



INVESTIGATE CAUSES & DEVELOP
METHODS TO MINIMIZE EARLY-AGE DECK
CRACKING ON MICHIGAN BRIDGE DECKS

DECEMBER 2003

Center for Structural Durability
A Michigan DOT Center of Excellence

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RESEARCH

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16. Abstract The research identified the major parameters influencing the concrete transverse deck cracking and made recommendations to modify these parameters in order to minimize deck cracking. The synthesis of the data collected revealed that the tensile stress due to early-age thermal load alone could cause deck cracking. Volume change of concrete due to temperature and shrinkage occurs simultaneously. An increase in drying shrinkage arising from delays in concrete placement and wet curing also affect the deck cracking. Additionally, drying shrinkage, beyond the very early ages, increases the crack widths that have previously formed due to thermal loads. Concrete parameters influencing the thermal load levels are: cement type, content, and fineness, ambient temperature at the time of concrete placement, and the time of inception of curing. An important recommendation of this study is the implementation of measures to control and manage thermal and shrinkage stresses in RC decks. The first conclusion is related to current practice. If the curing related stipulations of the Michigan Department of Transportation - Standard Specifications for Construction is strictly adhered to, the density of transverse deck cracks will be reduced. This research established that approximately a 20 °F of thermal load initiates deck cracking. Second, in order to reduce transverse cracking, the primary recommendation is to develop and optimize project specific mix design for the minimization of thermal load. Additional recommendations include the reduction and/or substitution of cement with mineral admixtures and use of current and forecast weather data in optimizing the mix design and placement time in order to minimize the thermal loads.			
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EXECUTIVE SUMMARY

The need for this research was based on an observed deck deterioration mechanism that is accelerated by the existence of cracks. The primary objective of this research was to identify the major parameters influencing concrete deck cracking. The second objective was to develop recommendations for the modification of these parameters that are within the control of the bridge designer, the materials engineer, the contractor, and/or the maintenance engineer. The project tasks consisted of literature review, nationwide survey on the subject of reinforced concrete (RC) deck cracking, field inspection and data collection of existing RC bridge decks of age five years or less, construction monitoring of new decks, laboratory and field testing, and data analysis and synthesis.

The literature review revealed that early-age cracking is the single most prevalent deck distress reported by all of the State Highway Agencies. Although there have been many studies performed with regard to the cause of early-age deck cracking, the problem still persists. Synthesis of the literature indicated that the restrained thermal and shrinkage effects coupled with construction practices are the main parameters influencing deck cracking.

The information extracted from the nationwide survey regarding the experience of other states with the problem of early-age deck cracking was also compared; specifically, with the states of the Central North East Region (Illinois, Minnesota, New York, Pennsylvania, and Wisconsin) with a similar climatic exposure to that of Michigan. All of these states indicated that they observe early-age cracking on concrete bridge decks and the prevalent type is transverse cracking. In order to control deck cracking, the most popular measures taken by these states are: use of mineral admixtures, and changes to mix design and curing procedure. Illinois, New York, and Pennsylvania started adding fly ash, silica fume, and ground granulated blast-furnace slag as mineral additives in their mix design, whereas Wisconsin has been adding fly ash and ground granulated blast-furnace slag. New York, Minnesota, and Wisconsin are using retarder, air entrainer, and mid-range water reducer admixtures, but Illinois and Pennsylvania are using only air entrainer in their concrete mix design. The most often specified deck thickness among the states is 8 inches, but Minnesota utilizes a 9-inch deck. The common curing practice among

these states is continuous wet curing with the exception of Illinois. The top three causes of cracking identified by the respondents from these States are substandard curing, construction practice, and mix design. These top three causes of cracking matches with Michigan's responses. Illinois and Minnesota use a reduced cement content of 6 sacks/yd³.

The field inspection data analysis of twenty bridges indicated that crack density is higher on continuous bridges than on simple span bridges. Transverse and diagonal crack densities are higher on side-by-side box-beam bridges than on other girder types. Bridges with PCI girders show minimum longitudinal crack density compared with other bridge girder types (i.e., steel, side-by-side box-beam, and spread box-beam). However, there is no clear relationship between deck crack density and bridge skew, deck thickness, span length, or ADTT.

During construction monitoring, observed curing procedures were often in conflict with the Michigan Department of Transportation-Standard Specifications for Construction. A Standard Specification requirement, which states that no more than 10 feet of textured concrete surface should be left exposed without curing compound at any time, was never observed. In two of the five deck placement projects, curing compound was applied upon the placement of the full deck. In the Standards Specifications for Construction, wet curing requires covering concrete with clean, contaminant-free wet burlap as soon as the curing compound is sufficiently dried and the concrete surface is sufficiently hardened. Again, according to the Specifications, wet curing should commence within two hours upon concrete placement. In all five of the deck replacement projects monitored, the burlap when first placed was never wet and was placed after 12 to 36 hours upon concrete placement.

Laboratory tests on concrete samples taken during construction monitoring indicate that out of the five bridges monitored, three had a deck concrete 28-day compressive strength in excess of 6000 psi. Concrete with a 28-day compressive strength greater than 6000 psi is classified as high-strength concrete and needs to comply with special construction and curing procedures. The laboratory tests also showed that the gain in compressive strength and elasticity modulus from 3 to 7 days were rapid, indicating high early strength concrete properties. Concrete with such properties generates high thermal loads during hydration and high drying shrinkage during early ages. Consequently, increased deck cracking should be expected.

The synthesis of all the data collected revealed that the tensile stresses due to early-age thermal load alone could cause deck cracking. Volume change of concrete under thermal and shrinkage effects occur simultaneously. An increase in drying shrinkage from delays in wet curing will increase tensile stresses. Drying shrinkage, upon curing, will increase crack width that have formed under thermal loads. For a fixed mix design, the ambient temperature at the time of concrete placement governs the early-age concrete thermal properties. Concrete mix parameters controlling the thermal load are the cement type, content, and fineness, and the time of inception of curing. The temperature difference between the peak temperature during hydration and the ambient temperature establishes the thermal load on the deck concrete. The thermal load controls the magnitude of the tensile stresses that develops in the deck. Use of retarders in the concrete mix delays the hydration process and may be an advantage or a drawback depending on the ambient temperature at the time of peak hydration temperature.

The first conclusion of this study is related to current practice. If the curing related stipulations of the Michigan Department of Transportation - Standard Specifications for Construction is strictly adhered to, the density of transverse deck cracks will be reduced. This research established that approximately a 20⁰ F of thermal load initiates deck cracking. Second, in order to reduce transverse cracking, the primary recommendation is to develop and optimize project specific mix design for the minimization of thermal load. As an incentive for developing a project specific mix design, peak concrete temperature during hydration may be defined as a performance parameter. The limits to the hydration temperature may be specified in the Standard Specifications. Inclusion of concrete hydration temperature in the specifications is feasible since it is measurable and with certain limitations (cement mill properties) is within the control of the contractor by the concrete mix design and curing.

This study recommends a continuation research project for development of mix parameters that are optimized for the reduction of thermal and shrinkage loads. The proposed research should also include the development of tools that will utilize the current and forecasted climatic data as well as cement and aggregate properties in order to determine an optimized project specific mix design.

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1 INTRODUCTION

Reinforced concrete (RC) bridge decks are designed and constructed to provide a durable riding surface and at the same time to meet requirements concerned with, for example, safety, geometry, etc. They are subjected to severe and harsh environment in Michigan, such as high live load stresses, alternate wetting and drying, freeze-thaw cycles, severe thermal gradients, effects of deicing salt, etc. The deteriorating actions on RC bridge decks can be categorized into two groups: 1) physical actions generating internal stresses and 2) chemical action of agents penetrating into the deck material and causing deterioration such as steel corrosion. The two categories of action interact, especially when the internal stresses are large enough to cause cracking of the deck concrete. In order to improve bridge deck durability, both structural and material aspects of the problem need to be addressed in a systematic fashion. These aspects are also related to the construction procedure of manufacturing the material and building the structural system. Further, these aspects must be addressed with adequate consideration to the environment to which the deck is to be subjected, including truck wheel loading, thermal effects, freeze-thaw cycles, salt, etc.

RC bridge decks provide a “roof” to the supporting superstructure and substructure. They protect these components of the bridge from being directly exposed to surface runoff, deicing salt, etc. Thus a durable deck often results in a durable bridge which highlights the importance of the deck. It also has been documented that a large percentage of renewal funds for bridges have been spent on bridge decks for patching, overlay, and replacement. Therefore, there is an eminent need for more durable concrete bridge decks. In Michigan, this need is more profound because of the numerous freeze-thaw cycles, large amount of deicing salt usage, and high volumes of heavy trucks.

The primary objective of this research is to identify the major parameters influencing RC bridge deck life in Michigan, particularly those related to concrete cracking. The second objective is to develop recommendations to modify these parameters that are within the control of the bridge designer, materials engineer, the contractor, and/or the maintenance engineer in order to maximize deck life.

In this research project, the following tasks were fulfilled.

Literature review. The review has covered issues of concrete material, structural design, construction procedure, and environmental conditions in relation to RC deck cracking. In addition, the effects of shrinkage and thermal stresses on deck cracking received special attention in the review. The results of this review are documented in Chapter 2 of this report.

Nationwide survey on RC deck cracking. The survey was issued to state departments of transportation around the country. The purpose of the survey were a) to understand the extent of RC deck cracking in the country; b) to gather information on latest research efforts in other states on this subject; and c) to understand the experiences of other states in reducing and minimizing RC deck cracking. The questionnaire and the compiled data are presented in Appendix A. The survey results are presented and analyzed in Chapter 3.

The field inspection of existing RC bridge decks that were replaced within the last 5 years and analysis of the inspection data. This task was to establish the extent of RC deck cracking in Michigan. Data was collected and analyzed to identify factors that may contribute to RC deck cracking. Twenty bridges with RC decks were randomly selected from the population of trunk line bridges. The deck inspection procedure, statistical analysis procedure, and the results are discussed in Chapter 4. The compiled raw data is presented in Appendix B.

Construction monitoring. This task was to observe typical RC deck placement and compare against the Michigan Department of Transportation (MDOT) - Standard Specifications for Construction. Five RC deck placement projects were monitored, including data collection and inspection during concrete placement. Ambient Environmental data, including temperature and humidity, was collected all through the placement of the entire deck. In addition, the concrete placement process was recorded for each truckload. The concrete truck's arrival, placement, finishing, curing compound placement, and wet burlap cover placement times were recorded. The raw time data gathered during construction monitoring is presented in Appendix C. This data helped the researchers to identify violations to the MDOT Standard Specifications for Construction. The discussion of this task is presented in Chapter 5.

Laboratory and Field Testing. This task was to further examine the concrete materials used for placement of decks. The tests were performed on concrete samples prepared from the five concrete deck placement projects. The field testing included measuring slump, concrete temperature, and air content of fresh concrete. In addition, the following tests were conducted in the laboratory to achieve the concrete properties: Compressive Strength, Modulus of Elasticity, Ultrasonic Pulse Velocity (UPV), Permeability, Rapid Chloride Penetration (RCPT), and Absorption tests. This task is summarized in Chapter 6.

Parameters Influencing Deck Cracking. This task was to integrate the data collected in the above tasks in order to establish the parameters that control or influence deck cracking. The structural parameters that establish the influence of girder and reinforcement restraints on the concrete deck leading the crack formation are derived and assessed. This task presented in Chapter 7 focused on the thermal and shrinkage loads as the two factors responsible for the volume change of hardened concrete deck, including analysis for typical bridges.

Summary and Conclusions. Chapter 8 of the report summarizes the findings of the research project and presents recommendations for possible follow-up steps.

Suggestions for further research are given in Chapter 9.

2 STATE-OF-THE-ART LITERATURE REVIEW

2.1 INTRODUCTION

Early-age bridge deck cracking is a problem throughout the United States. Many highway agencies studied and investigated the causes of bridge deck cracking. However, the problem of early-age bridge deck cracking has not been resolved. Many researchers divided the parameters that influence bridge deck cracking into four main categories:

1. Material related issues
2. Design related issues
3. Environmental or site conditions
4. Construction related issues

This chapter is organized in eight sections. Section 2.2 discusses the studies of material effects on early-age bridge deck cracking. Section 2.3 presents the previous studies regarding design related issues. In Section 2.4, the summary and main recommendations from previous studies regarding the environmental related issues are presented. The investigation of construction related issues with regard to deck cracking are explained in Section 2.5. Section 2.6 briefly discusses the effects of temperature and thermal stresses on early-age bridge deck cracking. Effects of shrinkage on deck cracking are explained in Section 2.7. Section 2.8 examines the crack formation phenomenon. Section 2.9 summarizes the major findings and remedial measures taken to minimize early-age bridge deck cracking.

2.2 MATERIAL RELATED ISSUES

Concrete bridge decks develop transverse cracking when longitudinal tensile stresses in the deck exceed the tensile strength of the concrete with sufficient accompanying strain levels. The tensile stresses are caused by the volume change of concrete due to shrinkage and thermal loads, and internal and external restraints. Therefore, many studies are focused on minimization of bridge deck cracking through modifications to construction materials and concrete mix designs.

Dakhil et al., (1975) reported that higher slump concrete and larger reinforcing bars tend to be found in decks observed to be more susceptible to cracking. In contrast, Cheng and Johnston (1985) observed a decrease in transverse cracking in bridge decks with increasing slump. They

also observed reduced cracking with increasing air content. The use of high slump concrete, an excessive amount of water resulting from inadequate mixture proportions, and retempering of concrete causes concrete cracking (Issa 1999).

Purvis et al., (1995) studied premature cracking of concrete bridge decks in three phases. The first phase was the examination of existing bridge decks. This phase included visual walk-by surveys of 111 bridge decks built in Pennsylvania. The bridges selected were built within 5 years of the survey and an in-depth survey was conducted on 12 of the 111 bridge decks. The in-depth surveys included the documentation of crack patterns, crack width, rebar location and depth, and finally concrete coring. The result of this phase indicated that almost all transverse cracks followed the line of the top transverse bars, regardless of the type of superstructure. Coring of concrete showed that transverse crack depth extended to the level of the top transverse bars and beyond. It was also observed that thicker cover depth caused wider cracks because the longitudinal crack control reinforcement is embedded more deeply into the concrete. Examination of concrete cores showed that transverse cracks often intersect coarse aggregate particles. This indicated that the cracks occurred in the hardened concrete as opposed to the plastic concrete. Therefore, the cause of cracking was most likely drying shrinkage and thermal shrinkage, rather than factors such as plastic shrinkage that is caused by surface evaporation prior to curing, or settlement of plastic concrete between the top transverse bars.

The second phase involved the observation of the construction of eight bridge decks to identify procedures contributing to shrinkage and possibly leading to cracking. The third phase involved using laboratory experiments that focused on examining the effects of aggregate, cement, and fly ash on shrinkage. The result of this phase indicated that the main cause of transverse cracking is the shrinkage of hardened concrete. Further, the type of aggregate used in the concrete mix is a major factor associated with shrinkage cracking. Aggregate contributes to drying shrinkage of concrete in two different ways. First, certain aggregates need more water in the mix to produce the desired slump and workability, and the extra water increases shrinkage. Secondly, certain aggregates yield to the pressure from the shrinking paste and do not provide sufficient restraint against shrinkage. Moreover, the study recommended using lower water content in the concrete mix in order to reduce drying shrinkage. Another important factor is the cement type. The study indicated that Type II cement has lower heat of hydration and less drying shrinkage than Type I

cement. It was indicated that the less cement used, the less heat generated, leading to less water being required for hydration.

NCHRP report 380 is the most comprehensive study performed to date on transverse bridge deck cracking. It presents the results of a study conducted by Krauss and Rogalla in 1996. The study surveyed all U.S. Departments of Transportation and several overseas transportation agencies. They performed analytical studies and laboratory research, and conducted field measurements of bridge structures during and shortly after deck construction.

Krauss and Rogalla (1996) ranked the concrete material properties and material-related mechanisms that lead to early-age bridge deck cracking as shown in Table 2-1. Also shown are the factors affecting cracking related to design and construction issues. The study indicated that concrete material factors that are important in reducing early-age cracking include low shrinkage, low modulus of elasticity, high creep, low heat of hydration, and selection of aggregates and concrete that provide a low cracking tendency. Other material factors that are helpful in reducing the risk of cracking include reduction of cement content, use of shrinkage compensating cement and avoidance of silica fume admixtures and other materials that produce very high values of early compressive strength and modulus of elasticity. Concrete with very high early strength and modulus of elasticity is prone to cracking because it creeps very little. It was also recommended that use of other cementitious materials having less drying shrinkage should be pursued. Air entrainment, water reducers, retarders, and accelerators have minimal effects on cracking.

Krauss and Rogalla (1996) utilized the restrained ring test with some modification to measure the tendency of concrete to undergo drying shrinkage cracking, as well as to compare various concrete mixtures, curing, and environmental factors. The major advantage of the restrained ring test is that it takes into consideration all material factors that influence shrinkage cracking from the time of casting. Furthermore, it does not require complex calculations and is simple to execute. The test procedure is as follows: A 3-inch thick concrete ring is cast around a steel ring instrumented with strain gages. The strain accumulation and the length of time before cracking occurs indicate the cracking tendency of concrete. Concrete that creates less strain on the steel ring and takes longer to crack has a lower cracking tendency. The test apparatus consists of a

steel ring with an outside diameter of 12 inches, a radial thickness of 3/4-inches, and a height of 6-inches, as shown in Figure 2-1. The outside form is made from thin 1/8-inch polyethylene sheeting. Polyethylene-coated plywood is used for the base to minimize the friction restraint of the concrete.

Krauss and Rogalla (1996) used the ring test to investigate the effects of many factors such as water to cement ratio, cement content, aggregate size and type, silica fume, set accelerators and retarders, air entrainment, cyclic temperature, evaporation rate, curing, and shrinkage compensating cement. Rings cast with Type K expansive cement cracked much later than the control mix. Moreover, the mixes containing silica fume cracked 5 to 6 days earlier than the companion mixes without silica fume content. The test also showed that a mix with 28 percent of the Portland cement replaced with a Type F fly ash cracked only slightly later (4.3 days) than the control specimens.

