Report No. CDOT-DTD-R-2003-3
Final Report

ASSESSMENT OF THE CRACKING PROBLEM IN NEWLY CONSTRUCTED BRIDGE DECKS IN COLORADO

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DOT
DEPARTMENT OF TRANSPORTATION
RESEARCH

March 2003

COLORADO DEPARTMENT OF TRANSPORTATION
RESEARCH BRANCH

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
CDOT-DTD-R-2003-3		

4. Title and Subtitle ASSESSMENT OF THE CRACKING PROBLEM IN NEWLY CONSTRUCTED BRIDGE DECKS IN COLORADO	5. Report Date March 2003 6. Performing Organization Code CDOT-DTD-R-2003-3
7. Author(s) Yunping Xi, Benson Shing, Naser Abu-Hejleh, Andi Asiz, Suwito, Zhaohui Xie, Ayman Ababneh	8. Performing Organization Report No.
9. Performing Organization Name and Address Colorado Department of Transportation - Research 4201 E. Arkansas Ave. Denver, CO 80222	10. Work Unit No. (TRAIS) 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Colorado Department of Transportation - Research 4201 E. Arkansas Ave. Denver, CO 80222	13. Type of Report and Period Covered 14. Sponsoring Agency Code 84.20

15. Supplementary Notes
Prepared in cooperation with the US Department of Transportation, Federal Highway Administration

16. Abstract
Early age cracking on concrete bridge decks has been experienced by many DOTs. In Colorado the cracking problem on newly constructed bridge decks has not been completely solved. In this study, the extent and causes of the cracking problem were investigated, and necessary changes to alleviate the cracking problem were identified. To achieve these goals, current CDOT practice was reviewed and compared with other DOTs practices for construction of bridge decks. A database analysis of field inspection results was performed. Presently, 18% of newly constructed bridge decks in Colorado have no early cracking problem, and the rest (82 %) have various degrees of cracking problems. The results of the database analysis were then confirmed by field inspections conducted on nine newly constructed bridge decks. An extensive literature review was performed. Past research activities in Colorado were reviewed, current CDOT practices for controlling the bridge deck cracking problem were assessed, and recommendations to alleviate the cracking problem were identified. From the literature survey and previous research studies, the causes of cracking in newly constructed bridges can be categorized as material, design, construction, and environmental factors. High early age shrinkage of concrete is a major contributor among the adverse material factors. The cracking can also be a direct result of several structural design factors. Based on the results of field inspections, the cracks typically form above supporting members, such as girders or piers, and have large and uniform spacing. This is a result of tensile stresses from negative bending moments in the deck above supports. The construction and environmental factors (such as wind speed, temperature, and curing procedures) greatly affect the shrinkage of the concrete. Due to very limited information regarding deck curing or placement conditions, it has not been possible to determine the specific impact of these factors on early age deck cracking of the inspected bridges.

Implementation:
Recommendations to reduce the cracking problem in newly constructed bridges in terms of materials, design, and construction factors are presented. Based on the findings of Research Report No. CDOT-DTD-R-2001-11, new concrete mix designs Class H and Class HT have been implemented in the 2003 CDOT Standard Specification for Road and Bridge Construction. Most of the recommendations for construction of bridge decks have also been implemented in the 2003 CDOT specifications. The study recommendations for the structural design factors of bridge decks should been implemented in CDOT Bridge Guidelines

17. Keywords	18. Distribution Statement
concrete bridges, early age cracking, concrete mix design, concrete durability	No restrictions. This document is available to the public through the National Technical Information Service 5825 Port Royal Road Springfield, VA 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
None	None		

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Report No. CDOT-DTD-R-2003-3
Final Report

Prepared by
The Colorado Department of Transportation
Research Branch

March 2003

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Acknowledgements

The financial support provided by the Colorado Department of Transportation for this study under PG HAA 2000HQ00419 00HAA00069 (Task Order #3) is gratefully acknowledged. Partial financial support under NSF grant CMS-9872379 to University of Colorado at Boulder is gratefully acknowledged. Partial financial support under NSF grant ACI-0112930 to University of Colorado at Boulder is gratefully acknowledged. Partial financial support provided by the FHWA and Colorado Department of Transportation under FCU-CX 083-1(049) 00HAA00069 (Task Order #2) is gratefully acknowledged.

The writers would like to express their thanks to the Research Branch of CDOT for continuous support and encouragement throughout this study, and specifically to Skip Outcalt, Ahmad Ardani and Richard Griffin of CDOT Research Branch; Michael McMullen of CDOT Bridge Division; Greg Lowery of CDOT Staff Materials Branch; and Matt Greer of FHWA for their valuable suggestions and inputs. Special thanks to the graduate students and undergraduate students who helped on concrete testing, distributing and collecting questionnaires, and discussing current practice of concrete bridge decks at other DOTs: YoungSook Roh, William Phaup, Jessa Ellenburg, John Frazee, and Russ Hermanson.

Executive Summary

Early age cracking on concrete bridge decks has been experienced by many DOTs. In Colorado the cracking problem on newly constructed bridge decks has not been completely solved. There is a need to find the causes of cracking and study how to reduce premature cracking. The objectives of this study are: (1) to determine the extent and causes of the cracking problem in newly constructed bridge decks in Colorado, and (2) to identify necessary changes in the material properties, construction processes, and design specifications in order to alleviate the bridge deck cracking problem, thereby making bridge decks more durable.

To achieve these objectives, current CDOT practice was reviewed and compared with other DOT's practices for construction of bridge decks. A database analysis was conducted on field inspection results collected in 2002 on the extent of bare deck cracking problem in 72 bridges built by CDOT from 1993 to 2002. The database analysis was then confirmed with field inspections conducted on nine newly constructed bridge decks that showed excessive cracking. A broad literature review on related topics was performed including past research activities in Colorado. Current practice at CDOT for controlling the bridge deck cracking problem was assessed, and recommendations to alleviate the cracking problem were provided.

Based on the information collected from the CDOT database, presently, 18% of newly constructed bridge decks in Colorado have no early cracking problem, and the rest (82 %) have various degrees of cracking problems. According to an NCHRP Study (NCHRP Report 380) and various FHWA publications, the acceptable crack width from a corrosion and durability standpoint is between 0.004 in and 0.008 in. (0.1 and 0.2 mm). The widths of cracking observed on the inspected bridges in this report, however, vary from 0.01 to 0.10 in. (0.25 to 2.5 mm) in width. These cracks are usually severe, widespread, and spaced at a relatively uniform interval. Typically, they are oriented in the transverse and/or longitudinal directions. Occasionally, the cracks can form in random orientations. The cracks with widths larger than 0.004 to 0.008 in. have significant effects on permeability of concrete. Even with corrosion inhibitors applied in the concrete, the initiation of the corrosion of embedded rebars will be considerably accelerated.

From the literature survey and previous research studies, the causes of cracking in newly constructed bridges can be categorized as material, design, construction, and environmental factors. High early age shrinkage of concrete is a major contributor among the adverse material factors. Cracking can also be a direct result of several structural design factors. The cracks typically form above supporting members, such as girders or piers, and have large, uniform spacing. This is a result of tensile stresses from negative bending moments in the deck above supports. The construction and environmental factors (such as wind speed, temperature, and curing procedures) greatly affect the shrinkage of the concrete. Due to very limited information regarding deck curing or placement conditions, it has not been possible to determine the specific impact of these factors on early age deck cracking of the inspected bridges.

Several recommendations to alleviate the cracking problem have been discussed. Some of these have already been implemented in the CDOT specification, such as concrete mix designs and curing procedures. However, further studies are needed to investigate the effects of structural parameters and deck construction techniques on cracking of newly constructed bridge decks.

Implementation Statement

Recommendations to reduce the cracking problem in newly constructed bridges in term of materials, design, and construction factors are provided in Chapter 6. Most of the recommendations for material factors have been implemented in recent research projects, such as the project of I-225 Parker Rd. interchange and the project of O'Fallon Park Bridge. Among all recommendations for material factors, the use of ground granulated blast-furnace slag (GGBFS) in concrete mixtures, the early age strength test (at 1, 3, and 7 days) and drying shrinkage test have not been considered in the current practice.

New concrete mix designs Class H and Class HT were discussed in this study. Both of these mix designs have been implemented in the new CDOT specification. Class H concrete is used for bare concrete bridge decks that will not receive a waterproofing membrane. Class HT concrete is used as the top layer for bare concrete bridge decks that will not receive a waterproofing membrane.

The recommendations for design factors have not been considered in the current practice. These recommendations should be considered together with structural design requirements, written as a special note, and be available for bridge design engineers.

Most of the recommendations for construction practices have been implemented. The placement sequences recommended by NJ-DOT have not been considered. More studies should be performed to verify the effectiveness of the placement sequences before actual implementation in Colorado.

