulus of Rupture*, MPa**			
	3-day	7-day	28-day	56-day
C1	3.2, 3.5	3.6, 4.1	4.2, 4.6	5.2, 5.4
C1-FA-SC	2.8, 2.7	3.3, 3.8	4.3, 4.5	5.1, 5.3
C1-SRA	2.3, 3.1	3.4, 4.0	3.7, 4.1	4.4, 4.9
C1-SC-SRA	2.8, 2.9	3.5, 3.5	4.3, 4.0	5.1, 5.1
C1-FA-SRA	2.4, 3.1	3.1, 3.7	3.5, 4.6	4.6, 5.2
C1-FA-SC-SRA	2.7, 2.8	3.3, 3.6	5.0, 4.1	5.3, 4.6
C2	2.6, 3.1	3.7, 4.1	4.4, 5.1	4.9, 5.3
C2-SC$_A$	2.3, 3.3	3.4, 3.6	4.6, 4.2	5.1, 5.1
C2-SC$_B$	NA	2.8	3.6	NA
C3	3.9, 4.0	4.2, 4.3	5.1, 5.3	5.5, 6.0
C3-FA-SC	2.1, 2.1	2.6, 2.7	3.8, 3.9	4.2, 4.3
C3-SRA	3.7, 3.8	3.6, 4.1	4.1, 4.4	4.6, 5.0
C3-SC-SRA	3.0, 3.0	3.4, 3.3	4.0, 4.0	4.3, 4.4
C3-FA-SRA	3.3, 3.6	3.7, 4.1	4.5, 4.6	5.1, 5.6
C3-FA-SC-SRA	2.4, 2.6	3.0, 3.5	3.8, 3.8	4.2, 4.6

*Calculated using ASTM C 78. The first value is for dry cure; the second value is for moist cure.
**1 MPa ? 145 psi.
Values are average of two specimens.

Table A5-4 Compressive Strength Test Results: Phase 2

Mix Identification	Compressive Strength, MPa (psi)			
	3-day	7-day	28-day	56-day
C1	NA	36 (5296)	41 (6018)	44 (6387)
C1-SRA	NA	33 (4773)	40 (5833)	43 (6220)
C1-FA-SRA	NA	30 (4371)	38 (5570)	41 (5971)
C3	NA	41 (5976)	46 (6758)	50 (7296)
C3-SRA	NA	38 (5499)	45 (6518)	47 (6786)
C3-FA-SRA	NA	33 (4789)	42 (6066)	44 (6470)

Values are average of three specimens.

Table A5-5 Drying Shrinkage Test Results: Phase 2

Mix Identification	Drying Shrinkage*, Microstrain			
	3-day	7-day	28-day	56-day
C1	-170	-265	-535	-585
C1-SRA	-75	-170	-380	-445
C1-FA-SRA	-90	-150	-355	-435
C3	-190	-265	-560	-640
C3-SRA	-90	-185	-415	-490
C3-FA-SRA	-120	-210	-400	-495

Values are average of two specimens.

Table A5-6 Modulus of Rupture Test Results: Phase 2

Modulus of Rupture, MPa (psi)	Mix Identification					
	C1	C1-SRA	C1-FA-SRA	C3	C3-SRA	C3-FA-SRA
@ **7 days**	3.6 (523)	3.5 (510)	3.0 (446)	3.8 (553)	3.6 (523)	3.1 (456)
@ **28 days**	4.5 (653)	4.4 (640)	4.1 (596)	4.6 (656)	4.4 (640)	4.3 (630)

Values are average of two specimens.

Table A5-7 Chloride Ion Penetration Test Results: Phase 2

Coulombs Passed	Mix Identification				
	C1-SRA	C1-FA-SRA	C3-SRA	C3-FA-SRA	*C2-SC$_B$*
@ **56 days**	**4262**	**6040**	**5296**	**7757**	***5957***
Specimen 1	3775	6510	6138	7765	*5600*
Specimen 2	4750	5570	4455	7750	*4314*
@ **90 days**	**3212**	**2085**	**3960**	**1637**	***3623***
Specimen 1	2985	2045	4150	1790	*3269*
Specimen 2	3440	2125	3770	1484	*3977*

Bold values are average of the two specimens. Test results Reported by W. R. Grace Products Laboratory for the first 4 mix designs.
Column in Italics represents mix design tested by the University of Nevada Reno.

Table A5-8 Hardened Air-Void System Test Results*: Phase 2

Parameters	C1-SRA	C1-FA-SRA	C3-SRA	C3-FA-SRA
Total Air Content, %	3.6	3.7	2.7	3.3
Average Chord Length, mm (in.)	0.134 (0.0053)	0.116 (0.0046)	0.208 (0.0082)	0.134 (0.0053)
Spacing Factor, mm (in.)	0.279 (0.0110)	0.180 (0.0071)	0.228 (0.0090)	0.185 (0.0073)
Specific Surface, mm^{-1} (in.$^{-1}$)	19.48 (495)	29.68 (754)	29.56 (751)	33.89 (861)
Voids, mm^{-1} (in.$^{-1}$)	4.4	6.9	5.0	7.0
Calculated Paste Content, %	24.6	25.6	30.9	32.1
Total Traverse Length, mm (in.)	2286 (90)	2286 (90)	2286 (90)	2286 (90)

*Test results provided by W. R. Grace Products Laboratory.

Table A5-9 NDOT Mix Designs and Nevada Bridges Test Results: Phase 3

Parameters	Test Method	Mix Design Identification				
		NDOT-1	NDOT-2	NDOT-3	Steamboat	Sahara
Compressive Strength*, MPa***	**ASTM C 39**					
	3-day	21	23.5	26	28 (7d)	NA
	7-day	25.8	28.3	30.4	32.2 (14d)	NA
	28-day	33.7	36.6	39.5	34.1	NA
	56-day	NA	NA	NA	34.8	NA
Drying Shrinkage, Microstrain**	**ASTM C 157**					
	@ 3 days	-157	-240	-200	-535	NA
	@ 7 days	-277	-367	-290	-616	NA
	@ 14 days	-450	-557	-480	-729	-260
	@ 28 days	-560	-683	-600	-841	-410
	@ 56 days	-800	-890	-810	-876	-555
Chloride Ion Penetration, Coulombs Passed**	**ASTM C 1202**					
	@ 56 days	4962	5724	5615	6649	5621
	@ 90 days	2837	2159	1634	2214	3840

*Test results provided by Stantec Consulting Inc., Reno, NV, except for Steamboat and Sahara All values are average of three specimens

**Tests performed by the University of Nevada Reno. For ASTM C 157, values are average of three specimens. For ASTM C 1202, values are average of two specimens.

***10 MPa = 1450 psi.

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