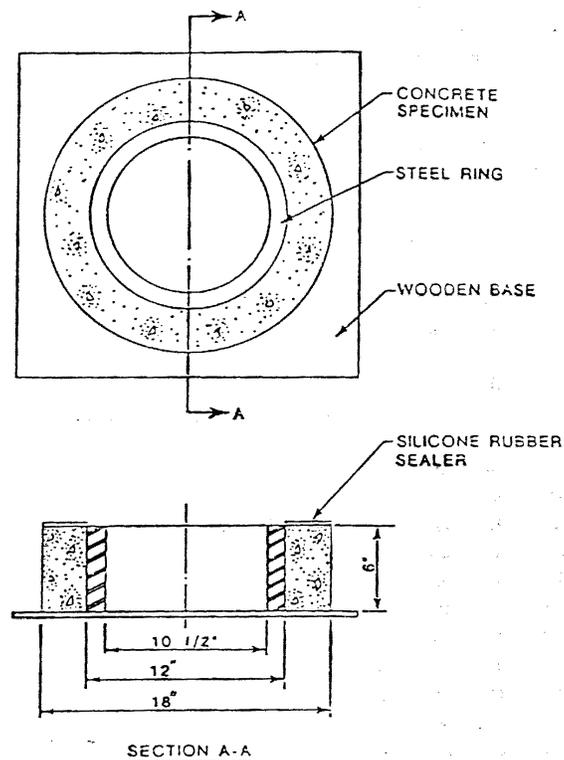


Figure 2-1. Cracking-tendency test apparatus reported by Krauss and Rogalla (1996)

Table 2-1. Factors Affecting Cracking (Krauss and Rogalla, 1996)

Factors	Effect			
	Major	Moderate	Minor	None
Design				
Restraint	✓			
Continuous/simple span		✓		
Deck thickness		✓		
Girder type		✓		
Girder size		✓		
Alignment of top and bottom reinforcement bars		✓		
Form type			✓	
Concrete cover			✓	
Girder spacing			✓	
Quantity of reinforcement			✓	
Reinforcement bar sizes			✓	
Dead-load deflections during casting			✓	
Stud spacing			✓	
Span length			✓	
Bar type—epoxy coated			✓	
Skew			✓	
Traffic volume				✓
Frequency of traffic-induced vibrations				✓
Materials				
Modulus of elasticity	✓			
Creep	✓			
Heat of hydration	✓			
Aggregate type	✓			
Cement content and type	✓			
Coefficient of thermal expansion		✓		
Paste volume—free shrinkage		✓		
Water-cement ratio		✓		
Shrinkage-compensating cement		✓		
Silica fume admixture		✓		
Early compressive strength			✓	
HRWRAs			✓	
Accelerating admixtures			✓	
Retarding admixtures			✓	
Aggregate size			✓	
Diffusivity			✓	
Poisson's ratio			✓	
Fly ash				✓
Air content				✓
Slump [†]				✓
Water content				✓
Construction				
Weather	✓			
Time of casting	✓			
Curing period and method		✓		
Finishing procedures		✓		
Vibration of fresh concrete			✓	
Pour length and sequence			✓	
Reinforcement ties				✓
Construction loads				✓
Traffic-induced vibrations				✓
Revolutions in concrete truck				✓

[†] within typical ranges

Babaei and Fouladgar (1997) recommended the use of a mix design with cement content as low as possible in order to control thermal shrinkage. When less cement content is used, less heat of hydration is generated. Pozzolans and slag can be used as partial substitutes for Portland cement. It was also recommended that Type II cement be used rather than Type I because Type II has a lower heat of hydration. Furthermore, as a means of controlling thermal shrinkage, use of retarders in the mix is recommended to delay the hydration process and reduce the rate of heat generated. The effect of aggregate type on drying shrinkage was also studied. Using a soft aggregate such as sandstone tends to result in increased drying shrinkage, while use of hard aggregates such as quartz, dolomite, and high limestone tends to result in decreased drying shrinkage. Also, use of less water in the mix in an effort to reduce the evaporation rate after curing could reduce the amount of drying shrinkage.

Michael et al. (1997) studied the use of shrinkage compensating concrete (SCC) or Type K cement to reduce early-age bridge cracking. The Ohio Turnpike Commission (OTC) has used SCC exclusively for its new and replaced bridge decks for past 12 years, and has been quite satisfied with its performance to date. The study indicated that use of SCC has greatly mitigated shrinkage cracking. From the construction point of view, using SCC requires stricter attention to construction procedures than does ordinary Portland cement concrete. For instance, SCC has a shorter workable period and therefore should be placed by pumping. In addition, curing requirements for SCC are more demanding.

Shah et al. (1998) observed that cracking has been shown to increase with the use of higher strength concrete, especially with the addition of silica fume. Randomly distributed fiber reinforcement can be used to reduce the time to the first visible cracking and can significantly reduce crack width. Different fiber compositions can alter the degree to which this occurs. It was also found that with a two percent addition of shrinkage reducing admixture (SRA) by weight of cement, drying shrinkage could be reduced by nearly 50 percent.

Similarly, French et al. (1999) studied 72 bridges in the Minneapolis/St. Paul metropolitan area. In this study, the dominant material parameters associated with transverse cracking in bridge decks were identified as cement content, aggregate type and quantity, and air content. Data obtained from material reports for 21 of the bridge deck mixes (12 prestressed and 9 steel girder

bridges) were used in this study. It was observed that the concrete mix with the highest cement content [846 lb/yd³] cracked first, while the mix with the lowest cement content [470 lb/yd³] cracked last. Comparison of 21 bridges on the basis of aggregate type and quantity showed that increased aggregate quantity could result in reduced cracking. The study also indicated that increased air content (>5.5 percent) could result in reduced cracking.

Burrows (1998) studied the effect of aggregate type and content used in concrete mixtures on the drying shrinkage of concrete. In this study, limestone was found to be one of the aggregates that exhibited the least drying shrinkage, while sandstone exhibited the most drying shrinkage. Table 2-2 shows the effect of the type of aggregate on drying shrinkage as recorded by Burrows (1998).

Table 2-2. Effect of Type of Aggregate on the Drying Shrinkage of Concrete

Aggregate type	One-year shrinkage (percent)
Sandstone	0.097
Basalt	0.068
Granite	0.063
Limestone	0.050
Quartz	0.040

2.2.1 Concrete Constituent Materials and Selection Criteria

Concrete properties and design should satisfy the requirements of specifications, expectations of construction engineers, and should be environmentally durable. Based on structural requirements, concrete strength can be designed, but strength is not the only parameter for achievement of a durable and sustainable structure. Severe environment conditions such as those in Michigan require more attention than regular environmental exposures. Since durability is the main concern for a concrete deck, selection of concrete constituent materials, construction practices, and construction intensity are essential factors to be considered. Selection of concrete constituent material is the most critical parameter to be examined. However, available durable materials may not result in a durable concrete design, regardless of practice (Shah et al. 1994).

In order to get maximum performance from all constituent materials, the performance of each individual ingredient of concrete should be known in terms of durability, mechanical and chemical properties, and interactions with other ingredients. However, it should be noted that complete certainty as to how ingredients will interact when combined in concrete mixture is not feasible. Particularly in production of durable concrete, any material incompatibilities will be highly detrimental to the concrete in service. Because of this, mix design optimization requires extensive testing of trial mixes (Shah et al. 1994).

In the European Concrete Standard for conclusive concrete performance, resistance to environmental exposure is given in terms of established concrete properties and composition limits. The local changes in environmental exposure may act differently on concrete. The requirements for each exposure class are specified in terms of permitted types and classes of constituent materials, maximum water/cement ratio, minimum cement content, minimum concrete compressive strength class (optional), and minimum air-content of the concrete. For instance, in a severe freeze-thaw environment where concrete is exposed to chemical salts, EN 206 requires at least 575 lb/yd³ cement, 5000-6500 psi concrete strength range, 0.4 water/cement ratio, and minimum 4% air entrainment. In addition, the maximum water/cement ratio is given in increments of 0.05, and the minimum cement content is given in increments of 33.7 lb/yd³. Recommendations for the choice of limiting values for concrete composition and properties are given separately, based on type of cement. If the service life of a structure is estimated to be more than 50 years under normal maintenance conditions, and severity of the exposure conditions is worse than predicted, specific concrete compositions or specific protection and maintenance procedures are specified (EN 206 2000).

Shah et al. (1994) state that a compressive strength of least 6000 psi should be considered to be high strength concrete. The use of such a high strength concrete requires extensive care and better understanding of material and design. For convenience, the materials used in concrete design are discussed separately in the following sections.

2.2.1.1 Portland Cement

Cementitious materials have different chemical compositions and properties, and most concrete properties depend on cementitious materials. Cementitious materials are the most active

ingredients of concrete constituent material. Generally, the cost of cementitious material is higher than that of other constituent materials. The selection of cementitious material is important to achieve the most economic and durable concrete design. Most cementitious materials provide sufficient strength and durability under normal environmental conditions. Under severe environmental conditions such as those found in Michigan, freeze-thaw, deicing salt, and alkali silicate reaction are factors that require special attention. Any selection failure may have serious consequences. It is usually satisfactory to select general-purpose cements that are available in the local market. General purpose cements are described in ASTM C 150 as Type I and Type II, in ASTM C 595 as Type IP and Type IS, and in ASTM C 1157 as Type GU. Since these cements are largely available and used in the market, their quality and performance under some effects are known. General characteristics of cements given in ASTM C 150 (standard specification for Portland cement) and ASTM C 595 (Standard specification for blended hydraulic cement) are given in Table 2-3 and Table 2-4 respectively (ACI 225R-99 2001).

Table 2-3. The Cement Types and Properties in ASTM C 150

Type	Description	Optional Characteristics	% of US Shipments
I	General use	Air entraining, low alkali	86.6
II	General use; moderate heat of hydration and moderate sulfate resistance	Air entraining, moderate heat of hydration, low alkali	-
III	High early strength	Air entraining, moderate or high sulfate resistance based on C ₃ A content, low alkali	3.3
IV	Low heat of hydration	Low alkali	Not available in USA
V	High sulfate resistance	Low alkali, sulfate resistant	

Cement chemistry has a great influence on the cement hydration process and therefore its mechanical properties. Portland cements are classified based on their chemical components. They are C₃S (alite), C₂S (belite), C₃A (aluminate), and C₄AF (ferrite). C₃S exists in the clinker, it as a complex structure and may form six or seven different crystal structures. C₂S also exists in clinker and has at least five crystal forms, unlike C₃S, the formation of crystals differs with

performance. C_3A can be found in different crystal forms in concrete and it hydrates faster than other clinker phases. C_4AF also has many different crystal forms but has a slow hydration rate. Because of this, it is often considered less important than C_3A (ACI 225R-99 2001).

Table 2-4. The Cement Types and Properties in ASTM C 595

Type	Name	Blended Ingredients Range		Optional Characteristics
		Pozzolan	Slag	
I (PM)	Pozzolan-modified Portland Cement	0 - 15	-	Air entraining, moderate sulfate resistance, moderate heat of hydration
IP	Portland Pozzolan Cement	15 - 40	-	Air entraining, moderate sulfate resistance, moderate heat of hydration, alkali reactivity resistance
P	Portland Pozzolan Cement	15-40	-	Air entraining, moderate sulfate resistance, low heat of hydration, alkali reactivity resistance
I (SM)	Slag modified Portland Cement	-	0-25	Air entraining, moderate sulfate resistance, moderate heat of hydration
IS	Portland Blast-furnace slag	-	25-70	Air entraining, moderate sulfate resistance, moderate heat of hydration, alkali reactivity resistance
S	Slag cement	-	70-100	Air entraining, moderate sulfate resistance, alkali reactivity resistance

The strength of cement basically depends on C_3S and C_2S content. Their percentages in clinker affect the heat of hydration and the appropriateness of additional cementitious admixtures to cement (ACI 225R-99 2001).

2.2.1.2 Shrinkage Compensating Cements

Shrinkage compensating cements are hydraulic cements that expand slightly during the early hardening period after setting. This expansion is used to compensate for the shrinkage of concrete. There are three different types of expansive cements based on their active ingredient types. They are referred to as K, M, and S. All types of expansive cements are manufactured to produce the proper amount of expansion without adverse effects.

Table 2-5. Expansive Type of Cements and their Constituents

Expansive Cement	Principal Constituents	Reactive Aluminates Available for Ettringite Formation
K	<ul style="list-style-type: none"> • Portland cement • Calcium sulfate • Portland like cement containing C_4A_3S 	C_4A_3S
M	<ul style="list-style-type: none"> • Portland cement • Calcium sulfate • Calcium-aluminate cement CA and $C_{12}A_7$ 	CA and $C_{12}A_7$
S	<ul style="list-style-type: none"> • Portland cement high in C_3A • Calcium sulfate 	C_3A

The effect of use of chemical admixtures with expansive cements is not clearly presented to date. The cement chemistry directly affects the performance of admixtures. Trial batches are required to be issued with job materials and kept under ambient environmental conditions. Mixing time, transportation period, and reaction of admixtures should be controlled during the trial tests. Results may vary even if the laboratory tests are satisfactory. Air entrainment agents are effective, but some water reducers and retarding agents may be incompatible. Existence of $CaCl_2$ in concrete may affect the expansion rate and increase shrinkage. Supplementary cementitious materials (fly ash and other pozzolans) may reduce expansion and other physical properties of concrete made with expansive cements.

Although expansive cements require more water than Portland cements, compressive strength is comparable. Other physical properties of concrete made with expansive cements, such as modulus of elasticity and creep, are identical to ordinary Portland cement (ACI 223-98 2001).

Restraint of expansive cement is important in construction of concrete decks. Restraint due to girders and adjacent structural elements is indeterminate and may cause either too much or too little restraint. High restraint will induce high compressive strength in concrete at an early age but provide less shrinkage compensation (ACI 223-98 2001).

2.2.1.3 *Supplementary Cementitious Materials*

Supplementary cementitious materials, also called mineral admixtures, contribute to the properties of hardened concrete through hydraulic or pozzolanic activity. Typical examples are natural pozzolans, fly ash, ground granulated blast-furnace slag, and silica fume. Supplementary materials can be used individually with Portland cement or blended cement, or in different combinations. These materials are often added to make concrete mixtures more economical, reduce permeability, increase strength, or influence other concrete properties (PCA@ 2003). Each of these supplementary materials has specific properties to be considered before mixture properties are ascertained. These supplementary materials are discussed separately in subsequent sections (ACI 211.1-97 2001).

2.2.1.3.1 *Fly Ash*

Fly ash is mainly a by-product of burning coal in power plants. The coarse burned coals are crushed and ground to a fineness of 75 μm . ASTM C 618 classifies fly ash into two main groups based on its chemical composition in terms of iron oxide (Fe_2O_3), aluminum oxide (Al_2O_3), and silicon dioxide (SiO_2). If the summation of SiO_2 (35% – 60 %), Al_2O_3 (10 % - 30 %), and Fe_2O_3 (4% - 20%) is greater than 70%, fly ash is categorized as Class F. If the summation is above 50%, it is classified as Class C. Generally Class C fly ash has more than 20% CaO (1-35%) (ACI 232.2R-96 2001).

The performance of Class C and Class F fly ash shows different characteristics; however the performance of fly ash is not directly based on its classification. The general approach for evaluation of fly ash is to evaluate its performance with regard to increasing the resistance of concrete to alkali silicate reaction and sulfate attack (ACI 232.2R-96 2001).

The shape, glass formation, particle size distribution, and density of fly ash are very important factors when hydration of cementitious material and hardened concrete properties are considered. Fly ash contains small glassy spheres because the burned coal cools rapidly. The composition of the glassy spheres depends on the composition of coal and the temperature at which coal is burned. Since the coal sources and technologies at each power plant are different, properties of fly ash (specific density, particle size and shape) also differ. The use of fly ash is based on a

given cement and its chemical composition. The chemical composition of fly ash varies more than other supplementary materials even if the source of fly ash is the same.

Fly ash addition generally increases the paste volume as well as the drying shrinkage. If the water added to the mixture is kept constant, fly ash has no effect on drying shrinkage. The amount of fly ash added to the mixture is important. When the amount of fly ash added to mixture is in a range of 0-20 % of cement mass, the mixture has the same or slightly less shrinkage than ordinary Portland cement concrete. According to ACI 232 (2001), Malhotra states that if the amount of fly ash added to concrete design is more than 40% of the mass of cement, the deicing scaling performance of such concrete is suspicious (ACI 232.2R-96 2001).

The effects of fly ash addition on hardened concrete are a reduction in heat of hydration, lowered permeability, and improved resistance to sulfate and chemicals. Curing practice is important when attempting to gain permeability and chemical resistance properties. Poor curing can result in unsatisfactory performance (Day 1999).

2.2.1.3.2 Ground Granulated Blast-Furnace Slag (GGBS)

A by-product of the iron working industry, GGBS is a blast-furnace slag that has been dried and ground to fine powder. The slag used in concrete production is specific for its hydraulic reaction. This reaction is related to rapid cooling of slag in steel production plants. This rapid cooling process creates glassy forms. Slow cooled slags are predominantly crystalline and do not possess significant cementitious properties. The principal hydration product of slag is essentially the same product formed when Portland cement hydrates. Although the hydration of slag is slower than that of ordinary Portland cement, the hydration of slag is more gel-like than the ordinary Portland cement. Hydration of slag adds denseness to the cement paste. The hydration of Portland cement, alkali salts, or lime increases the hydration rate of slag. The hydration of slag depends largely on the breakdown and dissolution of the glassy slag structure.

In general, hydration of slag in combination with Portland cement at normal temperatures is a two-stage process. When cement, slag, and water are mixed, slag reacts with the calcium hydroxide available in the mixture. This is referred to as initial hydration. After the cement begins hydration, the slag reacts with calcium hydroxide, which is a by-product of cement hydration. This is the second stage of slag reaction. The calorimetric studies of rate of heat

generation prove this two-stage process. If the early stage of slag hydration requires additional alkali salts, calcium hydroxide, or sulfates, the reaction of slag itself with water is negligible.

Elevated temperatures during the hydration increase the solubility of alkali hydroxides from cement, therefore increasing the reactions of slag. Slag is able to bind more alkalis than ordinary Portland cements due to its lower calcium silica ratio. Alkali hydroxide alone can hydrate slag to form a strong cement paste structure that may be used in special applications.

The usage of slag is predominantly based on the chemical composition of slag, alkali concentration of the reacting system, glass content of slag, fineness of slag and cement, and ambient temperature during early phases of hydration. Since the hydration of slag is influenced by so many complex factors, earlier attempts failed to provide adequate evaluation criteria for use in practice. The ASTM C 989 slag activity index is often suggested as a basic criterion for usage and evaluation of slag.

The proportion of slag is generally 25 –70 % of total cementitious materials. Recent research has shown that optimum slag usage is 50 % of cementitious material. Use of a large amount of slag makes proper curing practice more critical. Since the density of slag is lower than that of ordinary Portland cement, there is a need to change the solid content of concrete design. This change is often made to coarse aggregate content in order to reduce water demand and shrinkage (ACI 233R-03 2003).

Similar to fly ash, most of the good properties of slag concrete are related to curing practices. Poor curing of slag concrete can dramatically reduce most of its predicted properties (Day 1999).

With the exception of setting time, concrete properties of slag mixtures are almost identical to ordinary cement mixtures. The setting time of a slag mixture depends on environmental temperature. Paste volume and aggregate content of the mixture should be controlled. It should be noted that regardless of the concrete ingredients, the cement or supplementary materials must be kept moist during early hydration (ACI 233R-03 2003).

2.2.1.3.3 *Silica Fume*

Silica fume is a by-product of the production of silicon metal or ferrosilicon alloys. It contains a high content of amorphous silicon dioxide and some other very fine particles. Silica fume should not be used in concrete unless data with regard to the use of micro silica shows favorable performance (ACI 234R-96 2001).

Different types of silica fume can be found in the market. Examples are blended silica fume cements, as-produced silica fume, slurried silica fume, densified (condensed) silica fume, and pelletized silica fume. As-produced silica is difficult to handle as compared with cement, slag or fly ash. The flow of micro silica creates some problems with pneumatic systems in cement plants. Of primary concern, micro silica inhalation may create health hazards. Slurried silica is developed to overcome handling difficulties of as-produced silica fume. The slurried micro silica contains 40-60 % of micro silica by mass depending on the source. Some manufacturers include a superplasticizer that is capable of controlling silica fume concrete properties (ACI 234R-96 2001).

Micro silica has ultra fine smooth spherical glassy particles with a 97,634 ft²/lb specific surface compared to normal cement fineness of 1465-1953 ft²/lb. The fineness of silica fume is measured by using different methods, but results vary due to the extreme fineness and carbonic composition of silica fume (ACI 234R-96 2001).

Properties of micro silica differ from source to source, and the published data is very limited. The variation of silica fume quality can be compared to that of slag quality, but doesn't vary as much as fly ash quality. This is due to high quality control measures in the metallurgical industry (ACI 234R-96 2001).

The use of silica fume in concrete increases the mixing water demand. High range water reducers can control the increasing water demand and solve this problem. If appropriate precautions are taken, silica fume does not affect the workability and slump significantly. When water-reducing admixtures are used, setting time may be affected. Segregation and bleeding may be reduced. Use of silica fume significantly reduces early-age drying shrinkage (ACI 234R-96 2001).

The effects of silica fume on fresh and hardened concrete properties vary with the amount of micro silica used in the mixture. Discussion of the effects of silica fume addition is generally categorized with regard to percentage added, either more or less than 10%. As a general concept, addition of micro silica in higher percentages affects the water demand, drying shrinkage, and time of setting. Silica fume has finer particles than cement and other cementitious materials. Those finer particles fill the space between the relatively coarser cement particles and help the concrete flow. Excessive addition of silica fume, more than 20%, makes concrete sticky. However, the filling effect of silica fume is useful with regard to pumping and placement of concrete. If the addition is less than 10%, the effect of silica fume on setting time and workability is negligible. The effect of silica fume addition on drying shrinkage can be protected by common curing procedures (Swamy et al. 1986).