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1. Introduction

Many concrete bridge decks in the United States cracked soon after construction. From a survey of 52 states, it is estimated that more than 100,000 bridges developed early transverse cracking. Deck cracking has been reported in a variety of geographical locations and climates (McDonald et al, 1995). Cracking typically occurs before the concrete is one month old. These cracks are typically transverse, full depth, and spaced one to three meters apart.

Cracking may accelerate corrosion of reinforcing steel, deterioration of concrete, damage to structural members and components beneath the deck, and result in appearance concerns. Generally, it reduces durability of structures.

It is well accepted that the following mechanisms are responsible for bridge deck cracking:

Plastic shrinkage: Soon after placement and before curing of concrete, a plastic concrete surface may crack because of shrinkage of the concrete and restraints provided by the concrete mass. Rapid evaporation of surface moisture is the cause of the shrinkage.

Subsidence of plastic concrete: Fresh concrete subsides after finishing and during the bleeding period when the water rises to the surface. Horizontal reinforcement in the deck resists this subsidence, resulting in the cracking over and parallel to the reinforcement.

Thermal shrinkage of hardened concrete: Temperature of concrete rises during the curing process due to the release of heat of hydration. This initial temperature rise and tendency to expand do not induce residual stresses in the concrete because of its extremely low modulus of elasticity at this state. During the hardening process, the concrete cools down to the ambient temperature, and at the same time, longitudinal beams restrain the deck shrinkage induced by the cooling, which causes tensile stresses and thus transverse cracking in the deck. These cracks are usually formed above the uppermost transverse bars and are full depth.

Drying shrinkage of hardened concrete: After curing and exposure to the atmosphere, concrete bridge decks lose some original mix water to the environment and tend to shrink. As longitudinal beams restrain the shrinkage, transverse cracks occur. Drying shrinkage cracks are usually formed above the uppermost transverse bars and are full depth or partial depth.

Autogenous shrinkage: It is the macroscopic volume reduction of cementitious materials when cement hydrates. It does not include the volume change due to loss or ingress of substances, temperature variation, the application of an external force and restraint. The lower the w/c ratio is, the greater is the relative importance of autogenous shrinkage, as compared with drying shrinkage.

In the state of Colorado, results of the field inspection organized in 1997 by FHWA division office showed that the cracking problem of concrete bridge decks has remained unsolved. Recently (October 1999), CDOT concluded a study entitled "Cracking in Bridge Decks: Causes and Mitigation." (CDOT Report 99-8). In this study, a limited survey was conducted on seven newly constructed bridges to examine the extent of cracking in concrete decks that were constructed with different concrete mix designs and curing procedure currently used by CDOT. This study conlcuded that:

- The recently adopted curing procedure by CDOT reduced the extent of the bridge deck cracking.
- Light doses of silica fume will not increase cracking if suitable curing procedure is applied.

However, the current database is limited and more information on the performance of newly constructed bridge decks is needed. It is suggested that the performance of selected new bridge decks constructed with new and existing concrete mixes be systematically monitored over a duration of one year or more to assess the severity of the deck cracking problem. The present study was developed to address two objectives:

- Determine the extent and causes of the cracking problem in newly constructed bridge decks in Colorado with different concrete mixes.

- Identify necessary changes in the material properties, construction processes, and design specifications in order to alleviate the bridge deck cracking problem, thereby making bridge decks more durable.

To fulfill these objectives, several tasks were performed in this study and the results are summarized in six chapters and six appendices. The following is a short summary of each chapter and appendix.

Chapter 2: CDOT Practice for Construction of Bridge Decks and Comparison with Practice at Other DOTs. This Chapter describes CDOT construction and material specifications for bridge decks as they appeared in CDOT specifications in 1999, concrete mixes used in neighboring DOTs for bridge decks, and lessons learned from their experience for CDOT to consider.

Chapter 3: Extent of the Cracking Problem in Colorado in Newly Constructed Bridge Decks. This chapter summarizes field inspection results collected in 2002 on the extent of the bare bridge deck cracking problem in 72 bridge structures built by CDOT from 1993 to 2000. These results are analyzed to determine the extent of the bridge deck cracking problem in Colorado. In order to confirm these results, nine bridges was selected and inspected by the research team. The detailed results are included in this chapter.

Chapter 4: Literature Review. The main factors that contribute to early deck cracking, the degree of their influences, remedial considerations, and new technologies are summarized. In addition, detailed background information is provided in order to help the readers to have a better understanding on the damage mechanics in concrete decks and specific functions of the recommended remedial considerations.

Chapter 5: Summary of Past Research Activities in Colorado Regarding the Bridge Deck Cracking Problem. This chapter assesses CDOT current practice for controlling the bridge deck cracking problems, summarizes possible causes for the bridge deck cracking problem in Colorado, and identifies recommendations to alleviate this problem.

Chapter 6: Recommendations. After reviewing CDOT's 1999 practice for construction of bridge decks as presented in Chapter 2, recommendations are presented in this chapter for CDOT to consider for future construction projects. These recommendations are based on reviewing practices and experiences of other DOTs, inspection results of the bridge cracking problem in many newly constructed bridge decks in Colorado, the extensive literature review, and past CDOT research findings.

Chapter 7: Implementation Update. This chapter summarizes the recommendations that were recently implemneted in CDOT standard specifications for construction of bridge decks, as well as those that have not been impelmented.

Chapter 8 lists all references.

Appendix A: A part of CDOT's 1999 specifications for construction of bridge decks with concrete mixes D and DT.

Appendix B: A part of CDOT's 1999 standard special provision on projects that include concrete Class SF bridge deck overlays.

Appendix C: A part of the revised CDOT 2003 standard for construction of bridge decks with concrete Classes H, HT, D and DT.

Appendix D: Basics of the ultrasonic test and interpretation of the test data. Only one example of the inspected bridges is shown in the appendix. The complete results of ultrasonic analyses can be seen elsewhere (Xi et al. 2003).

Appendix E: Crack surveying results. Only a part of the results are shown in the appendix. The complete inspection results can be seen elsewhere (Xi et al. 2002; Xi et al. 2003).

Appendix F: Forensic investigation of the bridge deck cracking problem at the I-70 structure over Clear Creek at Hidden Valley. The results of the investigation were summarized in the appendix. Complete analysis of the results can be seen elsewhere (Xi et al. 2001b).

2. CDOT Practice and Comparison with Practices of Other DOTs

2.1 CDOT Specifications for Construction of Bridge Decks

This section describes CDOT's construction and material specifications for bridge decks as they appeared in CDOT specifications in 1999. Appendix A lists the portion of the 1999 CDOT standard materials and construction specifications for two concrete mixes (Classes D and DT), which were utilized by CDOT for construction of bridge decks on a routine basis. Appendix B summarizes the CDOT 1999 standard special provisions for projects that use Class SF bridge deck overlays.

There are no accurate statistics about the percentages of each class of concrete mix being used for construction of bridge decks in Colorado. Our interview with CDOT personnel showed that almost all recently built bridge decks were made with Class D concrete, and about 25% of concrete decks received a topping of Class SF concrete. Class DT is normally used for rehabilitations, although a few decks might have been topped with Class SF. Seventy one of bare decks were constructed in Colorado from 1993 to 2000. Sixty two of the decks were built with Class D only, and nine of them with Class D and overlays (Class SF most likely). The minimum thickness for the decks is 8.0”. The typical concrete cover from reinforcing steel is 2.5” for the top cover and 1” for the bottom cover. This cover does not include the thickness of overlay. CDOT also requires that for bare concrete deck slabs with a mechanical saw cut finish, the top layer of reinforcing shall be 3”.

2.1.1 Material Description of Bridge Deck Concrete

Table 2.1 lists the material requirements for concrete Classes D, DT, and SF.

Table 2.1 Requirements on concrete mixes for bridge decks

Concrete Class	Required 28 Day Field Compressive Strength (MPa)	(1) Cement Content Minimum or Range (kg/m^3)	Air Content % Range (Total)	Additional Requirements
D	30 (4500 psi)	365 to 400 615 to 660 (lbs/cu yd)	5-8	(1) (2) (3) (4)
DT	30 (4500 psi)	415 700 (lbs/cu yd)	5-8	(1) (2) (3) (4)
SF	40 (5800 psi)	400 660 (lbs/cu yd)	4-8	(1) (2) (3) (4)

Additional Requirements:

(1). The cement content tolerance of ±1% specified in AASHTO M 157 will be allowed.

(2). Classes D, DT, and SF require the use of an approved water reducing admixture.

(3). Bridge deck concrete shall have a maximum water/cement (w/c) ratio of 0.44. In determining the w/c ratio, the cement (c) shall be the sum of the weight of the cement and the weight of the fly ash.