The effect of silica fume addition to hardened concrete is also important. Since the finer particles of silica fume fill the spaces between coarser cement particles, bleeding and segregation of concrete drastically reduces. This reduction affects the drying shrinkage based on silica fume percentage. The finer structure of silica fume concrete results in changes to the pore structure of concrete. This change increases the resistance of concrete to aggressive chemicals, freeze-thaw, and abrasion, and reduces the permeability of concrete (Swamy et al. 1986). The use of silica fume regulates the pore structure by decreasing the number of large pores and distributing them homogeneously. Total porosity, however, will not be affected significantly. The available micro cracks and holes in the transition zone between cement paste and aggregate will be healed. These positive effects reduce the permeability of concrete (ACI 234R-96 2001).

The use of silica fume increases the strength of concrete. Addition of silica fume in amounts less than 5% does not increase the early-age strength, but it may increase the 7 and 28-day strengths as much as 10% and 20% respectively (Swamy et al. 1986). For high strength concrete, silica fume may be added in amounts as much as 20%. However, other consequences should be taken into account (Shah 1999).

Shah et al. (1994) examined the fracture mechanics of high strength concrete and the effects of concrete composition on fracture. An increase in the addition of silica fume increases the crack resistance of concrete; correspondingly, an increase in silica content increases the brittleness of

concrete. Concrete composition can be expressed as aggregate, cement paste and aggregate-mortar interface. Since high performance concrete has higher paste strength than that of normal strength concrete, its resistance to cracking is not related to aggregate size. Angularity of aggregate is found to be more critical than aggregate size when predicting crack resistance of high strength concrete. Also, an increase in water/cement ratio and air content adversely affect the crack resistance of concrete. Pores that are usually present in concrete have a large effect on cracking of concrete. Precautions that regulate the pore structure increase the tensile resistance, and therefore enhance the crack resistance of concrete. Use of silica fume reduces concrete cracking potential (ACI 234R-96 2001).

2.2.1.4 Ternary Systems

The use of supplementary materials in combination with Portland cement is not uncommon, especially in high performance concrete production. The use of Portland cement as the third cementitious material is referred to as a ternary blend or ternary system. Slag, micro silica, and fly ash are the most commonly used materials in ternary systems. Most of the durability problems that arise with the use of ordinary Portland cement can be treated by using ternary systems. In the meantime, Malhotra reported that these systems may provide similar properties to those of ordinary cements with the exception of resistance to a freeze-thaw environment. A freeze-thaw environment requires a minimum of 340 lb/yd³ of Portland cement and a low water/cement ratio (ACI 223-98 2001).

Haque (1996) examined the curing regime of some common ternary blends that contained 70-75% Portland cement, 5-10% micro silica, and 20% fly ash or slag, for 3, 7 and 28 days of moist curing. After samples were moist cured, they were kept under $40 \pm 5\%$ relative humidity. The mechanical properties (strength, modulus of elasticity, water penetration and drying shrinkage) and effects of curing on those specimens were tested. It was found that addition of 10% condensed micro silica improved the strength and other mechanical properties of concrete, but addition of fly ash was suspicious with regard to performance. Addition of condensed micro silica also reduced the consequences of short curing periods. The addition of condensed micro silica reduced the drying shrinkage; further addition of fly ash and slag, however, increased the shrinkage of concrete.

Recent research made on the durability characteristics of high performance concrete with a ternary mixture emphasizes that the curing of high strength concrete is unique and different from that of normal strength concrete. The autogenous shrinkage of ternary systems is found to be homogenous through the cross section. The autogenous shrinkage is more critical than drying shrinkage for ternary systems. Drying shrinkage is found to be effective at the surface of concrete members. Since permeability of high strength concrete is lower than that of normal strength concrete, the curing regime in Figure 2-2 is found to be critical. The curing of high strength concrete during the first 12-36 hour period following placement is very critical. After that, the pore structure of concrete will have developed and lower permeability will result in self-desiccation of the concrete member. Furthermore, the first 2-3 days of curing is also essential for high strength concrete (Aitcin 2003).

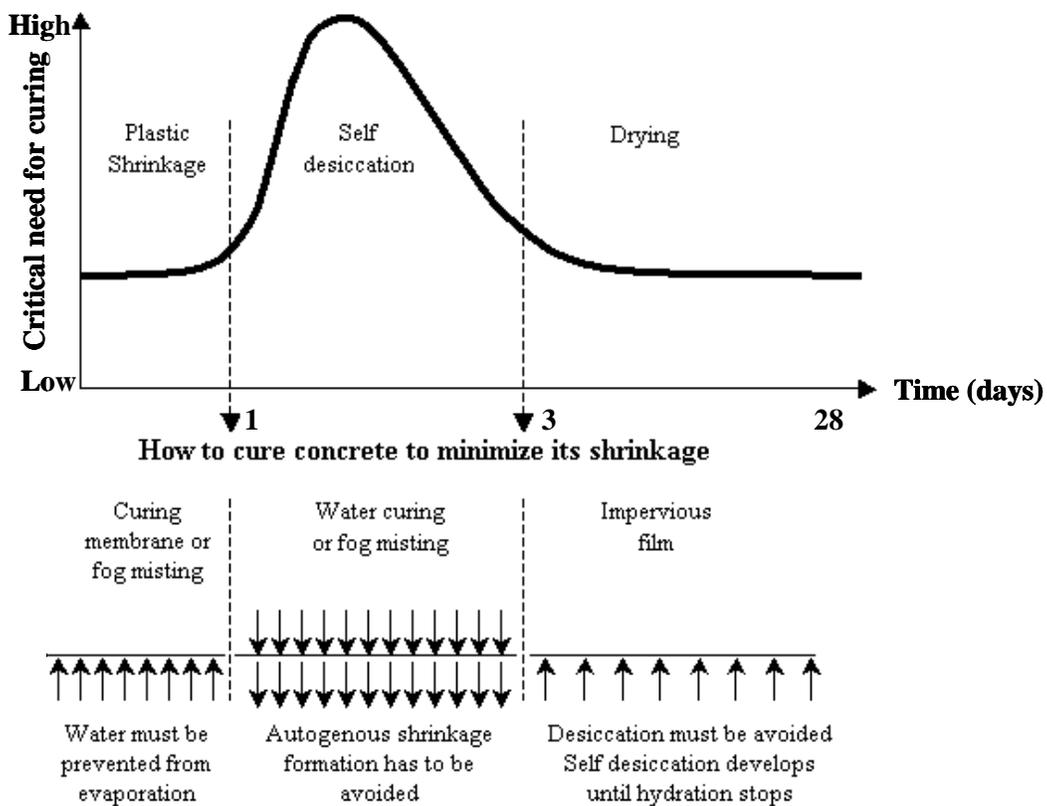


Figure 2-2. The most appropriate curing regimes during the course of the hydration reaction

2.2.1.5 Effects of Chemical Admixtures on Concrete

Chemical admixtures are defined as materials used in concrete other than water, aggregates, hydraulic cement, and fiber reinforcement. They are added to concrete or mortar immediately before or during mixing.

Since the strength and other properties of concrete are controlled by the cement content and water/cement ratio, chemical admixtures are often used to reduce the water/cement ratio, and improve concrete properties. Chemical admixtures should conform to ASTM C 494 requirements. The types of chemical admixtures in ASTM C 494 are listed in Table 2-6.

Table 2-6. Types of Admixtures in ASTM C 494 and Definitions

Type of Admixture	Definition
Type A	Water Reducing
Type B	Retarding
Type C	Accelerating
Type D	Water reducing and retarding
Type E	Water reducing and accelerating
Type F	Water reducing, high range
Type G	Water reducing, high range and retarding

When used in the recommended dosage rates Types A, D, and E admixtures ordinarily reduce the water demand of a mixture by 5% - 8 %, while Types F and G reduce water demand by 12% - 25 %. Types F and G are often called superplasticizers.

The manufacturers' recommendations and literature should be considered when determining the required chemical admixture dosage for each design. Chemical admixtures have some adverse effects when used in high dosages. Examples are excessive retardation (uncontrolled setting) and increased air entrainment. Also the total chloride in an admixture should be accounted for in order to ensure durability. Types A, B, and D are generally recommended to be used in small dosage (2 to 7 oz/100 lb. cementitious material) in concrete design. Types C, E, F and G are generally recommended to be used in large quantities (10 to 90 oz/100 lb. cementitious material),

and their water content should be included when calculating total water content of the concrete mixture. Sometimes there is a need to use a combination of admixtures in the same mix design. The total amount of combined chemical admixture should be taken into account when calculating the water content of the concrete mixture.

Before using the admixtures manufacturer's recommendations, ASTM C 494, ACI 318 and ACI 301 should be considered.

2.2.1.5.1 Air Entraining Admixtures

Air entraining admixtures create an air void system that can protect the concrete from freeze and thaw exposure. The amount of air and void spacing are essential for air entrainment applications. The nature and content of the air entrainment admixture, concrete constituent materials, other chemical admixtures, employed mixing time, slump, and degree of consolidation are key factors that must be taken into account to ensure achievement of the projected air content (ACI 212.3R-91 2001).

Interactions between air entrainment and other water reducing and/or shrinkage reducing admixtures should be controlled. The effect of decreasing water content may increase the spacing between voids, or employing shrinkage reducing admixture could reduce the efficiency of an air entrainment agent (ACI 212.3R-91 2001).

The effect of increased air on fresh concrete is beneficial for reduction of water/cementitious materials ratio at a constant workability, since increased air content increases the workability of concrete. The bleeding and segregation also decreases when increasing the amount of air entrainment. Contrarily, increasing cement content may create some problems with air entrainment (ACI 212.3R-91 2001).

The effect of increased air on hardened concrete is a general reduction in the strength in concrete with moderate to high cement contents, in spite of decreased water content. The loss of strength is proportional to amount of air entrainment, but the rate of reduction increases with higher amounts of air entrainment (ACI 212.3R-91 2001). On the other hand, flexural strength of concrete does not show any significant change when increasing air entrainment up to 6%.

Not every chemical that is capable of producing air bubbles in cementitious material can be considered to be an air-entraining admixture. Furthermore, an air-entraining admixture should be capable of developing a sufficient air void system. Wood resins, synthetic detergents, sulfonated lignin salts, petroleum acid salts, organic sulfonated hydrocarbons, fatty and resinous acids, and their salts can be classified as agents that are able to develop a sufficient air void system in cementitious materials (ACI 212.3R-91 2001). A sufficient air void system makes the concrete more curing tolerable (Rixom et al. 1999).

An air content of 6% may not be sufficient when durability is considered. The amount of air entrainment should be controlled by the size of air voids and spacing. Research performed by the Virginia DOT showed that an increase in the amount of cementitious material may require control of the amount and distribution of air entrainment (Ozyildirim et al. 2003).

2.2.1.5.2 Water Reducing and Set Controlling Admixtures

Water reducing admixtures are a group of products that are added to concrete to achieve certain workability at a lower water/cement ratio than that of the control concrete. The water reducing admixtures are produced using one of the following materials: lignosulfonic acids, hydroxylated acids and salts, derivations of lignosulfonic acids and hydroxylated acids, sulfonated melamine product salts, high molecular weight condensation of naphthalene sulfonic acids, blends of naphthalene or melamine condensates with other water reducing or set controlling agents, and other inorganic materials such as amines, sugar acids, polymeric compounds, melamine and naphthalene derivations, silicones, and sulfonated hydrocarbons.

The use of set controlling or other chemical admixtures has no effect on the heat of hydration under the same adiabatic temperature. The occurrence of peak heat of hydration may be regulated (accelerated or delayed) but cannot be reduced with the use of the same type and amount of cement.

Water reducers have an impact on fresh as well as hardened concrete. Fresh concrete requires a lesser water/cement ratio than concrete without water reducers at the same workability. Because of this, the final strength of concrete using water reducers in their mix designs is higher. With improved concrete strength, other mechanical properties of concrete are similarly improved. Water-reducing admixtures can provide a certain level of air entrainment. The interaction

between air-entrainment agents and water-reducing admixtures should be controlled. The effect of water reducers on shrinkage shows conflicting data. As a general concept, use of water reducing admixtures increases the early-age shrinkage but decreases the long-term shrinkage.

2.2.1.5.3 Shrinkage Reducing Admixtures

Hydrated cement paste shrinks as it loses moisture from its extremely small pores. As the moisture is lost in these pores, the surface tension of the remaining water tends to pull the pore walls together and results in loss of volume over time (ConcreteNetwork@ 2003). Shrinkage reducing admixtures are designed to decrease the effects of drying shrinkage by reducing the surface tension in the pores. The amount of shrinkage depends mainly on water/cementitious material ratio. It is also affected by parameters that affect pore size distribution such as cementitious material type and fineness, and chemical admixtures. Some glycol ether blends have a tendency to reduce surface tension. This type of chemical admixture is called a shrinkage reducing admixture, which is available in the market.

Shrinkage reducing admixtures are added to mixing water in amounts of 1-2.5% of the mass of cementitious materials. The optimum range is found to be around 1.5%. The effect of shrinkage reducing admixtures on fresh concrete is negligible.

As a general concept, use of shrinkage reducing admixtures with lower water/cementitious material ratios result in a greater percentage of shrinkage reduction. Concrete with water/cementitious material ratio of 0.6 or less resulted in 80% or more shrinkage reduction at 28 days and 70% or more shrinkage reduction at 56 days. Concrete with a higher water/cementitious material ratio had a level of shrinkage reduction at 28 and 56 days of 37% and 36% respectively. In addition to the shrinkage reduction effect, shrinkage-reducing admixtures help to reduce thermal cracking by retarding the hydration process of concrete.

The main effect of a shrinkage-reducing admixture is on compressive strength of concrete. Addition of approximately 2% of shrinkage reducing admixture can reduce the 28-day strength as much as 15%. This strength reduction is observed with increasing water/cementitious material ratio. Therefore, it is recommended that shrinkage-reducing admixtures be used with superplasticizers (Rixom et al. 1999).

2.2.1.6 Aggregates

Aggregates are a major constituent of concrete volume, and they influence the properties and performance of concrete. When an aggregate performs below the expected level, the resultant concrete will be unsatisfactory and therefore, the service life of the structure is shortened (ACI 221R-96 2001).

The aggregate should be durable enough to sustain the properties of concrete under all exposure conditions. For this reason, durability and strength of aggregates are quite important. Freeze-thaw resistance, strength, shrinkage, thermal properties, and surface properties should be considered when selecting aggregates. The effect of aggregates on fresh concrete properties such as slump and workability, pumpability, and bleeding also contribute to concrete durability and should be taken into consideration during selection (ACI 221R-96 2001).

Important parameters of coarse aggregates in high strength concrete are their shape, texture, and maximum aggregate size. In the case of normal strength concrete, aggregate is generally stronger than paste. The strength of the aggregate is not a major factor in early ages of concrete. However, as the concrete strength rises, the aggregate contributes a great deal to strength and other properties of concrete (Shah et al. 1994).

2.2.1.7 Other State DOTs' Practice

By use of their websites, an evaluation of other state highway agencies with similar climatic exposure to that of Michigan was performed. Deck concrete strength, maximum aggregate size, water/cement ratio, cementitious material content, air content and types of chemical admixtures were extracted from specifications posted on the sites (Table 2-7).

Most of the states prefer the use of coarse to total aggregate ratio or coarse to total weight ratio rather than volumetric ratios. Water/cement ratio is nearly constant at 0.4. Total cementitious material content varies from state to state. This is due to regional practices and durability considerations. Wisconsin is the only state that uses Type I cement mix in deck concrete. The other State DOTs prefer blended mixtures. Generally, a blending operation is performed using Type I cement. The variations in slag and fly ash usage can be expressed with regard to regional source characteristics and construction practices. Air entrainment is about 6 % but some lower

(4.5%, Wisconsin) and higher (9%, Ohio) percentages are also observed. The type of admixture is important in order to satisfy water/cement ratio and workability requirements. Ohio and Illinois DOTs require a high range water reducer because their workability need is very high, around 8 inches at the construction site. The retarding effect is required for hot weather conditions.

Table 2-7. The Deck Concrete Design in other States with Similar Exposure to Michigan

DOT	Strength (psi)	D _{max} (in)	W/C	Cementitious material content (lb)	Type of cementitious material				Air %	Admixture
					Cement %	Fly ash %	GGBS %	Silica fume %		
Wisconsin Class D	4000	1	0.4 – 0.47	610	Type I	-	-	-	6±1.5	Water reducer
Minnesota			0.4	611	Type I	20	-	5	6.5	Water reducer
Illinois Class BD	4000*	1.5-1	0.44	607	Type I	Type F 15 or Type C 20	-	-	5-8	High Range water reducer
					Blended					
Ohio Class HP 3 or 4	4000 >	1-3/4	0.4	660	Type I	Type C 23		5	7±2	High range water reducer
				660	Type I		29	5		
New York Class HP	4000	1.5	0.4 – 0.44	682	Type SF, Type IP, Type SG				Av. 6.5 (5-8)	Water reducer
					Type I	20	6			

* 14-day strength

2.2.1.8 Concrete Design

Concrete is made up of aggregates, a hydraulic binder, and water and may contain other cementitious materials and/or chemical admixtures. Chemical admixtures are used to accelerate, retard, or improve consistency, reduce water demand, and increase compressive strength or alter other properties of concrete as explained in ACI 212.3R-96 (2001). Depending on the type and the amount of supplementary cementitious materials, it may be possible to enhance the specific

properties of fresh and hardened concrete. Examples of properties that can be changed are reduced heat of hydration, improved late age strength, reduced permeability, and increased resistance to alkali aggregate reaction, sulfate attack or abrasion. Reduced heat of hydration and reduced early-age strength reduces deck cracking.

The selection of concrete mix proportions requires an optimization of economy and other characteristics such as placeability, strength, durability, density and appearance. The selected materials as well as the technique used for placement govern the required characteristics. These concrete characteristics should be stated and explained in the specifications.

Concrete designs should be subjected to trial batches for purposes of revision. The trial batches should include a field trial if possible. Observations should be made for characteristics that may suggest potential changes in the mix design. Any change in the mixture proportions, chemical admixtures and/or materials should be verified by additional trial batches.

Concrete proportions are determined based on placeability, density, strength and durability requirements for a particular application. Placeability of concrete is of major concern. This term can roughly be compared to consistency and workability of concrete. Workability of concrete can be expressed as a property of concrete that determines its capacity to be placed and consolidated properly, as well as to be finished without segregation. Workability embodies such concepts as moldability, cohesiveness, and compactability. The grading of aggregate, maximum size and shape of aggregate, amount of cementitious materials, chemical admixtures, and entrained air affect workability. Each of these factors is important and should be considered in order to achieve satisfactory placeability of concrete.

Consistency can be explained as the mobility of concrete. It is measured in terms of the slump of fresh concrete. The ease of concrete flow during placement is affected by the consistency. The aggregate size, angularity and texture, available air entrainment, and chemical admixtures affect the amount of water demand in properly portioned concrete (ACI 211.1-97 2001).

Strength is a main characteristic of concrete, but other characteristics such as durability and permeability should be considered. Generally, the 28-day strength of concrete is used for structural design, proportioning, and evaluation.

For given materials and conditions, concrete strength is determined by the cementitious material and water content. The actual water content that should be considered in concrete design is the water content after deducting any water absorbed by aggregates. In addition, water demand may vary due to size, grading, shape and stiffness of the aggregate. Some cement types, air content of mix design, and chemical admixtures also affect the water demand. These effects are predictable and should be considered during the concrete mix design process (ACI 211.1-97 2001).

Concrete should be capable of enduring various exposures and service loads during its service life. Freeze-thaw, wetting-drying cycles, chemical materials, and deicing salts are included among the potential exposures. Some of these exposures require specific cementitious material properties that ordinary Portland cement cannot provide. Using low water/cementitious material ratio prolongs the life of concrete by reducing the penetration of water and aggressive chemicals into concrete. Severity of exposure should be considered during concrete design, such as the case of Michigan's environment (ACI 211.1-97 2001).

The hydration of cement is an exothermic chemical process that generates about 150 to 350 joule/gram, depending on cement type. This generated heat has an important impact on concrete when it is fresh. Heat of hydration is roughly proportional to amount, fineness, and C_3A content of cement. A lower water/cement ratio generates decreased heat and increased tensile strength. Addition of silica fume increases the generation of heat and the heat of hydration impact. For members that have a thickness less than 1.6 ft, the effect of this temperature rise may be ignored. For a bridge deck, which is highly sensitive to cracking, a mix design may be utilized to provide a relatively low heat of hydration (Mehta et al. 1994)

2.2.1.8.1 Concrete Design (ACI 211.1-97 2001)

Estimating the required mix design values for concrete involves a sequence of logical and straightforward steps that fit the characteristics of regionally available materials. Some exposure conditions and specifications may require some of the following items: maximum water/cementitious material ratio, minimum cement content, a certain air content, workability (slump), maximum size of aggregate, strength (at 28-day or other specific early ages), etc. The ACI 211.1 (2001) procedure is very straightforward and follows some simple steps as discussed below.

Step 1 consists of the determination of workability. The workability should be selected based on placement system, use of admixtures, and type of structure. For deck and pavement construction, slump is specified as 1-3 inches.

Step 2 involves choosing the maximum aggregate size. The larger the aggregate size, the lower the water demand, therefore, choice of a larger aggregate is more economical. However, selection of aggregate size is also related to reinforcing steel, workability, and consolidation technique.

Step 3 estimates the mixing water and air contents. For normal strength concrete, workability is related to nominal aggregate size, particle shape, grading, concrete temperature, air content and use of chemical admixture. ACI provides a summary table depending on the air entrainment and maximum aggregate size. The values given in the table is for concrete without admixtures. They give a good estimation for first trial batches.

Step 4 involves the selection of water/cementitious material ratio. Water/cementitious material ratio is not only related to strength but also to durability. Lower water/cementitious material ratio is desirable for better results with regard to strength and permeability.

The proportion of materials is based on an absolute volume equivalency technique. When water and water/cement ratio are defined, cement content is calculated. The volume of water, cementitious material and entrained air are deducted from a unit volume. The proportion of coarse and fine aggregates is determined by their gradation (ACI 211.1-97 2001).

2.3 DESIGN RELATED ISSUES

Dakhil et al. (1975) studied the influence of concrete cover, diameter of rebars, and slump on crack formation in plastic concrete. They reported that crack occurrence tends to decrease with increasing concrete cover over the reinforcement. Concrete slump and reinforcement bar size also influence cracking, but to a considerably lesser degree than concrete cover. Higher slump concrete and larger reinforcing bars tend to be found in structures more susceptible to cracking.

Krauss and Rogalla (1996) listed various design factors and ranked them according to their influence on cracking, as shown in Table 2-1. These design factors are restraint, bridge type

(continuous/ simple span), girder type (e.g., steel, concrete, composite, etc.), girder size and spacing, form type, skew, concrete cover, and other factors. It was found that cracking is more common among steel girder structures. Multi span continuous composite large steel girder bridges are most susceptible to cracking because of additional restraint. Stay-in-place deck forms were also found to have influences on deck cracking. Concrete cover over reinforcement is an important factor affecting cracking. Cover thickness between 1.5 inches and 3 inches was recommended.

Ramey et al. (1997) recommended 14 specific actions to be considered by structural designers to mitigate concrete cracking and in turn enhance bridge durability. These actions cover the size of deck reinforcement, rebar arrangements, concrete cover, deck thickness, w/c ratio, cement type, and deck construction sequence.

Similarly, French et al. (1999) reported that bridges with simply supported prestressed girders were observed to be in better condition than continuous steel girder bridges. For steel girder bridges, end restraint and shrinkage were the most important factors contributing to extensive deck cracking. The cracking in steel girder bridges increases with curved bridges, and on interior spans as compared to end spans.

Schmitt and Dawin (1999) indicated that cracking in bridge decks is generally believed to be caused by settlement adjacent to reinforcing bars and volume change.

Insufficient top reinforcement cover, improper placement of reinforcement, and insufficient deck thickness cause concrete cracking (Issa 1999).

2.4 ENVIRONMENTAL OR SITE CONDITIONS

Many researchers studied the effects of environmental conditions at the time of casting on the properties of fresh and hardened concrete. These environmental conditions are air temperature, relative humidity, and wind velocity. Elevated temperature and low relative humidity, or a combination of the two accelerates plastic shrinkage of concrete. If plastic shrinkage is restrained, cracks are formed. Plastic shrinkage cracks appear when the surface evaporation rates exceed the rate at which bleeding water rises to the concrete surface. Several investigators

and transportation departments believe that the effects of environmental conditions during concrete placement are the most significant factors affecting deck cracking.

Babaei and Hawkins (1987) observed more cracking in concrete cast during low humidity and with high evaporation rates. Transportation agencies reported that casting at night can significantly reduce deck cracking.

Purvis et al. (1995) recommended the use of retarders to reduce a rise in the temperature of concrete when ambient temperature is expected to reach 75°F or more. In hot weather, attempts should be made to cover concrete with wet burlap no more than 30 minutes after the surface is finished and textured; the burlap should be kept wet continuously. The study also recommended placement of concrete at night in hot weather to minimize heat that builds up from cement hydration, as well as ambient heat.

Krauss and Rogalla (1996) indicated that the evaporation rate should be measured at the job site, and wind breaks, fogging and evaporation retarder films should be used during periods of high evaporation, and during cold or hot weather. Moreover, concrete bridge decks should be placed during early or mid-evening whenever possible. Doing so reduces hydration temperature and the resulting thermal stresses, early shrinkage, and the risk or severity of transverse bridge deck cracking.

Healy and Lawrie (1998) defined the factors that control shrinkage cracking. These factors include: size of placement, time of day when placement is made, time for initiation of curing, maximum and minimum temperature where placement is made, rate of placement, design and detailing considerations, and mix design. As a result of their study, it was recommended that a comprehensive procedure be utilized, including mist spray and burlap curing with continuous wetting as the method for curing bridge decks.

Issa (1999) linked concrete cracking, and thus high magnitude of shrinkage, to a high evaporation rate resulting from inadequate concrete curing procedures during hot weather conditions. This effect is attributed to lack of concrete protection, inadequate coverage with a curing compound, and delay of concrete protection application. Kwak et al. (2000)

recommended that the slump should be less than 5 inches if the average relative humidity is 60 percent to avoid transverse cracking at interior supports with live loading.

Almusallam (2001) indicated that exposure conditions at the time of casting significantly affect the properties of both fresh and hardened concrete. The rate of water evaporation, shrinkage strain, and the area of cracks increased with increasing exposure temperature and wind velocity, and decreasing relative humidity. Plastic shrinkage cracks were noted earlier in concrete specimens exposed to elevated temperature and low relative humidity when compared to specimens exposed to low temperature and high humidity. The exposure conditions also influence the properties of hardened concrete. Elevated temperature exposure decreased the compressive strength. Moreover, the exposure conditions significantly affect the pore structure of concrete. Coarser pores were noted in the concrete specimens cast at 113°F than pores of those cast at 86°F.

2.4.1 Evaporation of Water from Fresh Concrete

Heat of hydration in fresh concrete, amount of plastic shrinkage, and plastic shrinkage cracking depend to a great extent on the rate of evaporation from fresh concrete, which influences the strength and durability of the concrete. It is recognized that the magnitude of evaporation from the surface of fresh concrete depends on the prevailing air temperature, relative humidity, wind speed, and the temperature of the fresh concrete. The evaporation rate depends on climatic conditions because at the beginning of casting, water exists at the surface of the concrete (bleeding water). Additionally, the moisture movement within the solid is rapid enough to maintain a saturated condition of the surface. The drying process of porous media can be divided into two periods referred to as initial and terminal drying period. In case of concrete, initial drying period consists of two evaporation stages. The mechanism of moisture removal during the first stage is equivalent to evaporation from a liquid water surface. The evaporation-drying rate of concrete can be calculated from heat transfer relationship (Razek and Enein 1999):

$$R_c = h_c (T_{da} - T_w) / L \quad (2-1)$$

where, R_c is the evaporation drying rate (lb/ft²/hr), h_c is convection heat transfer coefficient (Btu/ft²/hr/K) and can be calculated by relation $h_c = 4.28 + 0.375v$ (v is the wind velocity in ft/s), T_{da} is the dry bulb temperature (ambient air temperature) (K), T_w is the wet bulb temperature (K) and can be calculated from ASHRAE Psychrometric Chart No.6 (ASHRAE Handbook 1992) by knowing relative humidity and dry bulb temperature of air, and L is the latent heat of vaporization of water (Btu/lb).

2.5 CONSTRUCTION RELATED FACTORS

Construction-related factors include concrete mixing and delivery parameters, control of ambient temperature and humidity during placement, consolidation of fresh concrete, deck placement sequences, traffic-induced vibrations, construction loads, finishing procedures, and curing. Perfetti (1985) and Purvis (1989) reported that placement length and sequence did not influence transverse cracking. In contrast, some agencies recommend sequenced placements.

Purvis et al. (1995) observed the construction of eight bridge decks to identify procedures contributing to shrinkage and cracking. The general observation was that the construction procedures would not affect the performance of the eight sites observed with respect to deck cracking. During the observations, temperature was measured and shrinkage tests were conducted on the specimens to measure thermal and drying shrinkage. Based on this data, it was predicted that transverse cracks would develop in four of the eight bridge decks. Visiting the observed bridges later validated the prediction. It was concluded that these results further confirm that thermal and drying shrinkage are the primary causes of transverse cracking in bridge decks.

Krauss and Rogalla (1996) indicated that finishing procedures can affect cracking and any delay in finishing can increase cracking. They recommended that to prevent cracking, construction loads should not be allowed until the concrete has sufficiently hardened. Regarding mixing procedures, no correlation between the number of revolutions of transit mix trucks and deck cracking was found.

Similarly, Issa (1999) indicated that it is important to initiate placement in the positive moment regions prior to the negative moment regions. Traffic-induced vibrations do not cause any

damage to deck concrete that is well-proportioned and well-compacted with low slump. However, additional vibrations due to adjacent traffic and construction equipment may increase cracking in the case of under-consolidated, high slump concrete. Issa and Yousif (2000) stated that the vibration and movement imposed on the extended reinforcement is a major source of concrete bridge deck cracking.

Kwak et al. (2000) studied theoretically the effect of slab concrete placing sequence on transverse cracking of slab decks. They found that the effect of slab placement sequence is negligible for both short-term and long-term behavior of steel box girder bridges.

2.6 TEMPERATURE AND THERMAL STRESS DEVELOPMENT

2.6.1 Overview

This section focuses on thermal stresses as the main cause of restraint shrinkage cracking. Thermal stresses develop as the result of deck temperature changes and the restraint imposed by the girders. Very early thermal stresses develop in a bridge deck as a result of hydration temperature. The other sources of temperature changes are due to diurnal and seasonal temperatures. Thermal stresses are greatly affected by many factors such as continuity, relative deck to girder stiffness, the concrete material itself, geographic location, etc. The NCHRP report 380 developed elastic equations to predict shrinkage and thermal stresses in a bridge deck. This topic is organized in six sections. Section 2.6.2 presents the hydration temperature stresses, Section 2.6.3 presents the cause of diurnal temperature stresses, Section 2.6.4 presents the effects of seasonal temperature stresses, and finally Section 2.6.5 presents the summary factors affecting thermal stresses and precautions.

2.6.2 Hydration Temperature Stresses

Hydration of cement is an exothermic process that causes a temperature rise within a concrete mass. This initial temperature rise and expansion induces no residual compressive stresses in concrete when changing from a plastic state to a solid state. This is because of the extremely low modulus of elasticity of the concrete at this plastic-to-solid state. When the concrete reaches its peak temperature, it has also solidified. Subsequently, the hardened concrete begins to cool to ambient temperature. During the cooling process, longitudinal beams restrain the deck shrinkage. This phenomenon will in turn cause tensile stresses and transverse cracking in the

deck. The magnitude of thermal shrinkage in the deck depends on the difference between the peak concrete temperature and the temperature of supporting beams at that time. The temperature of supporting beams is usually equal to the ambient temperature, unless the deck is heated underneath as part of cold-weather curing. Unlike deck drying shrinkage, which may take over a year, thermal shrinkage affects the concrete over a short period (a few days). Thus, concrete creep properties cannot be fully used to relax the concrete and mitigate cracking. Analytical work established that 228-microstrain thermal shrinkage would be necessary to initiate cracking (Purvis et al. 1995).

Typically, for conventional concrete bridge decks cured under normal weather conditions, the amount of restrained thermal shrinkage is on the order of 170 microstrain or less, which corresponds to a differential deck/beam temperature of about 30 °F or less. This is usually not sufficient to initiate cracking (less than the threshold of 230 microstrain). However, the cracking threshold may be exceeded when the thermal shrinkage is superimposed upon the drying shrinkage (Babaei and Fouladgar 1997).

Krauss and Rogalla (1996) stated that thermal stresses from early hydration temperatures are most extreme in steel-girder bridges, and also high when the deck is cast separately over precast concrete girders. Concrete conducts less heat than steel, and the large girder mass will cause a delay in the adjustment of the girder temperature to the change in deck temperature. Steel girders typically conduct heat more quickly than concrete girders, and upper flanges will adjust to the deck temperature, reducing the temperature difference at the interface. A 50 °F temperature change in the deck relative to the girders can cause stresses greater than 200 psi when the concrete has an early effective modulus of elasticity of only 0.5×10^6 psi, and greater than 1000 psi when the early effective modulus is 2.5×10^6 psi.

2.6.2.1 ACI 207.2R Procedure for Calculating Hydration Temperature

The rate and magnitude of concrete temperature rise during the hydration process depend on several factors. These factors include cement composition (cement type) and fineness, amount of cement per unit volume of concrete, ambient temperature, concrete placing temperature, and amount of heat lost or gained during hydration process. The exposure conditions and volume to exposed surface area ratio of bridge deck governs the amount of heat lost or gained.

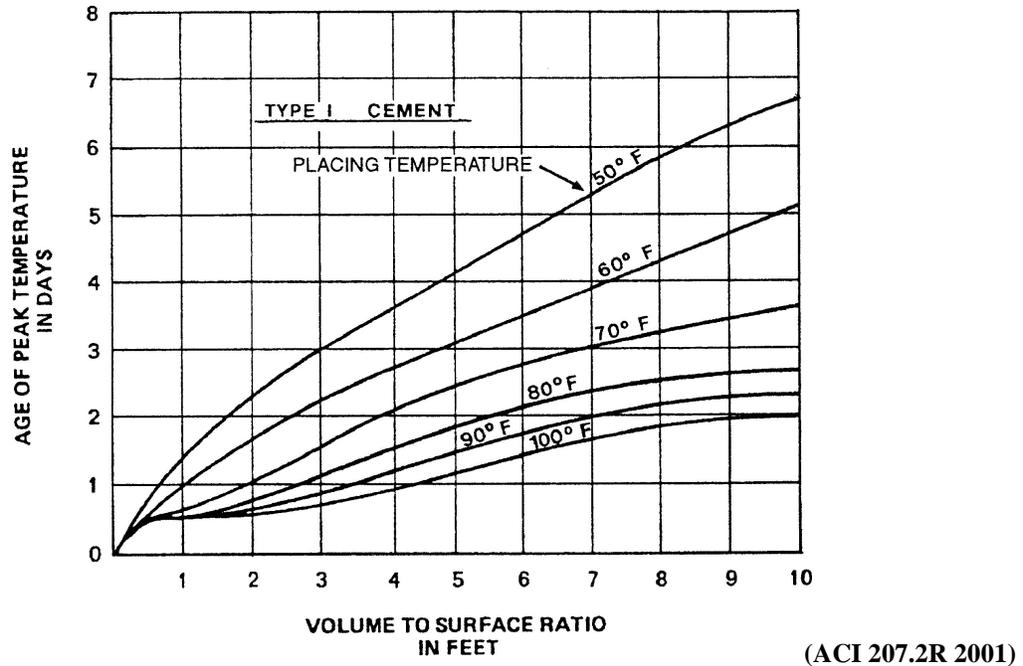
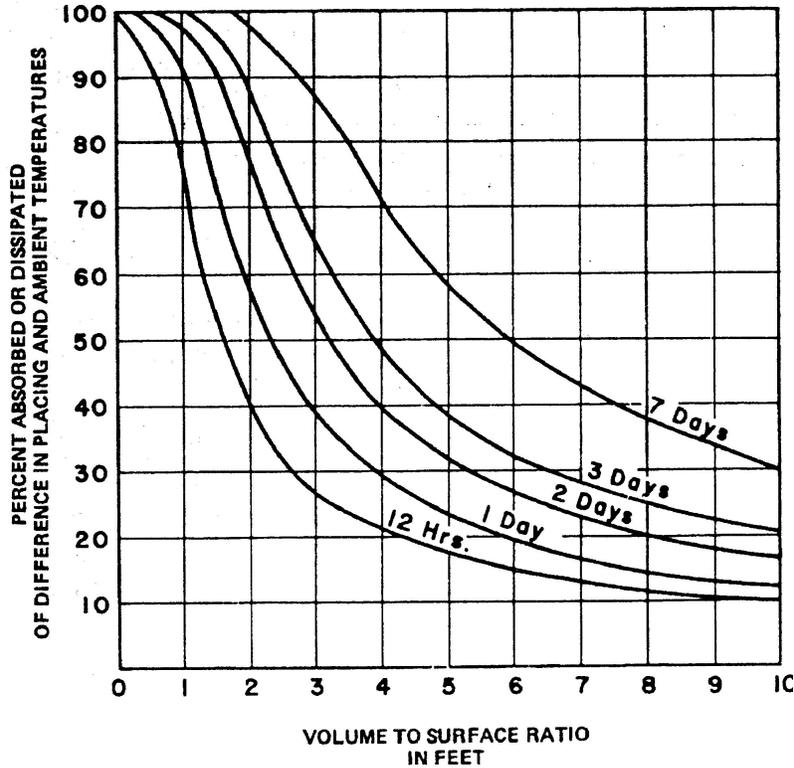


Figure 2-3. Effect of concrete placing temperature and volume to surface (exposed to environment) ratio on age at peak temperature for Type 1 cement

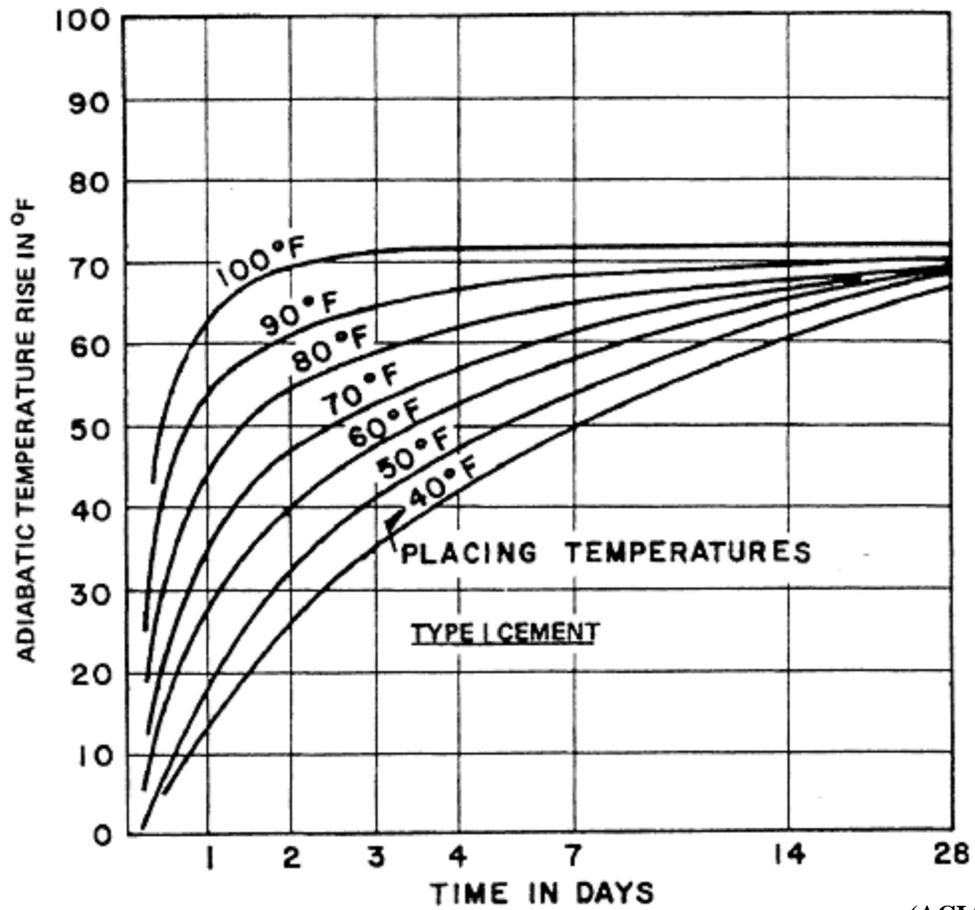
If the concrete placing temperature and volume/surface ratio of bridge deck are known, the age at which the peak temperature could be attained can be determined from Figure 2-3, provided Type 1 cement is used. During the hydration process there is a temperature difference between the deck and the ambient air. After the age at peak temperature is determined, Figure 2-4 can be used to compute the difference in percent absorbed or dissipated in placing and ambient temperature.