(4). The slump of the delivered concrete shall not exceed the slump of the approved concrete mix design by more than 38 mm.

Class DT concrete shall contain a minimum of 50% coarse aggregate. The coarse aggregate shall be AASHTO M 43, size No. 7 or size No. 8.

The use of Class SF overlays started from about 1993. The mix has not been changed significantly since then, except the use of fibers was dropped in this mix.

Class D has been used the same way since about 1976, although w/c ratios in actual mixes have been decreased (it was 0.48 in 1976). Cement content had the minimum value lowered to 615 #/cy a few years back and had a maximum applied of 660 #/cy at the same time.

Before 1996 many post-tensioned bridges had decks of Class S, which had requirements on minimum air content, cement content, and maximum w/c ratio.

Class DT is a mix for deck topping intended for rehabilitation. The use of this mix started in 1980, although the first few projects used a mobile mixer rather than ready mix.

Additional notes:

- Portland Cement
    - Type I and Type II allowed
    - Type IP (blended cement) was tried in 1997-1998, and did not continue.

- Concrete strength
    - Compressive tests based on 28 day strength.

- Silica Fume
    - Not more than 7.5% by weight of cement

- Fly Ash
    - ASTM C 618 <u>(Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete)</u>
    - From approved source with documented results of ‘contaminant’
    - Not more than 25% by weight of cement.

2.1.2. Construction Requirements of Bridge Decks

Curing by Wet Burlap

AASHTO M 182

Minimum five-day cure time

Curing by Liquid Membrane Forming Compounds

AASHTO M 148

Curing by Sheet Materials

AASHTO M 171

Minimum five-day cure time

Placement Conditions

Temperature must be greater than 40 °F at deck surface

Temperature must be less than 90 °F

2.2 Practice for Construction of Bridge Decks at Other DOTs

Several neighboring states were contacted and asked about their experiences with the bridge deck cracking problem. Recommendations were solicited from these DOTs for CDOT to alleviate the problem. The following are the results.

2.2.1 Nevada DOT

A telephone interview was conducted with Bill Crawford, the Chief Bridge Engineer of Nevada DOT (Crawford 2002).

Severity of Nevada's cracking problem:

- 75% of all new bridges have a significant cracking problem

Current standards or measures used to prevent cracking:

- Contractor is responsible for providing a good deck. There are some penalties for poor deck production
- Curing compounds used in place of water
- 20% fly ash used in all paving concrete
- No Silica Fume used due to the high alkali-silica reaction

Current Rebar corrosion protection:

- Sand/salt mixture is used for de-icing
- Top and Bottom layers are epoxy coated (beginning in the early 1980's)
- Looking into applications of MMFX steel

Suggestions for CDOT consideration:

- Apply cure immediately after placement (saw-cut used for roughening the surface after deck placement is complete)
- Pay for Cure as a separate pay-item to avoid contractor skimping

2.2.2 Kansas DOT

A telephone interview was conducted with bridge deck engineer Dave Meagers of Kansas DOT (Meagers 2002).

Severity of Kansas' deck cracking:

- It was a problem, but it is now sporadic and only partial depth cracking occurring

Current standards and measures to prevent the cracking:

- Wet Burlap 7 day – very helpful (reduced cracking by 50%)

Corrosion protection

- Top and bottom epoxy coated rebars
- Black steel decks get 2 polymer coats during restoration

Suggestions for CDOT

- Silica Fume overlay but watch for a consistency problem with permeability test
- Use wet cure specifications
- Polymer overlays are good but use a heavy grit blast at #6 or #7

2.2.3 Utah DOT

Website information was collected from Utah DOT, since there has been no response to telephone calls.

Current Utah DOT Standards

- Type II cement only
- Low alkali cement defined by ASTM C 150
- Pozzolan – ASTM C 618 (Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Concrete)
- Silica Fume – ASTM C 1240 (Standard Specification for Use of Silica Fume for Use as a Mineral Admixture in Hydraulic-Cement Concrete, Mortar, and Grout)
- Concrete Curing Compounds:
    - AASHTO M 148 type 1D Class A for structural/architectural
    - AASHTO M 148 type 2 Class B for paving

Bridge Deck concrete

- Apply membrane curing compound such that concrete is not exposed to atmosphere more than 20 minutes

- Cure applied at uniform rate of $100ft^2$/gal.
- Cover deck with burlap as soon as the concrete sets enough to support it
- Keep moist for 7 days

2.2.4 Concrete Mix Designs Used in Other States

As a comparison, Table 2.2 lists the concrete mix designs used by other state DOTs.

Table 2.2 Concrete mix designs for bridge decks used by state DOTs (Xi et. al., 2001)

States	Cement (lb/yd^3)	Fly ash (lb/yd^3)	Silica fume (lb/yd^3)	w/(c+m)	28-d Strength (psi)	Permeability 28d (Coul.)	Air content (%)	Slump (inch)
Colorado Shing, P.B.et al, 1999)	660	-	50	0.35	5800	-	4-8	
Colorado	615-660	<61-66	-	<0.44	4500	-	5-8	
Illinois (Detwiler,1997)	630	-	70	0.31	6950 at 14d	540	6-8	-
New York (Alampalli,2000)	505	149	42	0.4			6.5	3-4
Washington (FHWA-RD-00-124)	660	75	-	0.39	4000 5300 at56d	2800	6.0	-
Nebraska Beacham, M. W. (1999)	750	75	-	0.31	8000 at 56d	589 at 56d	6.0	-
Texas (Ralls, M. L., 1999)	382-610	88-131	-	0.31-0.43	4000	<2000	5-8	3-9
New Hampshire (Waszczuk, C. M. et al, 1999)	607	-	45	0.383	6000 7200 at 56d	<1000 at 56d	6-9	3-5
Virginia (FHWA-RD-00-123)	560	140	-	0.45	5000	2500	-	-

3. Extent of the Early Deck Cracking Problem in Colorado

3.1 Database Analysis on the Results Reported by CDOT's Inspection Unit

CDOT's Bridge Inspection Unit inspects Colorado bridge decks for deterioration and cracking problems. The inspection results are documented in a database. From 1993 until 2000, a total of 72 bridge decks were constructed (62 bare decks and 9 with concrete overlays). These bridge decks were inspected in year 2002 and the results of this inspection are listed in this section. First, the concrete deck is inspected for spalls/delaminations. If the surface deck has no repaired areas and there are no spalls/delamination in the deck surface, then smart flag element 358 (deck surface cracking) is used. This element addresses specifically the extent of the cracking problem as described in the following:

Condition State 1: The surface of the deck is cracked, but the cracks are either filled/sealed or insignificant in size and density to warrant repair activities.

Condition State 2: Unsealed cracks exist which are of moderate size or density.

Condition State 3: Unsealed cracks exist in the deck which are of moderate size and density.

Condition State 4: Unsealed cracks exist in the deck which are of severe size and/or density.

The width and spacing of for cracks in concrete decks under each condition state is shown in Table 3.1. Note that the NCHRP criteria for acceptable crack width is to be smaller than 0.2 mm which is very difficult to fulfill. Item g of Section 601.15 of CDOT specification requires the contractor to repair any crack wider than 0.9 mm (0.035"). The condition states under elements 358 provide an overall description of the severity of the cracking problem by provisioning limits on cracks width and spacing.

Table 3.1 Condition states defined in smart flag element 358 (deck surface cracking)

CDOT SUGGESTED CONDITION STATES FOR CRACKS IN CONCRETE DECK					
		SPACING (S) IN METERS (FEET)			
WIDTH IN mm (in) (W)		S > 3 M (> 10 ft)	2 < S ≤ 3 (6)(10)	1 < S ≤ 2 (3)(6)	S ≤ 1 M (≤ 3 ft)
	≤ 1 mm (≤ 1/32 in)	1	1	2	3
	1 < W ≤ 2 (1/32)(1/16)	1	2	3	4
	2 < W ≤ 3 (1/16)(1/8)	2	3	4	4
	> 3 mm (> 1/8 in)	3	3	4	4

Table 3.2 The 2002 Inspection results of bare concrete decks

Decks Constructed in:	1993	1994	1995	1996	1997	1998	1999	2000	Total (%)
Number of structures	23	7	3	3	4	4	13	5	62
State 1 (Decks with no problems or repaired areas)	3	1	1		1	1	3	1	11 (18%)
Deck with delamination/spalling problem	5	1	1	2	2	2	7	3	23 (37%)
Decks with cracking problem, State 2	11	4	1	1	1	1	3	1	23 (37%)
Decks with cracking problem, State 3	4	1							5 (8%)

Table 3.2 summarizes the condition states of the 62 bridge concrete bare decks built by CDOT from 1993 to 2000. In this study, if the deck is rated under condition state 1 of element 358, it is called a deck with acceptable performance in terms of cracking/spalling/delamination, otherwise it is called a bridge deck with a problem in terms of cracking/spalling/delamination. The number of structures shown in this table may underestimate the real number of cracked structures because it does not include structures with sealed cracks. Table 3.2 indicates that the cracking problem with bridge decks decreased from 1993 to 1995, but since then they have been at almost the same level, suggesting that the problem developed in the first year of service. The extent of the spalling/delamination problem seems at the same rate for all years. Presently, 18% of Colorado bridge decks have no problems (repaired) and the rest (82 %) have problems ranging

from spalling/delamination (37%), unsealed cracks with moderate size *or* density (37%), or unsealed cracks with moderate size *and* density (5%).