(ACI 207.2R 2001)

Figure 2-4. The effect of volume to surface ratio and age at peak temperature on percent absorbed or dissipated of difference in placing and ambient temperature

When bridge decks are considered, the volume to exposed-surface ratio (V/S) is less than one foot. According to Figure 2-4, due to a significantly higher rate of heat absorbed or dissipated, the effective placement temperature equals ambient temperature. The placing temperature is also a parameter for adiabatic temperature rise (Figure 2-5). The temperature rise within the concrete deck also depends on the exposed deck surface condition. As depicted in Figure 2-6, if the exposed concrete surface is kept wet, the temperature rise decreases significantly.



(ACI 207.2R 2001)

Figure 2-5. Effects of placement temperature on adiabatic temperature rise

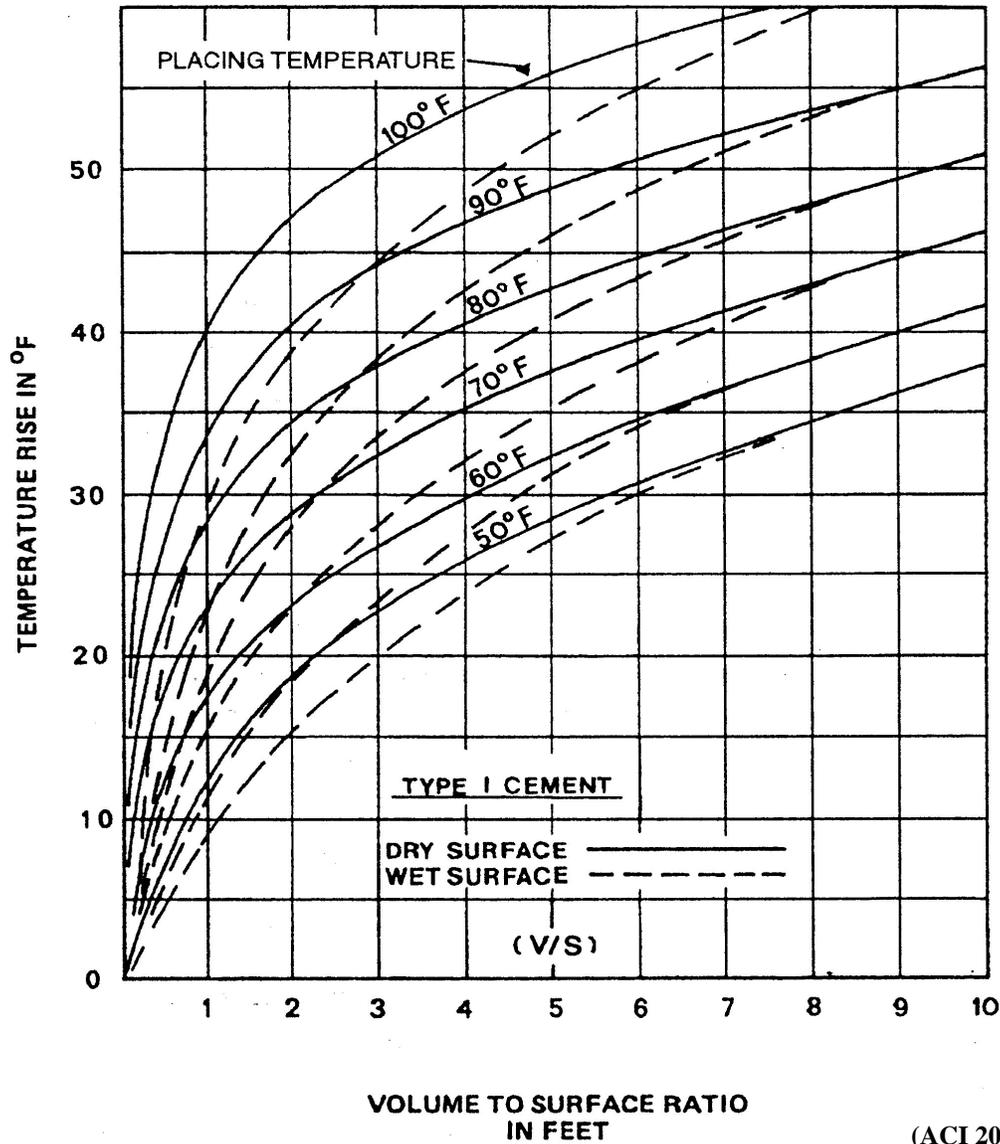
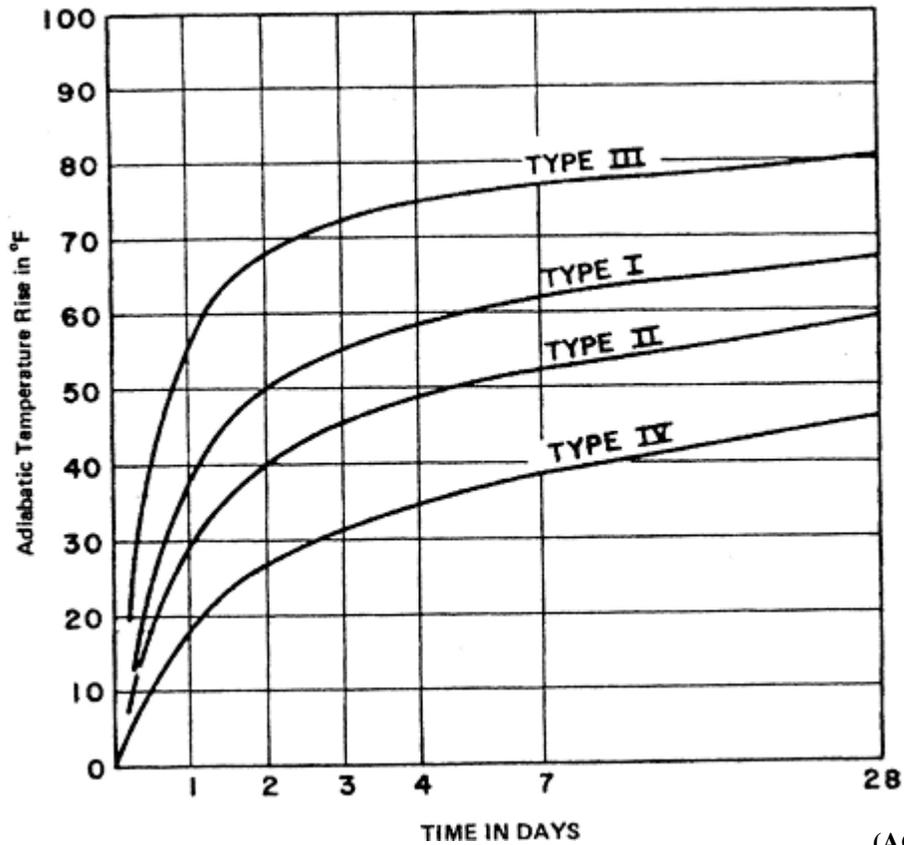


Figure 2-6. The effect of volume to surface ratio and exposed surface condition on temperature rise of concrete members (376 lb/yd³)

Cement type, content, and its fineness affect the adiabatic temperature rise in a concrete element. Figure 2-7 shows the adiabatic temperature rise for different types of cements. In developing these graphs, a cement content of 376 lb/yd³ was used and the respective fineness is given in Table 2-8. If cement with a different fineness is used, Figure 2-8 can be used to calculate the correction factor. In addition, the total quantity of heat generated at any age is directly proportional to the total amount of cement in the concrete mix.



(ACI 207.2R 2001)

Figure 2-7. Variation of adiabatic temperature rise with age of concrete for different types of cements

Table 2-8. Cement Types and Fineness used for Developing Graphs in ACI 207.2R

Cement Type	Fineness ASTM C 115 ft ² /lb
I	875
II	924
III	992
IV	933

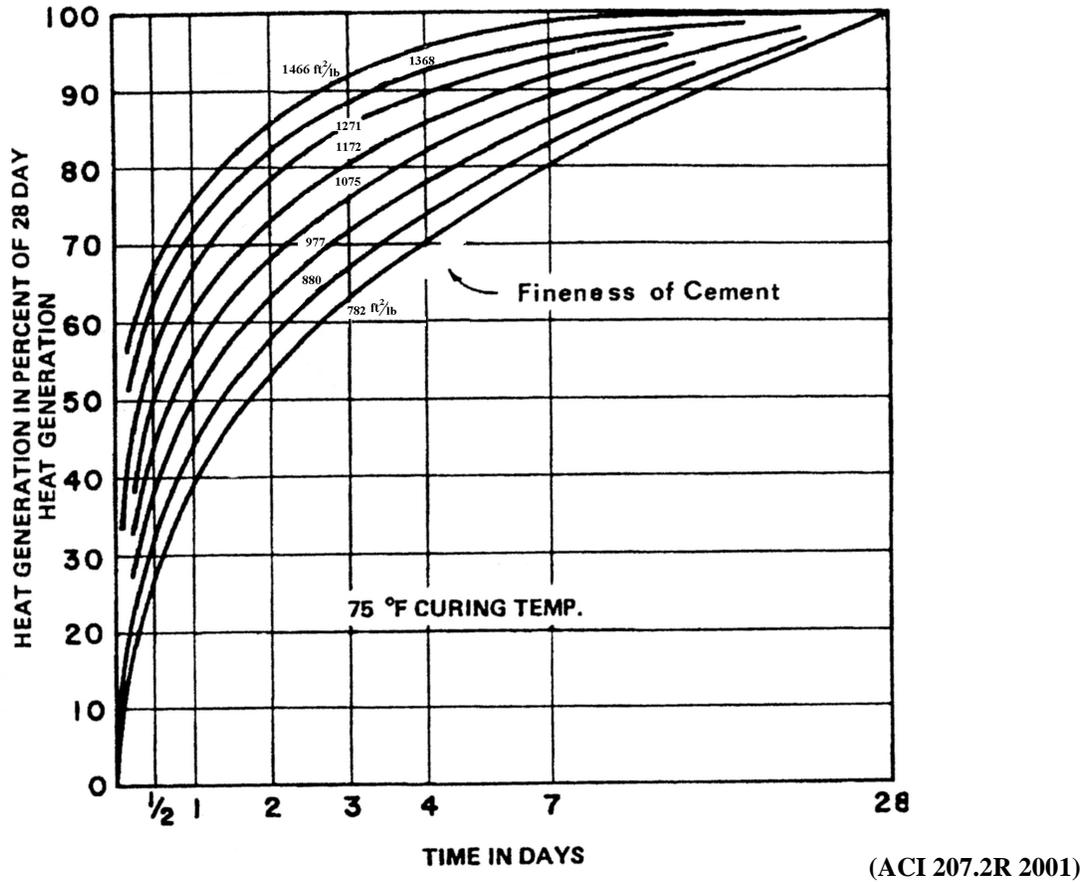


Figure 2-8. Effects of cement fineness on heat generation

2.6.3 Diurnal Temperature Stresses

For most bridges, diurnal temperature changes within the bridge produce the largest thermal stresses. The diurnal temperature cycle of the bridge decks usually exceeds the ambient air temperature cycle, especially on surfaces directly exposed to solar radiation. Bridge decks in moderate or extreme climates can easily experience 50°F diurnal temperature cycles. Because heat does not instantly transfer to the girders, temperature gradients usually exist within bridge decks. The parametric study conducted by Krauss and Rogalla (1996) revealed that a linear temperature gradient in the deck, not a uniform temperature gradient, typically produces the largest deck stresses and the greatest risk of transverse cracking. Diurnal thermal stresses are often larger over the interior supports of a continuous-span structure. Thermal tensile stresses above these interior supports may exceed 1400 psi with steel girders and nearly 2000 psi with concrete girders. All of these stresses are sufficient to cause transverse deck cracking, especially over the interior supports of a continuous-span structure.

2.6.4 Seasonal Temperatures Stresses

Stresses from seasonal temperature changes are small or negligible in most concrete bridges because decks and girders have similar or equal expansion coefficients. Deck stresses in concrete bridges caused by seasonal (uniform full-depth) temperature changes occur only because of expansion differences between the concrete and deck reinforcing. When steel girders support the concrete deck, seasonal temperature differences will cause thermal stresses if the concrete has a different thermal expansion rate than the steel (Krauss and Rogalla 1996).

2.6.5 Summary of Factors Affecting Thermal Stresses and Precautions

Many factors affect shrinkage and thermal stresses. The primary factors include the concrete material itself, the geometry of the bridge, construction techniques, and the environment.

Krauss and Rogalla (1996) indicated that aggregates affect shrinkage and thermal stresses, and therefore transverse deck cracking. Aggregates with a lower modulus of elasticity reduce the modulus of elasticity of concrete, and therefore reduce shrinkage and thermal stresses. These aggregates also often increase creep, further reducing shrinkage stresses. Thermally conductive aggregates may reduce thermal gradient within the deck, lowering thermal stresses. Increasing the aggregate content can reduce the concrete coefficient of thermal expansion by reducing the more thermally expansive paste content. Using concrete with a lower coefficient of thermal expansion will reduce the thermal stresses.

Babaei and Fouladgar (1997) recommended that thermal shrinkage should be limited to 150 microstrain. This can be achieved by maintaining the concrete/deck differential temperature to less than 22°F for at least 24 hours after the concrete is placed. It was also recommended that the following procedures be followed to control thermal shrinkage:

- Use of cement content as low as possible, such that lower heat of hydration is generated.
- Use of Type II cement instead of Type I cement, in order to reduce the heat of hydration.
- Examination of the cement chemistry, since for a given type of cement, some brands may generate a higher heat of hydration due to their chemical and physical properties.

- Use of retarders in the mix, because they delay the hydration process and reduce the difference of temperature within the concrete and the ambient temperature.
- Use of pozzolans and slag as a partial substitute for Portland cement.
- Protection of the concrete from solar radiation to reduce the temperatures due to hydration and insulation of the bridge to reduce the rate of cooling.

2.7 SHRINKAGE EFFECTS ON DECK CRACKING

2.7.1 Overview

Transverse cracking of concrete decks takes place at early ages, some immediately after construction or during some period of time thereafter. Cracking at early ages may be expressed in terms of the volume change of concrete. One of the important volume change effects in concrete at early ages is shrinkage. There are four types of shrinkage that take place in concrete. They are evaporation, drying, autogenous, and carbonation shrinkage.

As concrete hardens, evaporation shrinkage occurs. Concrete is a plastic material when it is fresh. Hardening of concrete is due to hydration of cement. This hydration process increases the concrete mass to a certain temperature. Meanwhile, the surface of the deck is subjected to environmental effects such as high or low temperatures, evaporation, etc. These external effects cause thermal stresses due to evaporation cooling of water in concrete. The amount of stress developed by the evaporation shrinkage is related to ambient and concrete temperatures. Greater temperature differences between the concrete surface and its surrounding environment cause a considerable amount of evaporation shrinkage (Kovler 1996).

Autogenous shrinkage is associated with the loss of moisture from capillary pores of concrete due to hydration of cement. Since the autogenous shrinkage is related to cement hydration, higher cement content, water/cement ratio, cement paste volume, and rate of hydration of cement are important (Mokarem et al. 2003 and Bissonette 1999). It is important to know the difference between drying shrinkage and autogenous shrinkage in order to understand the effects of both parameters, since they occur simultaneously. Drying shrinkage can be expressed as the loss of absorbed water from capillary pores of concrete due to environmental effects (Alsayed et al. 1998). Macro and micro-diffusion and moisture distribution in concrete are important factors in

drying shrinkage. Macro-diffusion is the movement of water through the path of least resistance. This movement takes place in larger pores available in concrete. Micro-diffusion is the movement of water and/or moisture between capillary pores and gel pores. This movement of water in pores has an effect on concrete deformation (Mokarem 2002).

Carbonation shrinkage occurs at the surface of concrete members. The existence of carbon dioxide and humidity in the environment cause carbonation shrinkage. Carbonation reaction occurs during a long period of time, thus is not an issue for early-age deck cracking. The carbonation shrinkage amount and resulting defects are relatively small compared to other shrinkage types and age of concrete (Mokarem 2002).

2.7.2 Factors Affecting Shrinkage

Factors affecting shrinkage can be classified as controllable and uncontrollable parameters. Controllable parameters are cement type, aggregate, water content, and construction practices. Uncontrollable parameters are environmental factors and material properties.

2.7.2.1 Cement Type and Content

High strength concrete develops higher strength than normal strength concrete at early ages. Shrinkage in high strength concrete generates more strain than normal strength concrete because high strength concrete contains more cementitious material and better aggregate grading. The shrinkage rate of normal strength concrete is less than that of high strength concrete (Altoubat et al. 2001, Samman et al. 1996 and Wiegrink et al. 1996).

Cement composition controls many properties of concrete such as strength, durability, and stability for a given aggregate and cement. Therefore, cement composition cannot be disregarded when analyzing the durability of concrete. Concrete is composed of cement paste and aggregate. Concrete contains aggregate as approximately 70% of its volume; this aggregate acts mainly as a passive filler material. However, cement paste is responsible for the quality of concrete. The Ordinary Portland Cement (OPC) that is available in the market is much finer in specific surface than the cements used two decades ago. Cement technology has undergone significant changes in the world during the last two decades. Today's cement contains more C_3S and less C_2S when compared to the old cements. Therefore, rapid compressive strength gain is

observed. New cement requirements are different from old cements with respect to quantity and water demand. OPC consumes less water, but generates more heat of hydration, since it has a finer surface and higher C₃S content (Uzzafar 1992). Krauss (2003) reports similar consequences of new cements. In order to draw conclusions about the effect of cements on early-age cracking, old and new cement properties are compared. Old cements have more coarse particles and lower early-age strengths than OPC. It is obvious that OPC's shrinkage tendencies will be different due to its fineness and early-age strength as shown in Table 2-9 (Krauss 2003).

Table 2-9. Change of Cement Properties and Strength

Cement Properties					Compressive Strength (psi)				
Type	Composition (% by weight)				Fineness (ft ² /lb)	1 Day	3 Day	7 Day	28 Day
	C ₃ S	C ₂ S	C ₃ A	C ₄ AF					
1994									
I	51.8	18.2	10.6	7.4	1826	2485	3760	4620	5605
II	54.9	17.3	7.1	10.6	1855	2463	3850	4786	5570
III	51.7	18.0	10.4	6.9	2661	3981	5167	5923	6782
IV	42.2	31.7	3.7	15.1	1655	900	1725	2529	5417
1950									
I	44.6	27.4	11.2	8.3		520	1610	2760	4450
II	43.6	30.9	5.2	13.2		520	1400	2140	3790
III	52.9	18.6	10.5	9.5		1530	3680	5080	6340
IV	27.6	48.9	4.4	12.3		240	740	1220	2830

The effect of cement on concrete shrinkage is a complex topic that is still being investigated. For instance, some researchers state that cement properties have little or no effect on concrete shrinkage for a given amount of cement (Li et al. 1999). Furthermore, Mehta (1986) states that cement fineness and composition have an effect on the hydration process but not on hydration products, so the cement fineness and composition change have a slight effect on cement mortar. In terms of concrete shrinkage, those effects are negligible due to their insignificance. On the contrary, ACI 224.R-01 (2001) states that cement properties have a direct effect on concrete shrinkage. Neville (1995) states that higher shrinkage of cement does not result in higher concrete shrinkage. The chemical composition of cement is believed to be ineffective to concrete shrinkage. On the other hand, the ACI 224 committee report states that finer cements

generally cause increased shrinkage in concrete, but the increase in fineness is not proportional to shrinkage.

An advantage of coarse cement particles is that they generate relatively less shrinkage when compared with finer cements. On the other hand, coarser cement requires a longer curing period to avoid low strength development, larger pores, and high porosity (Bentz et al. 1999).

Cements that have lower C_3A/SO_3 ratios, lower alkalinity, and higher C_4AF contents shrink less. Changing the cement type is another means of reducing shrinkage. As mentioned earlier, Type II cement has a tendency to generate lower shrinkage when compared with Type I cement (ACI 224.R-1 2001)

The ambiguity of pozzolans is another topic to be discussed. For instance, Neville (1995) states that mineral additions such as fly ash and slag increase the shrinkage. Specifically, higher proportions of mineral admixtures in blended cements lead to higher shrinkage. On the other hand, ACI 224 states that the use of mineral admixtures could increase the mixing water requirement, thus increasing the water/ cementitious material ratio. This increase in water does not necessarily mean an increase in shrinkage (ACI 224.R 2001). Research showed that pozzolans have a positive effect on shrinkage. Li et al. (1999) states that ground granulated slag and silica fume blend could result in better performance and less shrinkage than OPC silica fume concrete.

The most commonly used cement replacement pozzolan is fly ash. The use of fly ash in concrete has many different advantages such as an increase in long-term strength and durability. On the contrary, disadvantages of the use of fly ash are an increase in autogenous shrinkage and variable permeability depending on the type of fly ash used (Naik et al. 1995). Swamy (1997) states that if satisfactory curing can be performed in the field, early-age autogenous and long-term drying shrinkage will be less than that of OPC concrete.

Adding silica fume at about 10% of cement weight could reduce long-term drying shrinkage as well as help in the treatment of insufficient curing duration problems. This replacement reduces the long-term drying shrinkage strain in both laboratory and field cured concrete specimens (Alsayed 1998). On the other hand, silica fume addition will strongly affect the autogenous

deformation and relative humidity change in concrete. The hydration reaction of silica fume has a heat sensitivity that is markedly different from cement hydration (Jensen 1999). As a general concept, adding some amount of micro silica often results in better performance in severe environments (Sabir 1997).

Pozzolans may also be used in the production of blended cements. Blended cement production is common in Europe and Asia. The research focusing on durability and shrinkage performance of concrete concluded that the blended cements perform better than OPC. A number of disadvantages of ordinary Portland cement can be overcome by using additional pozzolans such as fly ash, slag, and silica fume. Blended cements give users a chance to enhance cement performance when considering the harsh environments to which concrete will be subjected (Malhotra et al. 1995 and Nehdi 2001). Swamy (1989) states that the use of appropriate blended cement and high range water reducer is one of the most durable design considerations for concrete in severe environments.

2.7.2.2 Water/Cement Ratio

Some researchers focus on the role of water content of mix design in shrinkage. The U.S. Bureau of Reclamation test results showed that any increase in the amount of water content results in an increase in shrinkage. Reduction in concrete shrinkage is possible by keeping the water content to a minimum and total aggregate content to as high a level as possible (ACI 224-R-1 2001). On the other hand, some recent research findings show that lowering the water content could increase the concrete shrinkage (Igarashi et al. 2000, Samman et al. 1996, and Wiegrink et al. 1996). Mehta (1986) states that for a given cement content, an increase in water content increases the shrinkage. Similarly, for a given water/cement ratio, an increase in cement content also increases the shrinkage. Those results are expected due to increase in the amount of cement paste, but in practice they do not always occur. There is a need to have a better understanding of the role of water in concrete shrinkage. It appears that an optimum value of water/binder ratio could be determined for less shrinkage of concrete. Bissonnette et al. (1999) suggests an optimum water/cement ratio range of 0.35-0.50 and found a strong relationship between cement paste volume and shrinkage amount.

2.7.2.3 Aggregate Type and Fine Aggregate to Coarse Aggregate Ratio

Cement is considered to be the source of shrinkage in concrete. Shrinkage develops tension on the binder (cement) and compression on the aggregate. The amount of tension and compression is related to the age of concrete, curing, and mechanical properties of the coarse aggregate. If the mechanical properties of coarse aggregate are poor, then compression will cause a volume change in the aggregate. This volume change will also affect the concrete integrity by contributing to further shrinkage. In the same manner, the amount of coarse aggregate causes internal restraint to shrinkage. The volume of cement paste is a factor for shrinkage (Bissonnette et al. 1999). Mehta (1986) describes that elastic modulus of aggregate has a greater effect than shape and size of aggregate. Additionally, aggregate density that is related to the porosity of aggregate is a good parameter to be checked for the control of shrinkage. As a general concept, shrinkage compensating concrete mixes that consider the above parameters are preferred (Altoubat et al. 2001).

2.7.2.4 Curing and Drying Shrinkage

Curing has a direct impact on drying shrinkage of concrete members. Type and duration of curing are other factors. Moist curing has not been found to be sufficient in reducing drying shrinkage, whereas steam curing reduces the concrete shrinkage (ACI 224.R-01 2001). Depending on the curing conditions, approximately 66% of total shrinkage may be reached in the first three months after concrete placement (Alsayed 1998).

Sealing of concrete with a curing compound that forms an impervious membrane could reduce the drying shrinkage, but it will not eliminate the autogenous shrinkage (Altoubat et al. 2001). Silane treatment of concrete members in the field shows a resulting decrease in drying shrinkage. Therefore, this treatment should be considered in curing practices (Xu et al. 2000).

2.7.2.5 Relative Humidity

Microdiffusion from the pores of concrete is a time dependent process that takes place over a long period. The field tests performed over a wide range of mix designs for 20 years of monitoring showed that 20 to 25% of 20 year drying shrinkage was realized in 2 weeks, 50 to 60% in 3 months, and 75 to 80% in one year. An increase in the atmospheric humidity is expected to slow down the relative speed of diffusion. The Committee of European Concrete

Institute (CEB) states that at 100% relative humidity drying shrinkage is assumed to be zero. When relative humidity is about 80%, the expected drying shrinkage is about 200 microstrain, and for 45% relative humidity it is about 400 microstrain (Mehta 1986). In the same way, Bissonette et al. (1999) states that relative humidity in the range of 48 – 100 % is inversely proportional to the drying shrinkage.

2.7.2.6 Influence of Concrete Element Shape and Size

Since the diffusion of water in concrete to the environment is a function of humidity, the length of the path will control the rate of reaction traveled by water. For constant humidity, both the size and the shape of a concrete element determine the amount of drying shrinkage (Mehta 1986). Distress evaluations of field specimens proved that the size of a concrete member significantly affects shrinkage. If the volume to surface ratio is greater, less shrinkage can be expected (Bissonnette et al. 1999).

2.7.2.7 Effects of Restraint

If the shrinkage of concrete occurred uniformly due to loss of water, and curing with no restraints, concrete would not crack. However, concrete is always subjected to some restraint either by the presence of aggregate and/or reinforcing steel or by non-uniform shrinkage (ACI 224.R-01 2001 and Gilbert 2001).

Restraint of concrete gradually increases the tensile stress in concrete and the resultant stress causes time-dependent cracking of decks. These existing restraint cracks widen due to flexural effects and deflections increase with time. Since the designers do not consider shrinkage and shrinkage cannot be modeled in current design procedures, restraint shrinkage will be a challenging issue for concrete structures (Gilbert 2001).

The effects of restraint on cracking will be dealt with in significant detail later in the section entitled “Parameters Influencing Deck Cracking”.

2.8 ANALYSIS OF CRACK FORMATION

Long-term effects to crack initiation are investigated by Borst and Berg (1986). Although the study focused mainly on the creep effect, it enables the understanding of the cracking mechanism in concrete. It is discussed that in earlier research, the concrete stiffness is underestimated by

assuming it to be zero upon cracking. In order not to misjudge the concrete stiffness after cracking, a smeared finite element model is utilized in the analysis, which is established by combining cracking with inelastic concrete behavior between the cracks.

In the smeared crack model, total strain is modeled as a combination of the concrete strain and the crack strain. Concrete strain and similarly crack strain may be decomposed further into several components. In the model, shear and normal stresses on cracked concrete are derived with respect to normal and shear strains. The instantaneous concrete moduli and relaxation effects are also incorporated in the model. Shear crack strain is included by using rotation matrices to convert from local coordinates to global coordinates (Borst and Berg 1986).

According to Borst and Berg (1986), cracking due to time dependent effects such as creep, thermal dilatation, and shrinkage may not be explained only by the stress-strain relation. The cracking stress-strain concept, which is generally used, fails to explain the cracking due to the stresses below the cracking strength of the concrete. Therefore, to derive a more realistic solution, a stress-strain envelope shown in Figure 2-9 (a) is generated by joining the concrete strain from the linear-elastic model and normal strain due to cracking shown in Figure 2-9 (b). Any combination of stress and strain in the principal directions on the envelope generates cracking (Borst and Berg 1986). When a specimen is loaded such that the stress reaches the amount indicated by point A, path AB shown in Figure 2-9 (a) is followed. The concrete cracking initiates at point B.

Borst and Berg (1986) conclude that long-term cracking cannot be explained by basic stress criterion. When a combination of normal stress (f_c) and normal strain (ϵ) reaching the envelope is achieved, cracking initiates. As noted in Figure 2-9, f_c is the tensile stress, f_t is the tensile strength of concrete, E is the elasticity modulus, ϵ is the strain normal to the cracking plane measured before cracking, ϵ_c is the strain of concrete at the tensile strength of concrete, f_{tc} is the cracking strength (80% of ultimate tensile strength of concrete), ϵ_{tc} is the cracking strain, and ϵ^{cr} is the strain normal to the cracking plane measured after cracking. It is also stated that a more elaborate nonlinear stress-strain relation would be more accurate rather than the linear relation used in the model.

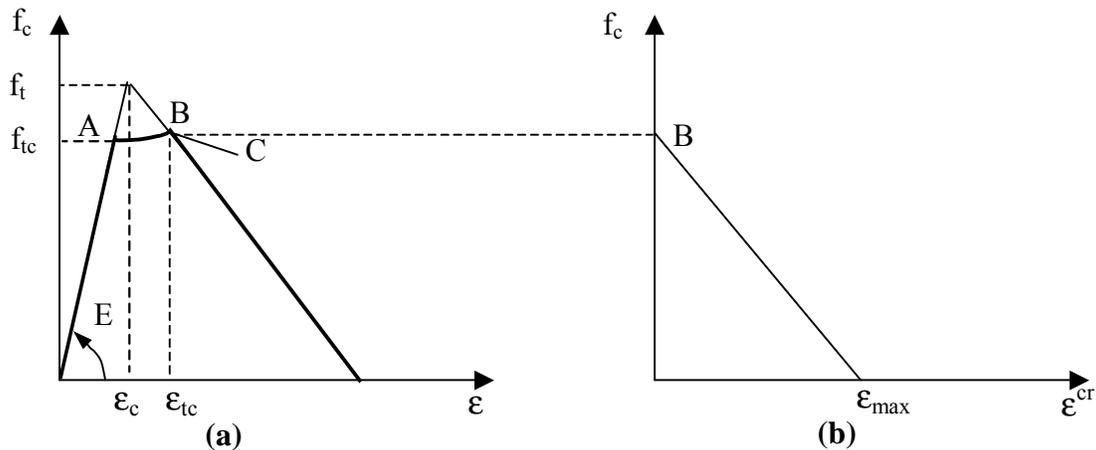


Figure 2-9. (a) Stress-strain envelope for cracking of concrete and (b) stress-normal crack strain after crack initiation (assuming linear-elastic behavior)

2.9 CONCLUSIONS AND THE NEED FOR FUTURE RESEARCH

Early-age bridge deck cracking is the most detrimental effect on bridges reported by all of the State DOTs. Although there have been many studies on the cause of early-age deck cracking, the problem still exists. This indicates the need for more research and investigation to solve this problem.

The restrained thermal and shrinkage effects coupled with construction practices are the main parameters in need of investigation.

The early-age deck cracking due to restraint thermal stresses can be controlled by maintaining the concrete/deck differential temperature under 22°F for at least 24 hours after the concrete is placed. These recommendations are for directly impacting the crack reduction on concrete bridge decks. However, before each recommendation is implemented a comprehensive review should be made and the cost should be evaluated. It is recommended that the following changes be implemented to control thermal and shrinkage effects on deck cracking:

1. Use of cement content as low as possible (Purvis et al. 1995, Krauss and Rogalla 1996, Babaei and Fouladgar 1997, French et al. 1999, Mehta et al. 1994, ACI 211.1-97 2001, and ACI 207.2R 2001).

2. Use of Type II cement instead of Type I cement, in order to reduce the heat of hydration (Purvis et al. 1995, Babaei and Fouladgar 1997).
3. Use of a cement brand whose chemistry generates a lower heat of hydration and therefore lower shrinkage due to its chemical and physical properties (Babaei and Fouladgar 1997).
4. Use of retarders in the mix, because they delay the hydration process and reduce the temperature difference between the concrete and the ambient temperature (Purvis et al. 1995, Babaei and Fouladgar 1997).
5. Use of pozzolans and slag are recommended as a partial substitute for Portland cement. The blended cements perform better than ordinary Portland cement (OPC). Blended cements give users a chance to enhance cement performance when considering the harsh environments to which concrete will be subjected (Babaei and Fouladgar 1997, Malhotra et al. 1995, Nehdi 2001, Li et al. 1999, ACI 212.3R-96 2001, and ACI 207.2R 2001).
6. Use of optimum water/cement ratio range of 0.35 to 0.50. This range agrees with the MDOT specifications (Babaei and Fouladgar 1997, Bissonnette et al. 1999, and ACI 211.1-97 2001).
7. Protection of concrete from solar radiation to reduce the temperatures due to hydration as well as insulation of the bridge to reduce the rate of cooling (Purvis et al. 1995, Babaei and Fouladgar 1997, Issa 1999, Almusallam 2001, and Razeq and Enein 1999).
8. Placement of concrete at night in hot weather (Purvis et al. 1995, Krauss and Rogalla 1996).
9. Use of correct type and duration of curing to improve concrete durability (Issa 1999, Almusallam 2001, Day 1999, Aitcin 2003, Healy and Lawrie 1998, Swamy et al. 1986, and Swamy 1997).
10. Avoidance of vibration due to adjacent traffic and construction equipment because it may increase cracking in the case of under-consolidated, high-slump concrete. Vibration and movement imposed on the extended reinforcement are major sources of cracking in concrete bridge decks (Issa 1999, Issa and Yousif 2000).

3 MULTI-STATE SURVEY

3.1 OVERVIEW

A nationwide survey on the experience of other states with the problem of early-age reinforced concrete (RC) bridge deck cracking was administered after reviewed and approved by the Research Advisory Panel (RAP) members. The survey was sent to the State DOTs in November 2002. Thirty-one State DOTs responded to the survey, giving a response rate of 62 percent. The response rate is satisfactory considering an average response rate of 20% is reported for most surveys. The survey was first administered by e-mail, requesting a web submission. After 60 days, personalized e-mails and letters were sent to each non-responding agency. The responses were received through email, fax, and postal mail. The list of respondents is shown in Table 3-1.

Table 3-1. List of Respondent States and Media of Response

Media of Response	Respondent States
Web Submission	Alabama, Arkansas, Connecticut, Georgia, Hawaii, Idaho, Kansas, Kentucky, Massachusetts, Michigan, Montana, New Hampshire, New Jersey, North Carolina, New Mexico, Pennsylvania, Rhode Island, South Carolina, South Dakota, Tennessee, Texas, Virginia, Washington, Wisconsin
Fax	Arizona, California, Minnesota, New York
Postal Mail	Illinois, Maryland, Oklahoma

In order to represent the geographic location of the responding states, the exposure map is given in Figure 3-1.

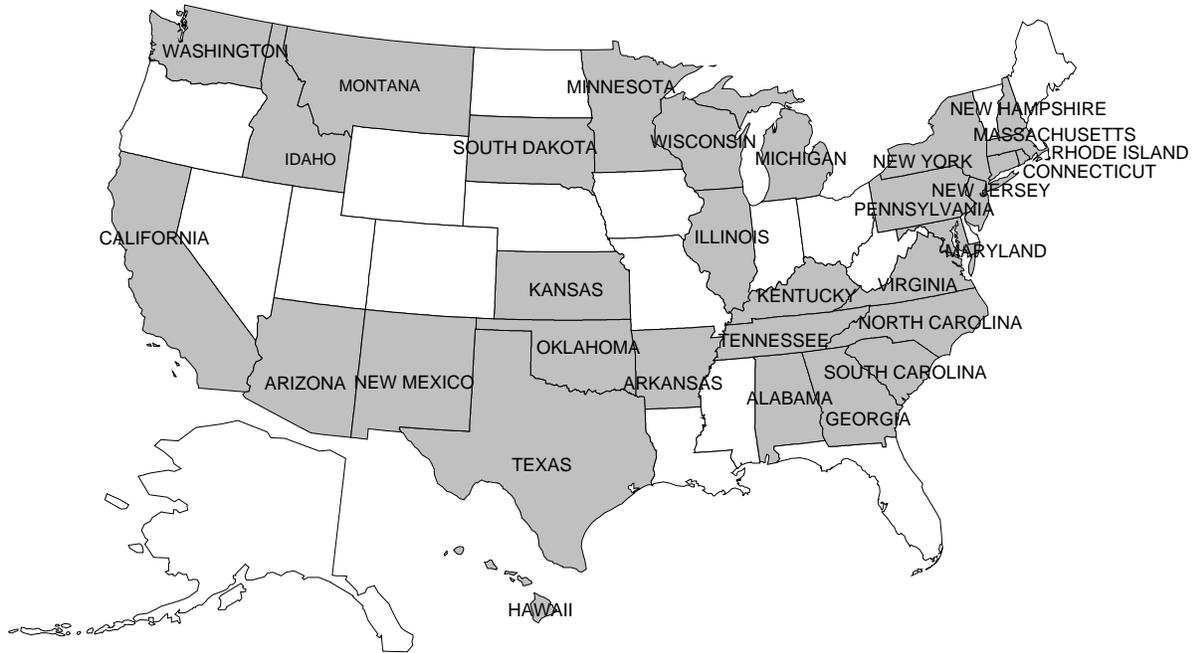


Figure 3-1. States who responded to the survey

3.2 ANALYSIS OF SURVEY DATA

Upon compilation of the survey data, an analysis has been performed. Summaries of the responses are presented in the following figures.

It has been found that 30 of 31 responding states (97%) have the problem of early-age deck cracking except Hawaii (Figure 3-2). Twenty-five states reported having experienced cracking during the first few months of service, while 11 responded as during the first year of service. Twenty-nine states (78%) identified the mode of cracking as transverse and six states (16%) as longitudinal.