Note that delemination and spalling problems with bridge decks could be caused by the cracking problem. Nine bridges with concrete overlays were constructed from 1993 to 2000. Only one bridge has no problems (11%), one with a spalling/delamination problem (11%), and five (56%) with unsealed cracks that are of moderate size *or* density, and two with unsealed cracks that are of moderate size *and* density.

3.2 Field Inspection for Newly Constructed Bridge Decks in Colorado

3.2.1 Crack Mapping Results from CDOT Research Report 99-8

The objective of the crack mapping was to determine length, width, shape, and location of the cracks in the selected zones on the bridge deck. From the durability standpoint, the acceptable crack width is between 0.1 mm and 0.2 mm according to a NCHRP study (NCHRP Report 380). Hence, our crack survey concentrated on cracks wider than 0.2 mm. The locations of both fine cracks and cracks wider than 0.2 mm on deck surfaces were determined. The widths of cracks wider than 0.2 mm were measured. Cracks were located by wetting the subject area. After the water had evaporated from the surface, the cracks were plainly visible. Each end of a crack was marked. For each end, the longitudinal and transverse distances from two reference locations were determined. Crack widths were visually determined using a crack comparator card. It should be noted that the reported crack widths are widths measured at the surface of the concrete deck, and that no measurements were taken to determine the crack depths or subsurface crack widths.

Material specifications, construction practice, and crack survey information have been collected for the following seven new bridge decks built with different representative concrete mixes (see CDOT Research Report 99-8 for more details):

1. 38th and Fox Avenue Bridge. The deck was placed in April of 1998 with Class D concrete mix. The crack survey was performed at the west half of the bridge deck, between piers 5 and 6, before it was open to traffic.

2. Founders/Meadows Bridge. The deck was placed in October of 1998 with Class D concrete mix. The crack survey was performed at the southern half of the bridge deck, before it was open to traffic.

3. Wolfensenburger Road Structure over I-25, Westbound. The deck was built in 1995 with silica fume, fly ash, and calcium nitrite added to Class D concrete mix. The crack survey was performed at the median shoulder only.

4. Wolfensenburger Road Structure over Plum Creek, Westbound. The deck was built in 1995 with Class D concrete mix. The crack survey was performed at the median shoulder only.

5. I-225 Structure over Colfax Avenue, Southbound. The overlay was placed in November of 1997 using Class DT with IP cement (see Chapter 5) concrete with Type F fly ash. The crack survey was performed at the outside shoulder only.

6. I-225 Structure over Tollgate Creek, Southbound. The overlay was placed in March of 1998 using Class SF concrete with Type F fly ash. The crack survey was performed at the outside shoulder only.

7. I-70 Structure over Box Elder Creek, Westbound. The overlay was placed in September of 1998 with Class DT concrete mix. The crack survey was performed at the median traffic lane, before it was permanently open to traffic (the lane had carried some traffic).

Transverse cracking was relatively minor in the first six bridge decks. In all these six decks, there was 10% fly ash in the concrete mixes. Furthermore, in these bridges, deck placement took place either in the evenings or in winter months with mild weather conditions and the air temperature between 40 and 80^{o}F. In two of the decks, it has been shown that light doses of silica fume will not increase cracking if suitable construction practices are implemented.

The seventh deck had a Class DT concrete mix with no fly ash for the overlay. Cracks in the overlay deck were wider than those in the first six decks. This could be partly attributed to the inadequate finishing operation. Inadequate finishing operations can lead to a considerable number of randomly oriented cracks limited to the surface of the bridge deck.

Furthermore, results of this survey indicate that the growth of cracks, especially longitudinal cracks, could be caused by a combination of several factors, such as the traffic load, the flexibility of the girders, and a smaller deck thickness.

A set of transverse cracks with relatively even spacing was noticed along the shoulder close to pier #5 of the 38th & Fox Avenue Bridge. To avoid the development of such cracks in similar conditions, the design engineer recommended the following: "When adding longitudinal steel to the top of a deck over piers, one should also add longitudinal steel to the bottom of the overhanging slab." However, these transverse cracks were relatively fine with a maximum width of 0.4 mm.

For the Founders/Meadows structure, the first inspection was performed 52 days after after deck placement. No cracking was observed. After the bridge deck was sand blasted and sealed, it was easier to see a few additional transverse cracks near the abutments. Soon after the traffic had been on the bridge, more longitudinal cracking was evident. On March 25, 1999, the project engineers observed quite noticeable cracking occurring parallel to the girders in a location where two girders butt up against each other. It was speculated that the differential deflections of the flexible girders under traffic load caused this type of cracking. The motion of the bridge deck could be witnessed when a large truck was driven over the deck. In addition, it was noticed that the girders lacked camber. This required minimizing the dead load by reducing the bridge deck thickness. A thinner deck is more vulnerable to cracking and the penetration of moisture. The above discussion suggests that the growth of cracks, especially longitudinal cracks, could result from the combination of several factors, such as the traffic load, the flexibility of the girders, and a smaller deck thickness. All these factors led to longitudinal shear cracks between adjacent girders in the inspected deck. In order to avoid the problems mentioned above, the design engineer suggested strengthening the joints between girders and adding a small amount of transverse unbonded post-tensioned steel in the deck.

3.2.2 Bridge Deck Cracking Results Reported by Colorado Highway Agencies

Colorado local highway agencies were asked through a questionnaire to report bridges with severe deck cracking in their areas. Tables 3.3 to 3.5 provide a brief summary of the information reported by local agencies regarding three bridges with severe concrete cracking. The last two bridges were field inspected in this study as will be presented in the next section.

Table 3.3 Information on Bridge 1

Name	Thornton high school pedestrian bridge
Location	Thornton Pkwy and Eppinger across Thornton high school
Construction Date	Not available
Problems	• Flange areas of the middle span have spalling and delaminating • Longitudinal cracking has developed in the flanges • The welded-wire-fabric reinforcement had extensive corrosion • Approximately 60 percent of the total reinforcement had been lost to corrosion • The pier supports of the middle span have developed cracking
Notes	Wiss, Janney, Elstner Associates, Inc. performed an evaluation in June 2000. They suggested further repairs.

Table 3.4 Information on Bridge 2

Name	Monument Creek bridge
Location	Colorado Avenue over Monument Creek
Construction Date	1998
Problems	• The structure exhibits an extraordinary amount of cracking in the bridge decks • Abutment 1, pier 2, pier 3, Abutment 4, bike lane showed areas with exposed aggregates and holes
Notes	City of Colorado Springs contacted the Engineer, CH2M Hill, and the contractor, Lawrence Construction, regarding the cracking problem in June 2000.

Table 3.5 Information on Bridge 3

Name	Colorado Avenue Viaduct bridge
Location	Colorado Avenue over railroad and Sierra Madre St.
Construction Date	1998
Problems	• Structure exhibits an extraordinary amount of cracking in the deck • Abutment 1, pier 2, pier 3, pier 4, pier 5, pier 6, Abutment 7 showed areas with exposed aggregates and holes
Notes	City of Colorado Springs contacted the Engineer, CH2M Hill, and the contractor, Lawrence Construction, regarding cracking in June 2000.

3.2.3 Field Inspection Conducted in this Study

Field Inspection Procedure

In the field survey conducted in the present study, one or two representative zones of 100 to 200 ft. long were chosen on each of the selected bridges. The field survey methods include parts or all of the following steps:

- Photos of the structure.
- Crack mapping as described in Section 3.2.1.
- Sounding test using chain-dragging to detect delamination of concrete decks. Sound of delaminated or hollow concrete can easily be detected if background noise (traffic noise) is low.
- Ultrasonic testing to detect severity of cracking in the deck and qualify of the concrete. At least three locations were selected for testing for each bridge. The basic principle of the ultrasonic test and interpretation of the test results are illustrated in Figs. D.1, D.2, and D.3 (Appendix D). The ultrasonic testing results were presented with the figures showing the time required (in microseconds) for ultrasonic pulses to travel between a transmitter and a receiver versus the distance between the transmitter and the receiver. Solid concrete of good quality exhibits low resistance to the ultrasonic pulses, and thus the arrival time of the signals is low. When a crack or void (in poor concrete) appears in the pathway of the signals, the arrival time will be considerably increased, represented by a deviation from the straight line in the figures. The results obtained from the bridges show that most of the points do not lie

in a straight line or there are variations in the arrival time. This indicates that the concrete deck is of variable quality or that cracks exist in the concrete within the region of the tests. An example of ultrasonic test results is presented in Appendix D. The complete results of ultrasonic analyses can be seen elsewhere (Xi et al. 2003).

- Coring was conducted for the selected bridges. At least two concrete cores were taken from each of the selected bridges, one from each direction, and one from the shoulder. The diameter of concrete cores was four inches, and the length of the cores was minimum 4 inches or deeper than the location of steel bars. The concrete cores were subjected to lab testing of chloride permeability or chloride profile, especially the chloride content of concrete near the surface of steel bar. The cores included a section of steel bar.

Field Inspection Results

Nine bridges were selected by the study panel for inspection. The following are the information of the nine bridges, including the name of the bridge, time built, and concrete mix design used. No. 7 and No. 8 are the two bridges managed by the city of Colorado Springs. There is no information available about the concrete mix designs used for the bridges.

1. SH71 & US24, summer 2000, Class D

2. US 285 S. Turkey Creek Rd. Bridge, summer 2000, Class D

3. Founders/Meadows & I-25 Bridge Near Castle Rock, summer 1998, Class D/FA

4. I-225 & Tollgate Creek Bridge, March 1998, Class SF

5. I-225 & Parker, Rd., Oct. 2001, Class D/FA

6. US 285 &Wolf Ave. Bridge in Aspen Park, Oct. 2001, Class D

7. Colorado Ave. Viaduct bridge in Colorado Springs, 1997, Project No. CSG-G.00-08.57

8. Monument Creek Bridge in Colorado Springs, 1998, Project No. CSG-F.99-08.57

9. I-70 structure over Clear Creek at Hidden Valley, summer 1999, Class D/FA

Only a part of the results are discussed here. Part of the inspection results are shown in Appendix E. The complete inspection results can be seen elsewhere (Xi et al. 2002; Xi et al. 2003). Photos of the structures and results of crack mapping are listed in Appendix E. In the crack mappings, the numbers marked next to cracks are crack width in 1/100 of inch. For example, 6 means 6/100 of inch.

For the SH71 & 24 structure, many transverse cracks are found on the decks. It was characterized by transverse fine cracks of 0.01 to 0.02 inch in width, typically spacing at 5 to 6 ft. More cracks in varying orientations were observed between 100 ft and 130 ft (mapping beginning from the south expansion joint), which implies that there was stress concentration in this area. The 2002 measured cracking condition states under elements 358 and 359 were 2, suggesting a moderate cracking problem. Concrete cores were taken from the bridge deck. The chloride permeability of the deck concrete, tested by the rapid chloride permeability test (ASTM C-1202-91) were 5566 coulombs for the traffic lane and 5707 coulombs for the shoulder. The permeability values are considered to be fairly high.

For the US 285 S. Turkey Creek Rd, the bridge suffered severe cracking, in terms of crack length, width and spacing. In addition, cracks in transverse, longitudinal and random orientations were developed and distributed. The longitudinal cracks mainly stretch along both edges of the lanes. It is important to notice that both of these bridges and the previous one (SH71 &24) used the same concrete mix design, Class D, and were constructed at about the same time. Cores were taken from this bridge. The chloride permeability of the deck concrete is 9866 coulombs for the traffic lane and 9829 coulombs for the shoulder, which are much higher than the permeabilities of the concrete in SH71& 24. From SH71 & 24 and this bridge, one can see that when crack width is small, the chloride permeability is low. Thus, the permeability may be used as an indicator for predicting the cracking potential of the concrete. Another example is the

I70 & Hidden Valley bridge (No. 9), in which severe cracking corresponds to high permeability values, as will be discussed later.

The results of crack mapping in CDOT Report 99-8 for the Founder/Meadows structure were discussed in Section 3.2.1. The 2002 measured cracking condition states under elements 358 and 359 were 2, suggesting a moderate cracking problem. In the present study, crack surveying was conducted on Nov.19, 2001. Major cracks appeared in all traffic lanes, shown as longitudinal cracks with almost equal spacing of about 6 ft stretching throughout the entire bridge length. There are five to six longitudinal cracks in all traffic lanes. The cracks are between traffic lanes, and probably on top of the girders. Between the section in 110 ft and 150 ft (with the east expansion joint as zero point), there are a lot of transverse cracks, which is thought to be related to the negative moment produced by the middle pier of the bridge. These observation support the reasons for cracking discussed in Section 3.2.1.

For the I-225 & Parker Rd, and US 285 & Wolf Ave. structures, no cracks were found on the surface of the deck within two months after deck placement. These two structures should be monitored in the future as suggested in the recommendation section.

Two adjacent bridges were inspected in Colorado Springs. One is Colorado Ave. Viaduct, and the other one is Monument Creek Bridge. For both structures, deck cracking occurred after the construction. The bridges were inspected in 1998 and 2000, and December 18, 2001 (see results of this inspection in Appendix E, Xi et al. 2002). It was observed that many transverse cracks were on the deck, with quite even spacing. There were curve-shaped cracks around the piers which may have resulted from the negative moment. It was noticed that the measured diffusion coefficients for Monument Creek Bridge were high (Xi et al. 2002), which is an indication of poor long-term durability of the concrete. These diffusion rates for the Monument Creek Bridge, four years old, were as high as those obtained from concrete bridge decks of 10 to 20 years old. This means that the concrete in the bridge decks of the two bridges in Colorado Springs do not have good durability property and this could be the reason for the severe cracking noticed in these structures.

For the I-70 bridge structure over Clear Creek at Hidden Valley, severe transverse deck cracking occurred after pouring of concrete decks on the bridge. The 2002 measured cracking condition states under elements 358 and 359 were 2, suggesting moderate cracking problem. A detailed investigation on the bridge was carried out in the summer of 2000. The summary of the report of the investigation is in Appendix F. The detailed report is in (Xi et al. 2001b). It seems that the early age shrinkage of the concrete used in the bridge deck is the major cause of the deck cracking.

4. Literature Review

CDOT Report 99-8 (Shing and Abu-Hejleh 1999) reviewed some recent literature on early age cracking in bridge decks. The present literature review is conducted based on CDOT Report 99-8. Additional details and recent progress in the related fields are included in this part of the report. The objectives of this literature were to assess CDOT's current practice for controlling the bridge deck cracking problems, summarize possible causes for the bridge deck cracking problem in Colorado and make recommendations to alleviate this problem.

Many factors play important roles in the formation of early cracks in concrete bridge decks. In the published results by researchers and professionals, there is inconsistency regarding major factors causing early age cracking of concrete decks. This is due partly to the fact that different agencies in different geographical regions utilized different concrete materials, different construction techniques, different design specifications, and partly to different environmental conditions experienced by the concrete. In general, these factors can be grouped into four major categories: (1) materials factors, (2) design factors, (3) construction practices, (4) environmental conditions. An in-depth review is carried out in this project for each category. Main factors that contribute to early deck cracking, the degree of their influence, remedial considerations, new technologies and ongoing research are summarized in this document. In addition, brief background information is also provided in order to help the readers to have a better understanding of the damage mechanics in concrete decks and specific functions of the recommended remedial considerations.

4.1 Material Factors

Cracking in concrete may occur due to the interaction among the volumetric change of the concrete mass, different kinds and different degrees of structural restraints, and environmental conditions. The volumetric change of concrete is greatly influenced by many materials factors. Concrete cracking as a general problem for reinforced concrete structures has been a research topic for several decades, and it will not be discussed in this report. We are going to focus on early age cracking of high-strength concrete and high-performance concrete, which has attracted considerable attention in the highway industry and research community. Several recent keynote lectures in large conferences have been focused on this topic, and even entire conferences have been devoted to this important topic (Bentur 2000, Holand & Sellevold 1999, Perssen & Fagerlund 1997, 1999, Tazawa 1998). In March 2001, A RILEM international conference was held in Israel on early age cracking in cementitious systems (see the Conference Web Page http://tx.technion.ac.il/~eac). Our survey conducted for the present project shows that most field engineers and local transportation agencies consider material factors be the most important group of factors responsible for early age deck cracking. The following is a summary of our literature review with an emphasis on the influence of material factors on early age cracking in concrete bridge decks.

4.1.1 Properties of Concrete Related to Deck Cracking

A. Shrinkage

There are four different types of shrinkage for concrete: autogenous shrinkage, plastic shrinkage, drying shrinkage and carbonation shrinkage. Autogenous shrinkage and plastic shrinkage are referred to shrinkages occurred at early age of concrete. The drying shrinkage and carbonation shrinkage are long-term material properties of hardened concrete. Among the four types of concrete shrinkages, carbonation shrinkage, induced by carbonation reaction, is not directly related to early age deck cracking and thus, will not be discussed here.