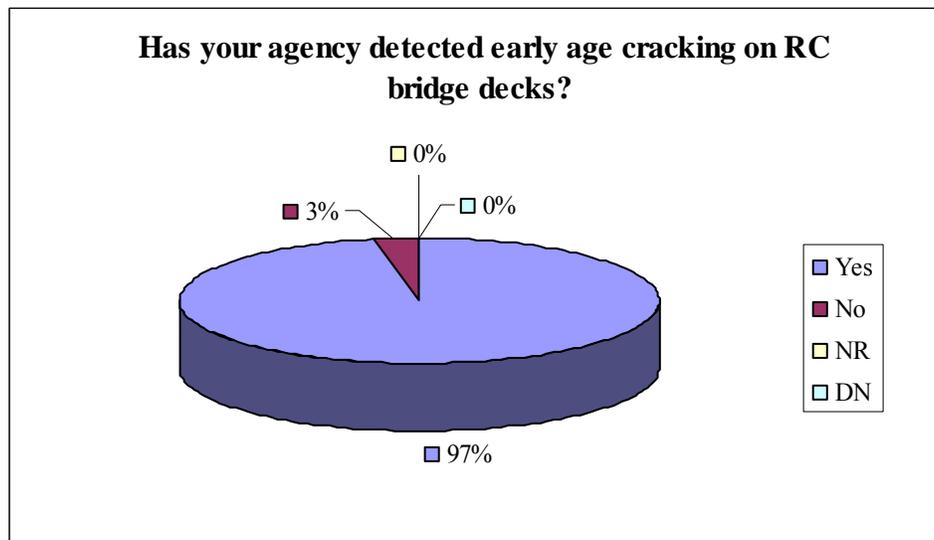


Figure 3-2. Frequency of early-age cracking observed on decks

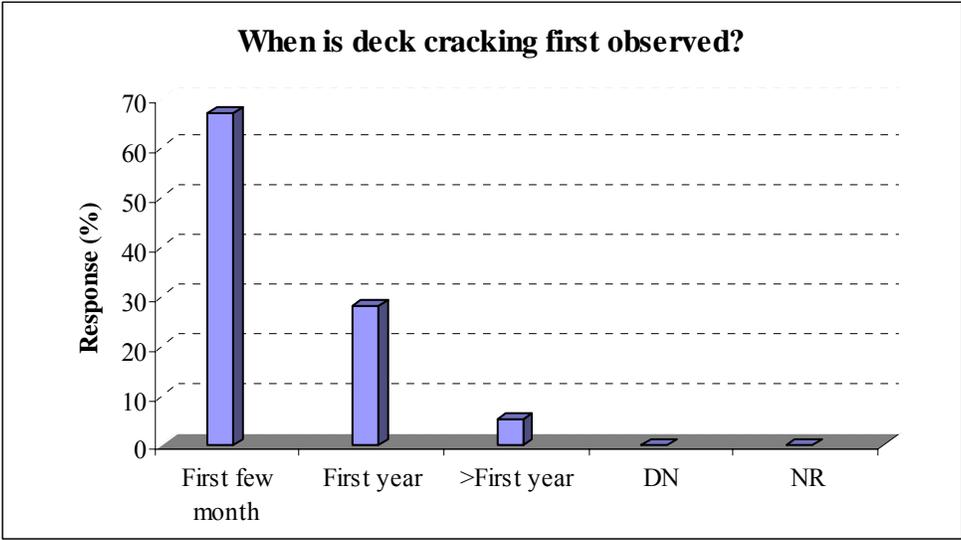


Figure 3-3. Frequency of cracks observed during different ages of bridge decks

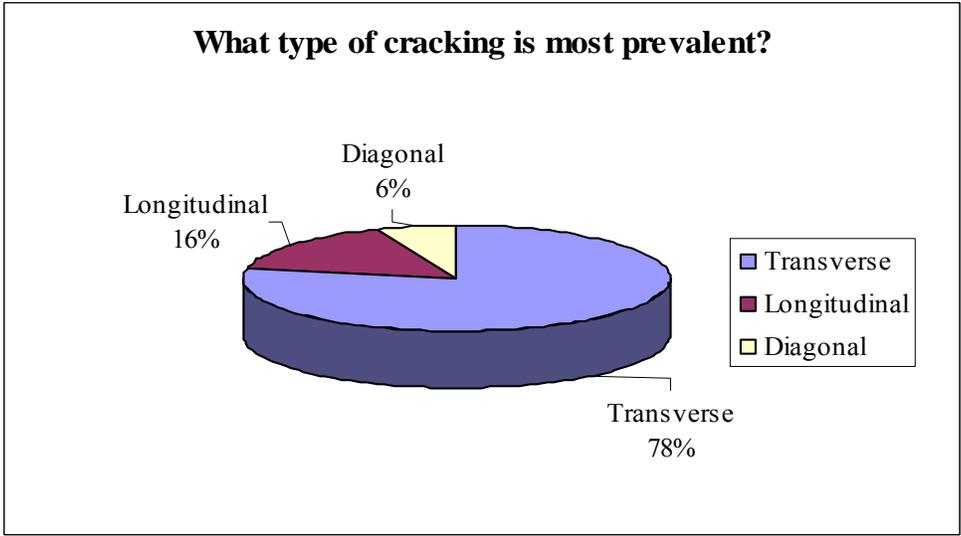


Figure 3-4. Frequency of crack types observed on bridge decks

Sixty-eight percent of the respondents reported that they are satisfied with the service life of concrete bridge decks under their jurisdiction. Most states identified the duration of service life of the RC bridge deck ranging from 15 to 60 years. As shown in Figure 3-6, 52 percent of the respondents identified the average service life as 30 to 40 years.

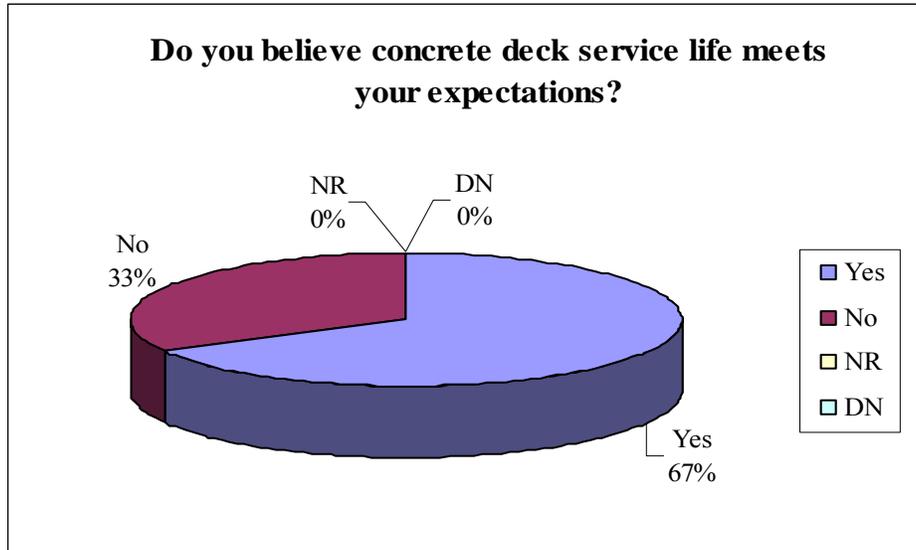


Figure 3-5. Expectations on bridge deck service life

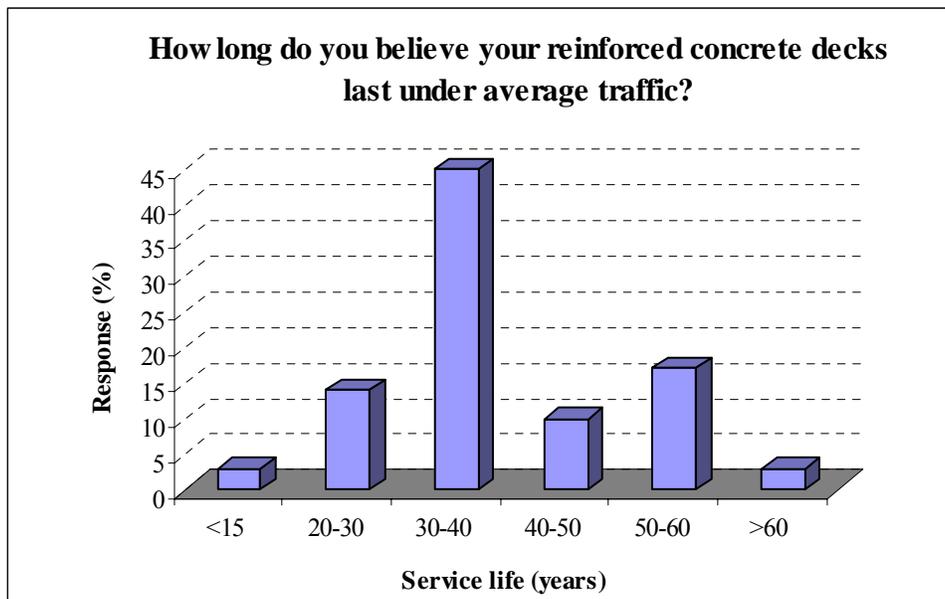


Figure 3-6. Frequency of expected service life of bridge decks under average traffic

After compiling the responses to Question 3 (Figure 3-7), the most prevalent actions taken to improve the durability of bridge deck are:

1. Increased reinforcement cover
2. Changed material
3. Changed reinforcement design

4. Changed mix design
5. Changed curing procedure
6. Increased deck thickness

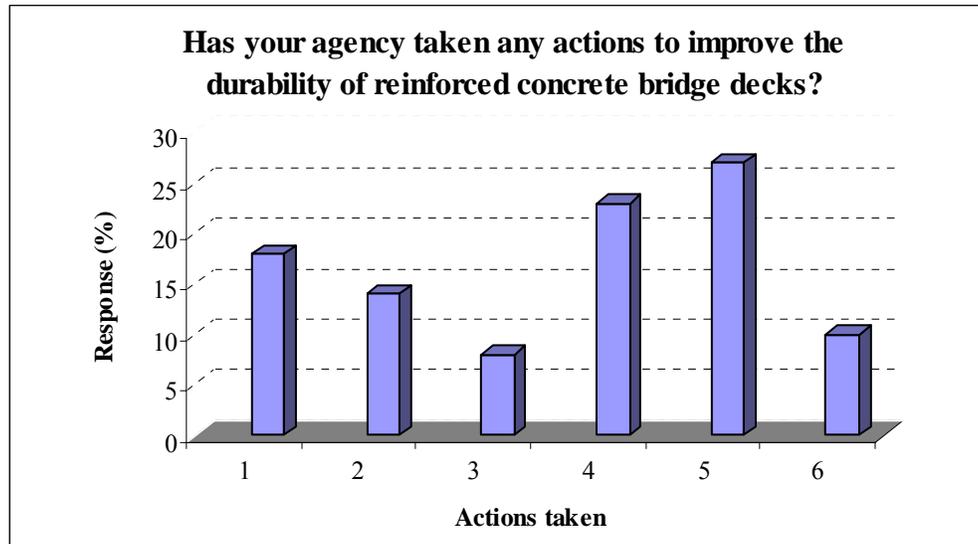


Figure 3-7. Frequency of different actions taken to improve bridge deck durability

The most commonly used mineral additives are fly ash (FA), silica fume (SF), and ground granulated blast-furnace slag (GGBS). Most of the states are using air entrainment (AE) and set controlling (R) admixtures. Some states are using mid range (MRWR) and high range (HRWR) water reducers. The percent of usage of different types of mineral additives and chemical admixtures among the responding states are shown in Figure 3-8 and Figure 3-9.

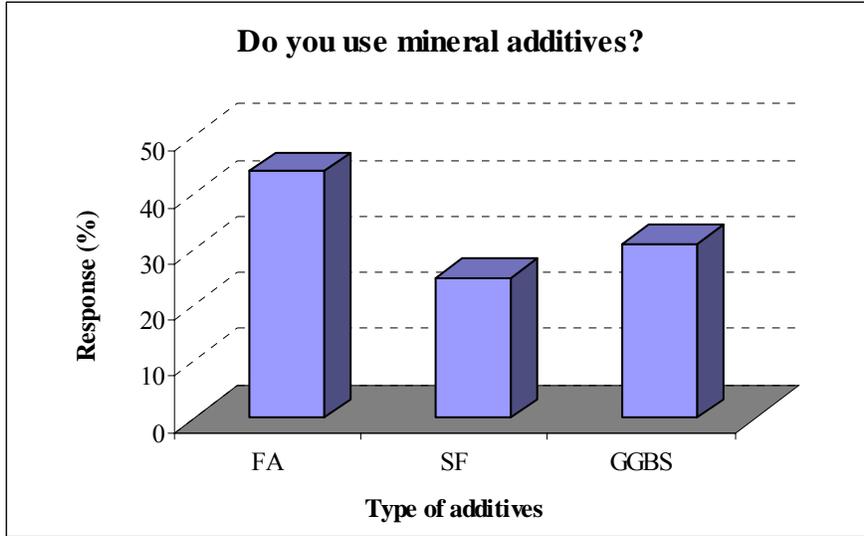


Figure 3-8. Frequency of using different types of mineral additives in concrete mix

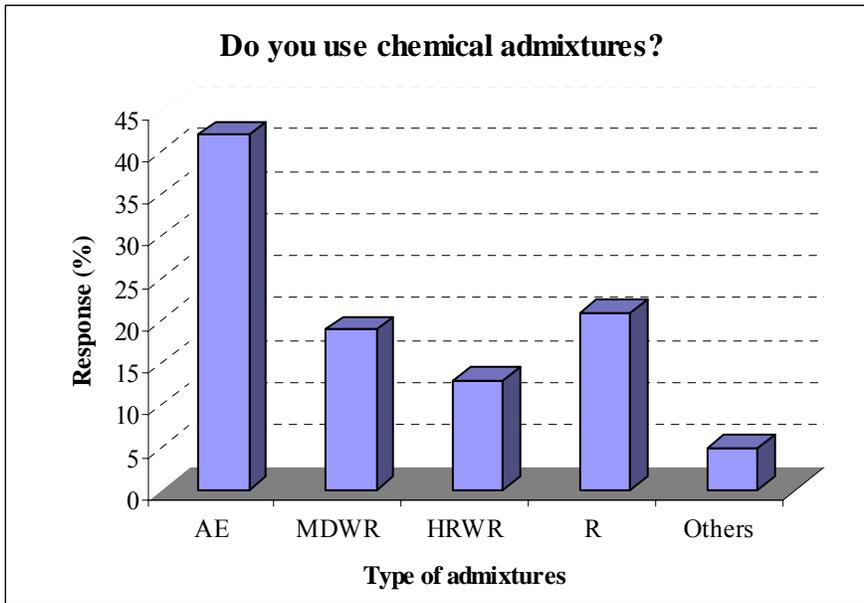


Figure 3-9. Frequency of using different types of admixtures in concrete mix

According to the survey response to Question 4 (Figure 3-10), the most commonly used top cover of RC bridge decks is 2.5 inches. There are a few states using 2- or 3-inch cover. Total deck thickness has varied from state to state ranging from 7 to 10.5 inches. A majority (92 %) indicated that different depth for top cover and total deck thickness have been used in the past. Most of the respondents have no knowledge of historical deck standards for top cover and total deck thickness.

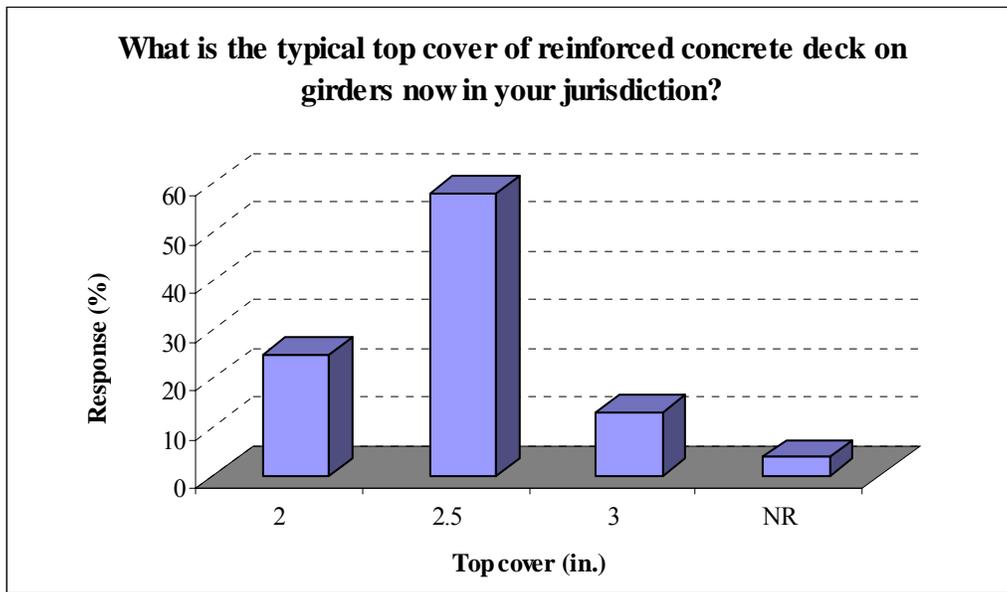


Figure 3-10. Frequency of using different top cover on reinforced concrete decks

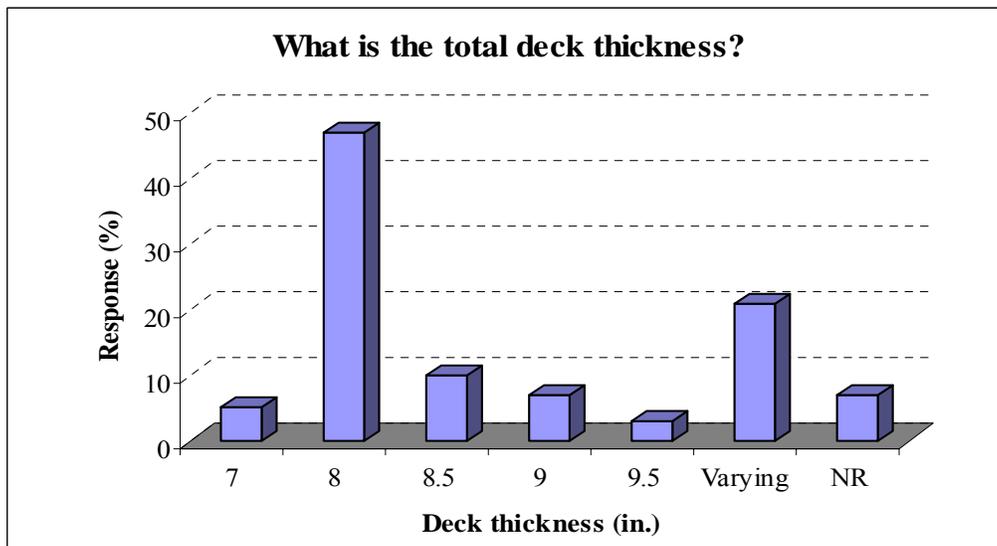


Figure 3-11. Frequency of using different total deck thicknesses for bridge decks

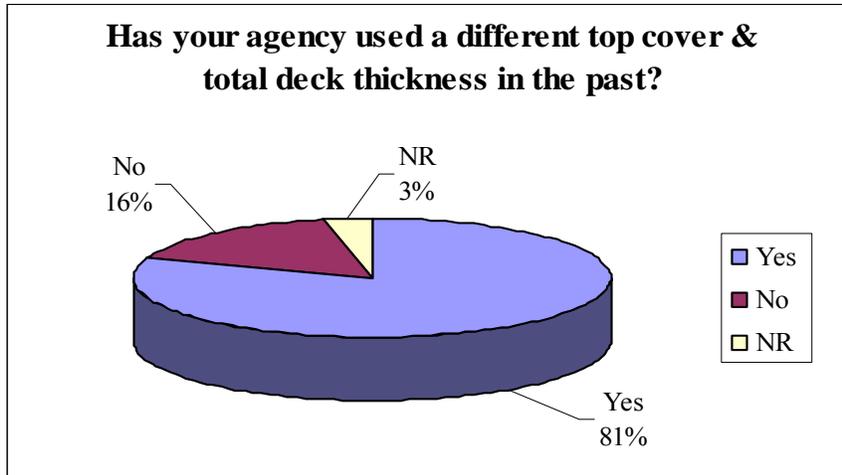


Figure 3-12. Frequency of using different top cover and total deck thickness in the past

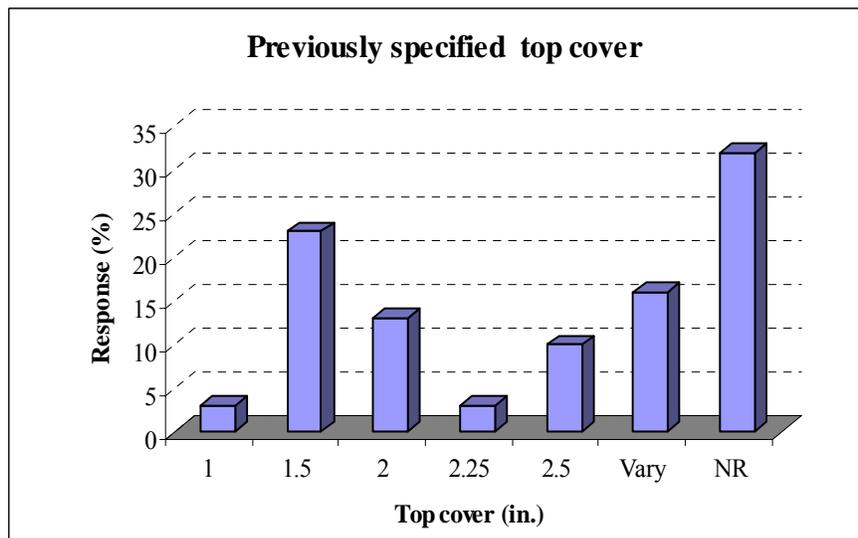


Figure 3-13. Frequency of using different top cover for bridge decks

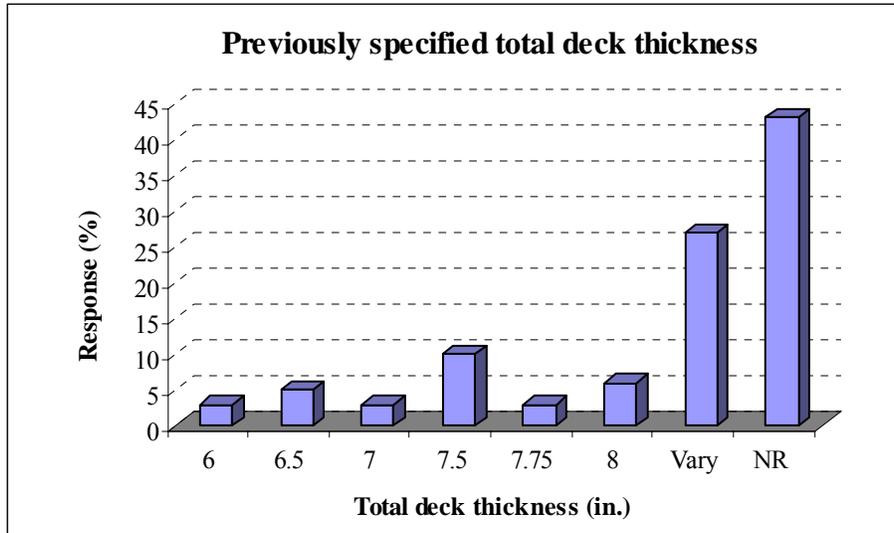


Figure 3-14. Frequency of using different deck thickness in the past

Epoxy coated reinforcements are the most commonly used among the responding states. Percentages of usage of different types of reinforcement such as epoxy coated, black, galvanized, and stainless steel among the states are presented in the Figure 3-15.

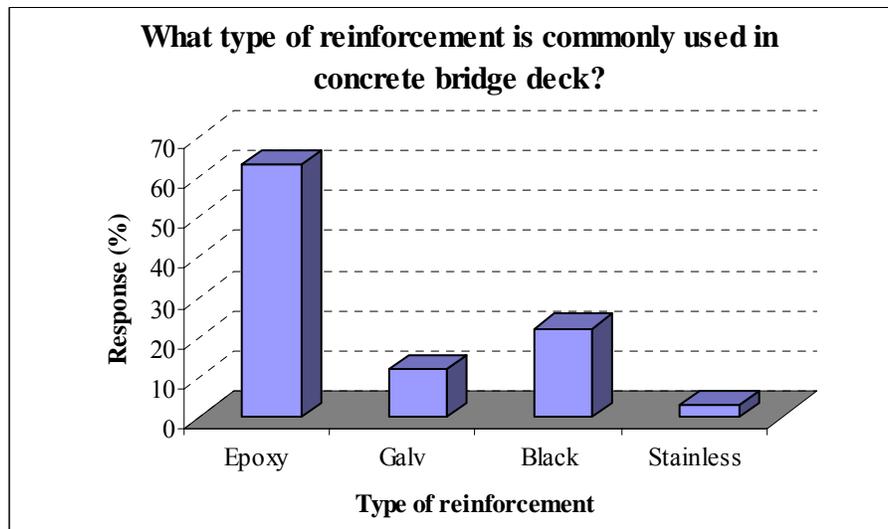


Figure 3-15. Frequency of using different reinforcement types on bridge decks

The most commonly used curing procedure is continuous wet curing. Other methods used are curing by burlap cover, air curing, and curing compound (Figure 3-16). The recommended duration for wet curing is seven days (Figure 3-17).

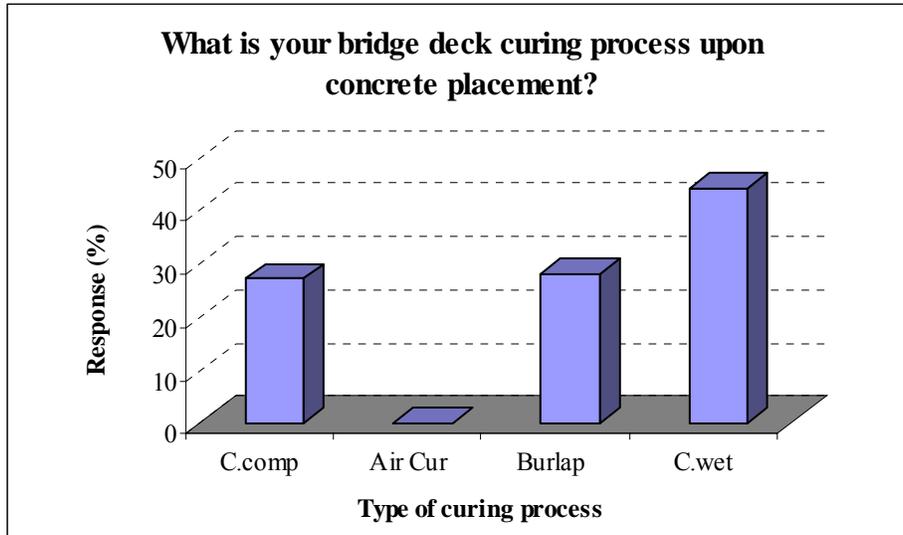


Figure 3-16. Frequency of using different curing methods upon concrete placement

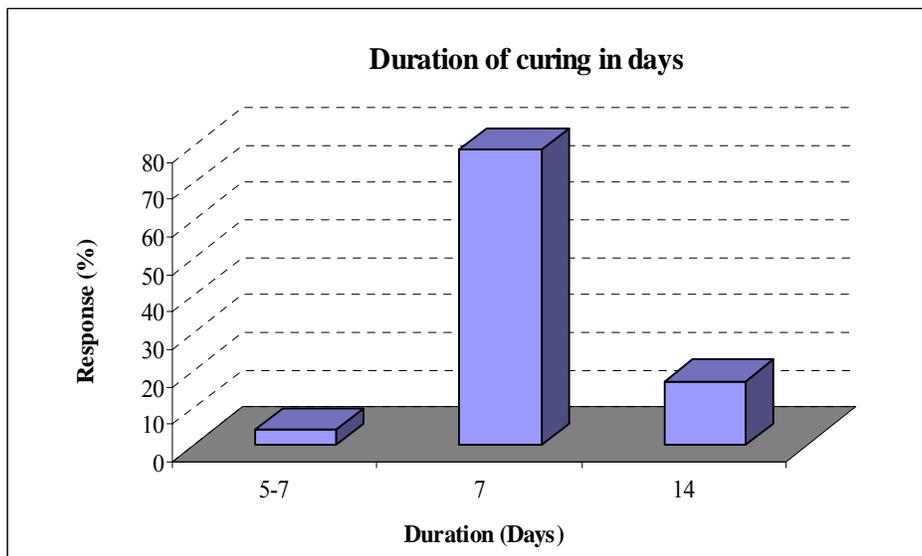


Figure 3-17. Frequency of using different curing durations

The causes of early-age bridge deck cracking identified by the respondents are:

1. Substandard curing
2. Construction practice
3. Mix design
4. Thermal stresses
5. Structure type
6. Epoxy reinforcements
7. Restraints

The percentage of responses for the causes of bridge deck cracking has been calculated based on the frequency of responses. In the survey questionnaire (Question 7), respondents were asked to identify the top three causes of early-age bridge deck cracking only, and not to categorize the causes according to their occurrence.

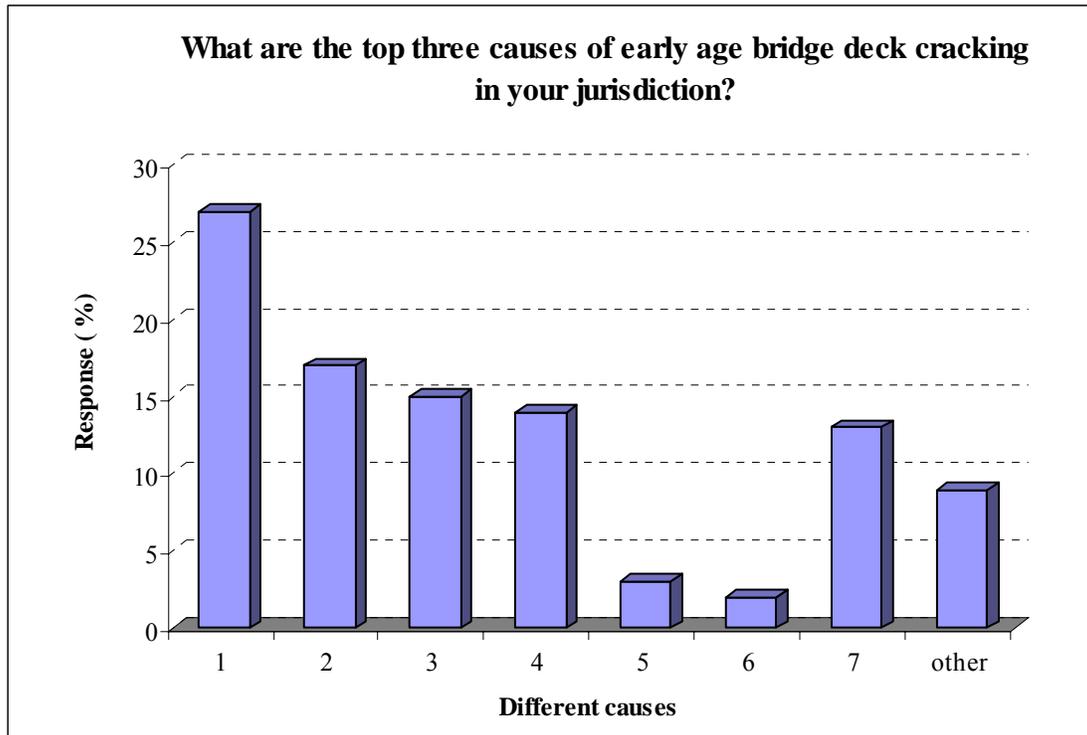


Figure 3-18. Frequency of top three causes of early-age bridge deck cracking

According to the compiled survey data, 52 percent of the responding states use a cement content of 7 sacks/yd³ and 32 percent use 6 sacks/yd³ (Figure 3-19).

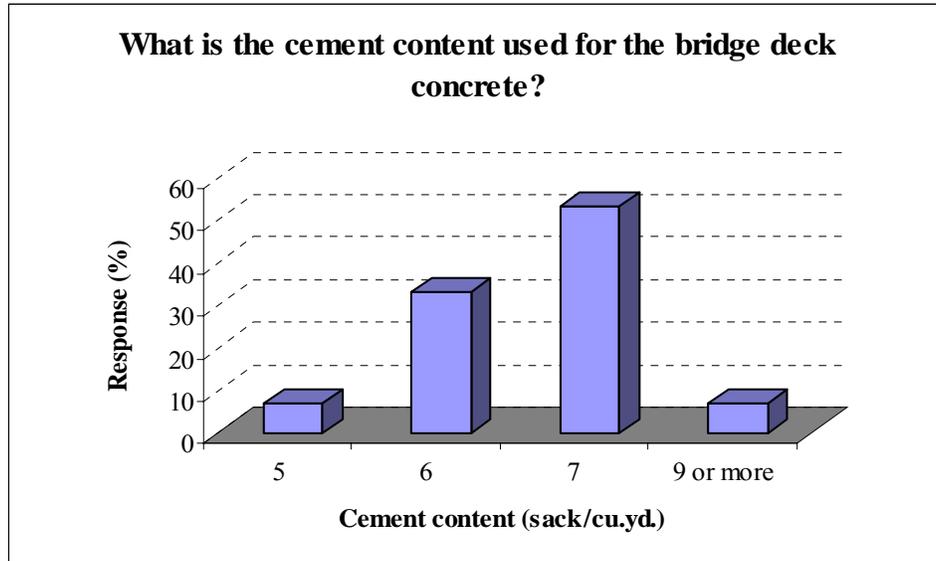


Figure 3-19. Frequency of using different amounts of cement in concrete mix

3.3 CONCLUSION

An extensive amount of information has been extracted regarding the early-age cracking problem of concrete bridge decks, materials, construction practices, and measures taken to minimize early-age cracking of concrete decks.

All the responding states (except Hawaii) acknowledged that they have experienced early-age deck cracking. Sixty-six percent of the responding states including Michigan observed cracking within the first few months of construction. The most prevalent type of cracking observed on decks is transverse cracks, which is similar to Michigan's observation. Sixty-eight percent responded that the average deck service life meets their expectations, unlike the state of Michigan. Most of the responding State DOTs including Michigan emphasized changing the mix design and increasing the reinforcement cover in order to improve the durability of decks. Forty-five and twenty-nine percent of responded states have been using fly ash and ground granulated blast-furnace slag, respectively in their mix design. In Michigan deck overlay mixes include mineral additives. The most popular type of admixture is air entrainment, which is similar to Michigan practices. Fifty-eight percent of the responding states have been using a top cover of 2.5-inch, but MDOT utilizes a 3-inch top cover. The most often used (45%) dimension for deck thickness is 8-inch, and MDOT has been using 9-inch decks. The most prevalent type of reinforcement is epoxy coated, 62% of responding states including Michigan uses this type. Continuous wet curing is the most popular method of curing specified by responding DOTs. The State of Michigan's curing practices are to use curing compounds and continuous wet curing for 7 days. The most predominant causes of early-age deck cracking are substandard curing, construction practices, and mix design analogous to State of Michigan. Fifty two percent of the responding states including Michigan have been using 7sacks/yd³ of cement in concrete mix design for decks.

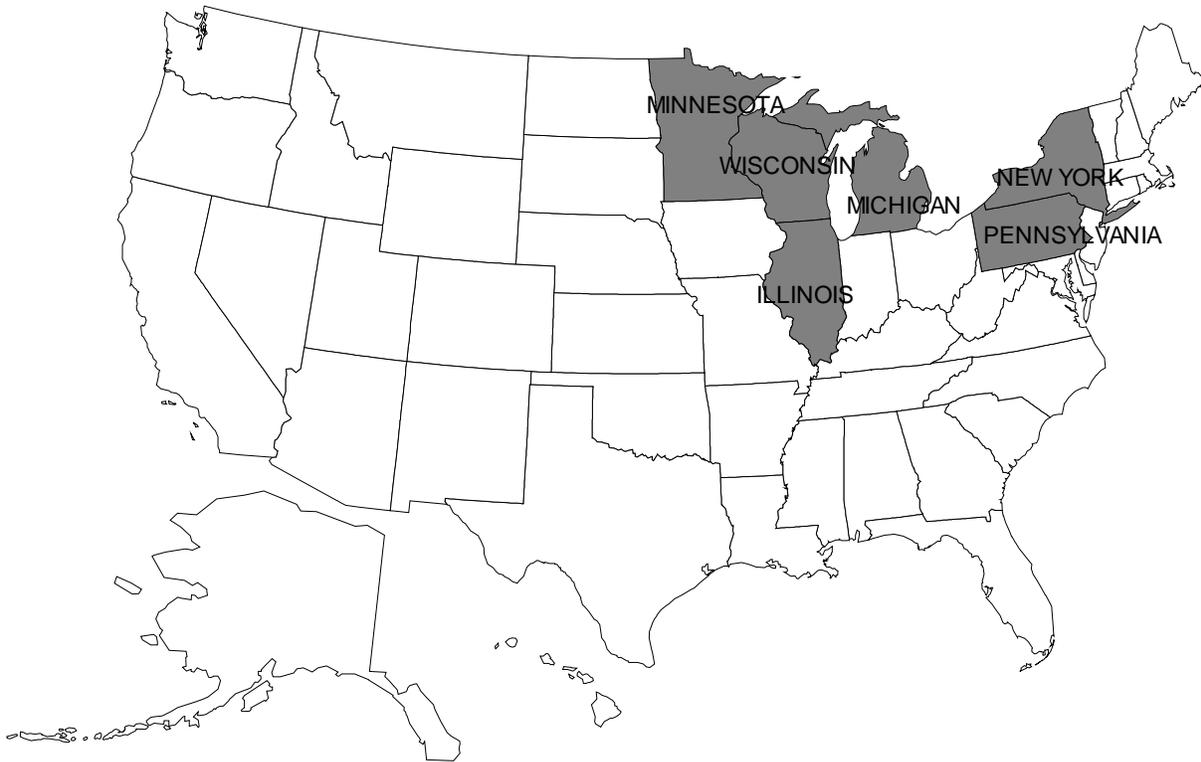


Figure 3-20. States with similar climatic exposures to Michigan

The information extracted from the nationwide survey was also compared specifically to the States of the Central North East Region (Illinois, Minnesota, New York, Pennsylvania, and Wisconsin) because they have similar weather exposure to Michigan. These states identified that they have experienced early-age cracking on concrete bridge decks during the first few months of service, and the prevalent type of cracking is transverse cracking. According to the responses, Illinois and Minnesota replied that their bridge deck service life meets their expectation, which is more than 35 years. But in case of Michigan, New York, Pennsylvania, and Wisconsin, the bridge deck service life is less than 35 years, which does not satisfy the expectations. The most popularly taken measures among these states in order to improve the durability of decks are increased reinforcement cover, changed material, changed mix design, and altered curing procedure. Illinois, New York, and Pennsylvania have been using fly ash (FA), silica fume (SF), and ground granulated blast-furnace slag (GGBS) as mineral additives in their mix design. Michigan has been using FA and SF and Wisconsin has been using FA and GGBS. New York, Minnesota, and Wisconsin responded that the chemical admixtures used are retarder (R), air-entrainment (AE), and mid range water reducer (MRWR), where as Illinois, Michigan, and

Pennsylvania are using only AE admixture. The most often-used deck thickness among the states is 8 inches, but Michigan and Minnesota's practice is to use 9-inch deck. Among these five states, epoxy coated reinforcements are very common, which is similar to Michigan. The State of New York has also been using black and galvanized reinforcement along with epoxy coated. The common curing practice among these states is continuous wet curing with the exception of Illinois. The top three causes of cracking identified by the respondents are substandard curing, construction practice, and mix design, which cause volume change of concrete due to thermal and shrinkage effects. The identified top three causes of cracking are also parallel to Michigan.