Autogenous shrinkage - Autogenous shrinkage is caused by the consumption of water during the hydration process of cement particles and it is accompanied by a reduction of relative humidity in the concrete and an increase in the surface tension in capillary water. The difference between autogeneous shrinkage and drying shrinkage is that autogeneous shrinkage is not due to the exchange of moisture between concrete and the environment. Autogeneous shrinkage occurs even if there is no moisture exchange at all (i.e., the entire surface of a concrete specimen is sealed). Most of the concrete mix designs with a low water-to-cement ratio (0.33-0.40) exhibit high autogenous shrinkage. This causes the concrete to be prone to cracking.

To reduce the risk of early cracking due to autogenous shrinkage, it is suggested to design concrete mix with water-to-cement ratio greater than 0.4. However, if the concrete has to be made with a low water/cement ratio, the literature suggests to replace 25% of the coarse aggregate by water saturated lightweight aggregate to reduce the autogenous shrinkage (Koenders et al. 1998).

Drying Shrinkage - Drying shrinkage is caused by the loss of water from the hardened concrete during exposure to air at less than 100 relative humidity (RH). It is one of the main causes of deck cracking. For commonly used concrete, the drying shrinkage ranges from 500 to 1000 $\mu\varepsilon$ (Rogalla et al. 1995). Curing conditions may change the rate of drying shrinkage but will only have a small influence on the ultimate drying shrinkage strain (the total long-term shrinkage strain). The effect of drying shrinkage on concrete deck cracking needs to be studied from two different aspects. One is the ultimate shrinkage strain, and the other is the shrinkage rate. Many studies indicate that the rate of shrinkage could have a more significant impact on deck cracking than the ultimate drying shrinkage. This is because, at a high shrinkage rate, most of the shrinkage occurs in a short period of time, which results in deck cracking. For early age concrete, the creep of concrete can relax most of the shrinkage stress, and thus reduces the possibility of deck cracking, this happens if the shrinkage rate is not very high. For a concrete prism fully restrained at both ends, cracks may develop at a shrinkage strain of around 200~250 $\mu\varepsilon$ if not accounting for the creep effect of concrete. Under high shrinkage rate, 200~250 $\mu\varepsilon$ could easily occur at the age of 10 days under normal room temperature and 50% humidity. Therefore, proper measures must be taken to reduce not only the ultimate shrinkage strain but also the shrinkage rate.

There is a range of parameters that are directly related to drying shrinkage of concrete. Aggregate type, water content, cement content, concrete placement temperature, and curing methods all affect the shrinkage of concrete to certain degrees.

Among various constituent phases in concrete, cement paste is the primary phase that causes the shrinkage, and thus, lowering cement content is a very effective to reduce the ultimate shrinkage. Compared with the ultimate shrinkage, it is more difficult to reduce shrinkage rate, which requires a comprehensive evaluation of the influential parameters. Without major adjustment of concrete design parameters, certain special admixtures can be used to reduce drying shrinkage of concrete effectively, such as shrinkage reducing agents (which will be discussed later).

Plastic Shrinkage **-** Plastic shrinkage is a special type of drying shrinkage, it is caused by a rapid loss of moisture on the concrete surface while it is still in a plastic state. It usually occurs when the rate of evaporation exceeds the rate of concrete bleeding. During the plastic state, the development of shear/tensile strength is counteracted by the high mobility of the solid particles in relation to each other (relaxation). The development of shrinkage stresses depends on the volume change and restrain conditions. During the plastic period, mixing water and cement particles form a dense suspension system, and the stability of the system depends on the distance between the cement particles. The dried zones in the fresh concrete will develop in regions of largest porosity. The coexistence of the dried zone and the saturated zones results in tensile stress in concrete. The cracks will occur when the tensile stress exceeds the tensile strength. Typically, plastic shrinkage cracks are no more than 2 or 3 ft (600 or 900 mm) long and are 2 to 3 in (50 – 75 mm) deep. However, the size of cracks could grow due to applied loads or drying shrinkage.

Casting concrete under a high-speed wind condition should be avoided to reduce the risk of plastic shrinkage. Proper curing methods to reduce the evaporation rate, such as the use of a fog mist or curing compound applied to the concrete surface, should be chosen to reduce plastic shrinkage (Rogalla et al. 1995).

B. Creep

Creep of concrete is classified into basic creep and drying creep, according to ACI 209 (the committee on concrete creep and shrinkage). Basic creep is due to external loading without drying; while drying creep is induced by simultaneous loading and drying.

Unlike shrinkage of concrete, creep of concrete has a positive effect on early concrete deck cracking. Creep of concrete (both basic creep and drying creep) reduces tensile stresses caused by restrained drying shrinkage and thermal effects, and thereby, reduces the risk of concrete cracking.

Like shrinkage of concrete, creep of concrete occurs only in cement paste matrix in concrete (i.e., aggregate does not creep). Therefore, more cement paste in a concrete mix means higher creep as well as higher shrinkage. To reduce the risk of early concrete cracking, concrete mix should be designed to have a low early compressive strength and a high rate of creep.

Increasing the paste content and selecting the aggregates that have low modulus of elasticity can increase the creep of concrete. However, high cement content increases autogeneous shrinkage and drying shrinkage, which is an adverse effect. Therefore, a proper balance between concrete strength, shrinkage, creep, and other long-term properties should be determined carefully. High early creep can also be achieved by slowing down the rate of the heat of hydration by using the cement low early strength or by using pozzolanic admixtures and hydration retarding admixture.

C. Modulus of elasticity and strength development

The modulus of elasticity does not directly affect the deformation due to thermal and hydro gradients, but it has a significant effect on the tensile stresses generated in bridge decks by thermal and shrinkage strains. The volume changes due to shrinkage and thermal effects create tensile stresses that are proportional to the concrete's modulus of elasticity. On the other hand, the modulus of elasticity is closely related to the strength of concrete. Generally, a higher strength corresponds to a higher modulus of elasticity.

In general, as the compressive strength increases, creep decreases in a much faster rate than the increase of tensile strength. This is one of the reasons responsible for the poor crack resistance of high strength concrete. Therefore, even through high strength concrete has higher tensile strength than regular concrete at all ages, its shrinkage cracking performance is substantially poorer. This may be due to the higher early age shrinkage and higher modulus of elasticity, in addition to the low creep (Mindess 1994, Wiegrink et al. 1996).

To prevent early concrete cracking, it is recommended to use aggregates with a low modulus of elasticity and a concrete mix that will produce concrete with low early age strength and modulus of elasticity. It is also suggested that the design of concrete deck should be based on later age strength, such as 56- or 90-day compressive strength, instead of the strength at age 28-days, especially when pozzolanic admixtures are used (Rogalla et al. 1995). Moreover, it is also suggested that early age strengths of concrete, such as one day, three days and seven days, be carefully controlled in order to avoid early deck cracking (Holley et al. 1999). Therefore, the concrete mix designs should satisfy the strength development requirement not only at one fixed time but in a certain time period, and also satisfy the general trend of low early strength and high later strength.

D. Heat of Hydration

Heat of hydration generates thermal stresses, which may result in concrete cracking in massive concrete structures. High heat of hydration is also accompanied by high early strength and modulus of elasticity. These conditions enhance the risk of early concrete cracking. Since concrete is very weak in tension, a small temperature difference of 10 °C can possibly result in thermal cracking in large concrete structures.

High heat of hydration can be induced by several causes as follows:

- The use of Type III cement (high early strength cement).
- The use of accelerators.
- High fineness of the cement used in concrete mix (smaller cement particle and thus higher surface area), even if Type I cement is used.
- High cement content in concrete mix.
- High environmental temperature, which accelerates the hydration reaction.

To reduce high heat of hydration, it is suggested to cast concrete at a temperature not higher than 80^{o}F (27^{o}C), and to use retarding agents to slow down the rate of temperature rise. Besides, portland cements with low hydration heat and pozzolanic admixtures are recommended (Rogalla et al. 1995).

E. Coefficient of thermal expansion

Since the development of thermal stresses is linearly proportional to the coefficient of thermal expansion (CTE) of concrete, the CTE may affect concrete cracking significantly. When concrete girders support the bridge deck, thermal stresses from seasonal (uniform full-depth) temperature changes are usually insignificant compared with thermal stresses from diurnal temperature fluctuation. However, if the steel girders support the bridge deck, the seasonal temperature changes may have a significant effect on concrete deck cracking because of the difference in the coefficients of thermal expansion. The CTE of concrete is in the range of 4 to 7 $\mu\varepsilon/^{o}F$ (7 to 12 $\mu\varepsilon/^{o}C$), while that for steel is 7 $\mu\varepsilon/^{o}F$ (12 $\mu\varepsilon/^{o}C$) (Shing and Abu-Hejleh 1999).

Concrete is a composite material with cement paste as matrix and aggregate as inclusion. Therefore, the CTE of concrete is an effective property of the composite, i.e., a combination of the coefficients of thermal expansion of the cement paste and aggregate. The CTE of the hardened cement paste is normally 2-3 times higher than the CTE of aggregate.

For concrete bridge decks in composite with steel girders, a concrete with a high CTE is desirable to minimize the thermal stresses caused by seasonal temperature changes. This is because the difference between the CTE of steel and concrete would be small. However, this will cause a larger diurnal thermal stress in the concrete.

4.1.2 Mix Properties and Mix Properties of Concrete

A. Aggregate size and type

As mentioned earlier, considering concrete as a composite material, the shrinkage, creep and thermal deformation of concrete depends on both the properties of the cement paste and the aggregate. Aggregate is comparatively non-shrinking and non-creeping filler, which has a restraining influence on bulk shrinkage and creep. So, the long-term deformations (shrinkage and creep) all result from the cement paste. It also implies that a high aggregate content leads to low shrinkage of concrete. This is a basic observation from test results shown in the literature, which explains why a high ratio of aggregate to paste (i.e., high aggregate content) can reduce the risk of concrete cracking.

If the aggregate does not shrink, the restraining effect will be maximum. However, if the aggregate shrinks the same way as cement paste, there will be no restraining effect. An extensive review of the effect of aggregates on shrinkage behaviors of concrete can be found in Jennings and Xi (1992). Commonly used aggregates are between these extremes and may be classified into three types (Carlson, 1939):

(A). High shrinkage aggregate: hornblende, pyroxene, marble, and sandstone,
(B). Low shrinkage aggregate: limestone, quartz, glass, feldspar, and dolomite,
(C). Intermediate shrinkage: granite.

In addition to the shrinkage of aggregate, other mechanical properties of aggregate also have a significant effect on the shrinkage of concrete. The ratio of elastic moduli can be defined as E_a/E_c, where E_a and E_c are elastic moduli of aggregate and cement paste, respectively. The higher the ratio the lower is the shrinkage of concrete. Similarly, the ratio of Poisson's ratios can be defined as v_a/v_c, where v_a and v_c are the Poisson's ratios of aggregate and cement paste, respectively. v_a/v_c exhibits the same effect as the ratio of elastic modulus, that is, the higher the ratio the lower is the shrinkage of concrete.

In general, the larger the maximum size of aggregate the smaller is the shrinkage. This is due to the fact that large aggregates tend to form a rigid framework with the shrinking cement paste within them. When the cement paste shrinks, it cannot pull the surrounding aggregates closer since they are already in close contact. As a result, the shrinkage of the cement paste generates some microcracks between the aggregates. As long as the microcracks do not coalescence into a major crack, the crack resistance of the concrete is considered to be enhanced (since no large crack is observed). This is why some studies recommend the use of larger size of aggregates to reduce shrinkage cracking. In fact, the total amount of shrinkage of cement paste does not change with the size of aggregate used, but the shrinkage is consumed partly in between the aggregates by generating microcracks and thus the bulk shrinkage of concrete is reduced.

The literature also shows that as long as the maximum size of aggregate and consistency of concrete are not varied, the gradation of aggregate does not have much of an effect on the shrinkage.

The time-to-cracking of concrete using different types of aggregate varies. Concrete with limestone aggregate shows higher resistance to cracking than those with other types. The literature also indicates that concretes with crushed aggregate are more durable against cracks.

The effects of strength and brittleness of aggregate on the mechanical properties of concrete are not very significant for regular concrete. However, when high strength concrete is to be used, the strength of aggregate must be considered (De Larrard and Belloc 1997, Ozturan and Cecen 1997). Chen et al. in China (2000) and Montgomery and Irvine in Ireland (2000) conducted some systematic tests to investigate the mechanical properties of various types of aggregates and their influence on strength of concrete. Although the specific test data cannot be directly used for the aggregate types in the state of Colorado, the testing procedures they developed are very useful.

B. Cement type

ASTM Type I – Type V. Modern cement has characteristics of high early strength, high heat of hydration and high elastic modulus. This means that the use of modern cement in concrete increases the risk of cracking. Concrete with Type II cement has a lower risk of cracking than that of Type I because Type II cement has a lower heat of hydration. Type III cement gains strength rapidly and may increase the risk of cracking. The literature indicates no information regarding the effect of cement Types IV and V against cracking.

Cement with low alkali content (less than 0.6% of equivalent Na_2O according to ASTM C150) tends to have a lower modulus of elasticity and a higher creep.

In general, to reduce the risk of cracking, it is recommended to use Type II cement, and to avoid finely ground cement and Type III cement (Rogalla et al. 1995).

Shrinkage-compensating cements Another type of cement that has been used in concrete bridge decks for reducing early age cracking is Type E-1, known as expansive cement. Among various expansive cements, Type K cement (ASTM C845-80) is the one that has been used in the U.S. It is also called shrinkage-compensating cement (SCC). The information pertinent to the use of the expansive cements is addressed in ACI 223R-90 (1992) and Army Corps EM1110-2-2000 (1985). This type of cement (Type K) is promising to reduce the risk of cracking in concrete. The mechanism is to create a certain amount of expansion during the hardening process of the concrete, which compensates the autogenous shrinkage and drying shrinkage. Apparently, the most important step in practical applications is to provide adequate restraint on the structure either externally or internally (by reinforcement), allowing the expansion of the cement to generate a prescribed level of prestress in the concrete during the hardening process.

One of the difficulties in practical applications is to predict the amount of expansion when Type K cement is used. Olek and Cohen (1991) conducted experimental research to develop a procedure that could be used to determine the amount of expansive clinker required for producing an expansive cement blend.

The Ohio Turnpike Commission (OTC) has used SCC deck extensively for nearly 15 years. It is the greatest user by far of SCC in the U.S. with over 500 SCC bridge decks. The New York Thruway Authority (NYTA) is probably the second largest user with 47 bridge decks of SCC in the early 1990s. Phillips et al. (1997) conducted a study with the two agencies on their experiences with SCC decks. NYTA had severe scaling and durability problems with SCC decks and issued a moratorium on the use of SCC for bridge decks. OTC had good experiences with SCC decks. They found that some special treatments are needed for successful applications of SCC: higher water-to-cement ratio, faster placement, faster implementation of curing and continuous moist curing for 7 days.

In general, the performance of SCC on the field varies. Some of the studies report the success of using shrinkage-compensating cement to reduce the early cracking in concrete; and some of the studies showed adverse effect (Cusick and Kesler 1977, Keith et al. 1996, Gruner et al. 1993, Mailvaganam et al. 1993, Ramey et al. 1999). Most recently, Pittman et al. (1999) conducted an experimental research to identify possible expansive concrete mix designs for bridge decks for the Alabama Department of Transportation, and further tested the developed concrete mix designs under specific construction and curing conditions. It will take some time to observe any durability problem with the SCC application.

Blended cements Silica fume concrete has been widely used in highway construction for bridge decks due to its high strength and low permeability (such as the Class DT used by CDOT). Despite the advantages of silica fume concrete, it has several undesirable qualities. For example, the fineness of silica fume requires the use of High Range Water Reducers (HRWR), which may result in rapid slump loss. Silica fume concrete has high head of hydration, tends to be sticky and difficult to finish. In addition, silica fume is expensive and is an additional material to add to the mix (ACI234 1996, Babaei and Fouladgar 1997, Whiting and Detwiler 1998).

In 1997, a new type of blended cement, Type IP cement appeared in the market. It is made with calcined clay and seems to be a promising alternative to silica fume concrete. Concrete made with Type IP cement seems to have low permeability but has less of the undesirable qualities of silica fume concrete. The calcined clay is not as fine as silica fume, so less HRWR is needed. Set and finishing properties are more like normal concrete. Cost is less than silica fume concrete and calcined clay is premixed by the cement suppliers and does not have to be added to the mix separately.

A comparative study (Ababneh et al. 2000) was conducted in the Materials Laboratory at CU-Boulder on some durability properties of Type IP and silica fume concrete. The placement of the two types of concretes was monitored and long-term properties of the concretes, such as rapid and long term chloride permeability, drying shrinkage and compressive strength, were tested. The results showed that although the type IP concrete has higher chloride permeability and higher drying shrinkage than the silica fume concrete, the specific values obtained in the study for Type IP concrete were in the acceptable ranges compared with regular concrete.

After the preliminary study, the Type IP concrete was not further investigated or used in the construction of bridge decks.

C. Water-to-cement ratio and cement content

Just as concrete is a composite material, the cement paste itself is also a composite material consisting of unreacted cement particles (the non-shrinking phase) and various hydration products. Concrete with high cement content has a higher crack risk (Bissonnette et al. 1999). This is because high cement content generates more hydration products, which directly contributes to high heat of hydration and high shrinkage, which, in turn, lead to concrete cracking.

Water-to-cement ratio is a dominant parameter that has a very strong effect on the shrinkage of cement paste. High water-to-cement ratio definitely results in high shrinkage. However, the literature indicates that there is no conclusive result regarding the effect of water-to-cement ratio against cracking. Some say that the risk of cracking increases with the increasing water-to-cement ratio, others say otherwise. In fact, low water-to-cement ratio results in less drying shrinkage which helps to improve crack resistance. On the other hand, it also results in less creep, high autogenous shrinkage and high plastic shrinkage, which reduce crack resistance. This is why the literature shows confusing test results.

If we consider both water-to-cement ratio and cement content, then, it is generally accepted that concrete with high cement content and low water-to-cement ratio is more susceptible to cracking than that with low cement content and high water-to-cement ratio. To reduce the risk of cracking, it is recommended to limit cement content to a maximum of 470 lb/yd^3. The literature also indicates that a cement paste volume less than 27.5% can significantly reduce cracking.

In practice, the range of water-to-cement ratio is controlled not just by crack resistance, but mainly by the strength requirement. Recently, due to widely used cementitious admixtures in concrete mixtures such as silica fume and fly ash, the water-to-cement ratio is further generalized to water-to-cementitious materials ratio, in which the cementitious materials include portland cement and all other cementitious admixtures. The commonly used water-to-cementitious materials ratio is between 0.4 and 0.5.

Our most recent experimental study shows that it is possible to develop an optimum concrete mix in terms of various durability requirements (chloride permeability, crack resistance, etc.) with cement content about 470 lb/yd^3 and water-to-cement ratio of about 0.4.

D. Air content

There is no definite conclusion on the effect of air content on concrete cracking. Some studies indicate that increasing air content reduces cracking, while other studies do not show a clear correlation between the two.

The use of air-entrained concrete would be advantageous. It is also suggested to use air content of 6% by volume or more (Schmitt and Darwin 1995).

E. Slump

Slump gives a good indication of concrete workability. It can be reasonably assumed that if an excessive slump is due to high water-to-cement ratio, then the concrete would have a high porosity and therefore high shrinkage. For most concrete with good quality control, the slump of concrete will be within the normal range, and most studies indicate that there is no conclusive relation between the slump and cracking. However, a study conducted by Schmitt and Darwin (1995) showed that cracking in monolithic decks increases with the increased slump, but not in bridge deck overlays. This can be due to settlement cracking in monolithic decks.

To minimize the problem related to slump, it is suggested to avoid the use of excessive slumps.

4.1.3 Admixtures in Concrete

A. Fly ash

Fly ash is a pozzolanic material, which reacts with calcium hydroxide (CH) and produces calcium silicate hydrates (C-S-H). The chemical reaction is called pozzolanic reaction. Both CH and C-S-H are hydration products when portland cement reacts with water. CH is a crystalline material and C-S-H is like a gel, which is the main hydration product which bonds all components in concrete together to provide concrete strength. So, when fly ash is added to the mix, it consumes CH and generates more C-S-H. Since CH is a product of the hydration reaction, the pozzolanic reaction cannot proceed until enough amount of CH is produced by the hydration reaction. Therefore, when fly ash, usually Class F and Class C, is used to replace a certain amount of portland cement in concrete mixtures, the rate of C-S-H growth is slowed down. It is a very effective method to reduce the rate of early age strength gain and early concrete temperatures, and at the same time, to remain the same ultimate strength. In addition to the pozzolanic reaction, other practical advantages have been attributed to fly ash, such as the improvement of workability of concrete, which is due to the spherical shape of fly ash. The small size (average size of 10 μm) and spherical shape of fly ash allow them to pack effectively and fill space between aggregates, which also enhances the strength and reduce the permeability of the concrete. A study indicated that adding pozzolans is more effective in reducing permeability than reducing the W/CM ratio (Ozyildirim, 1998).

The most important issue in the application of fly ash is the percentage of replacement for portland cement. In Germany, it is a common practice to use 100 lb. of fly ash in one cubic yard of concrete for bridge decks. In the U.S., the fly ash is also widely used as one of the additives in concrete mixtures. Numerous attempts have been made to use higher amounts of fly ash in concrete (Malhotra 1986, Langley 1988, Langley et al. 1989). The use of fly ash to reduce the risk of cracking still needs further studies. The use of fly ash in dry climate regions must be followed by proper curing procedure.

Fly ash is a solid waste from power industry. There is no control on the chemical composition of ashes. Therefore, two ashes classified as Class F (or Class C) may have a significant difference in their chemical composition. The implication to practical applications is that the optimum percentage of replacement for portland cement must be determined based on the specific cement, aggregate, and fly ash to be used in a construction project, and there is no standard formula that is valid for all aggregates and cements.

The study conducted at the Materials Laboratory at CU-Boulder has shown that the new concrete with the new smaller particles of fly ash certainly has some advantages over the conventional concrete, but may not be applicable for bridge decks, due to its high early strength, high ultimate strength, and low crack resistance.

B. Silica fume

Silica fume, similar to fly ash, is a pozzolanic material. But, the particle size of the silica fume is much smaller than fly ash, about 1.0 μm, and silica fume is also more reactive than fly ash when mixed with portland cement and water. Silica fume concrete has higher heat of hydration, which results in higher thermal stresses, and bleeds less and is therefore more prone to plastic shrinkage cracks. Some studies indicate that silica fume concrete undergoes intense autogenous shrinkage. It also has higher elastic modulus and lower creep. Because of the small particle size and high surface area, more water (or superplasticizer) in the concrete mix is required to reach the same slump. For the same reason, silica fume concrete appears more difficult for finishing. All these factors increase deck cracking. Therefore, it is generally accepted that silica fume concrete tends to crack in early ages.

To achieve the optimum results, the amount of silica fume used in concrete should be limited. Unless added for a specific reason, the content of silica fume in a range of 6 to 8% by mass of cementitious materials in concrete should be sufficient to reach the desired level of performance. When silica fume is used, it is suggested to use fog sprays or misting right after concrete placement and a 7-day continuous moist curing to reduce early age cracking (Schmitt and Darwin 1995).

When Type K cement is used, the effect of silica fume should be considered from two aspects (Bayasi and Abifaher 1992, Bayasi et al. 1990). One is that the reduction in chloride permeability is much more significant than the reduction created by the addition of silica fume in conventional portland cement concrete. The other one is that silica fume reduces the amount of expansion, which must be considered when silica fume is to be added to a shrinkage-compensating mix design.

C. Ground granulated blast-furnace slags (GGBFS)

GGBFS is a hydraulic cement that works synergistically with portland cement to improve concrete strength and durability. It is similar to pozzolanic materials (fly ash and silica fume) in that it reacts with calcium hydroxide to form CSH during hydration reaction, and more importantly, it directly reacts with water to form CSH. Some of the applications showed that GGBFS can significantly improve the durability of concrete (Prusinski, 2002).

The Wacker Drive Bordering project in Chicago used GGBFS. The concrete mix design used in the project and the durability test data are listed below (Kaderbek et al. 2002):

Type I/II cement	525 lb/yd^3
Fly ash (Class F)	53 lb/yd^3
Silica fume	27 lb/yd^3
GGBFS	79 lb/yd^3
Fine aggregate	1140 lb/yd^3
Course aggregate (max size ¾ in.)	1800 lb/yd^3
Water	254 lb/yd^3
Water reducer	41 fl oz
HRWR	55-110 fl oz
Air entrainment	As needed
Water/cementitious materials ratio	0.27
Air content	7%
Slump (after HRWR addition)	8 in.
Slump (after 45 min.)	4 in.
Initial set time	3 hours
Air void spacing factor	0.01 in.
Air void specific surface area	500 in^2/in^3
Freezing-thawing resistance	DF>90% at 300 cycles, DF>85% at 500 cycles.
Chloride permeability	<2000 coulombs at 28 days

Shrinkage	<600 microstrain at 90 days
Deicer scaling resistance	Rating of 0-1 at 50 cyc.
Chloride penetration	½ - 1 in. <0.03% Chloride by weight of concrete at 90 days.
	½ - 1 in. <0.07% Chloride by weight of concrete at 6 months.

D. Retarders and accelerators

The detailed information about retarders and accelerators can be seen in ACI 212 Committee Report (ACI 212, 1989). The use of retarders increases plastic shrinkage, but decreases the heat of hydration that reduces thermal stresses. It may increase the risk of cracking. The use of evaporation retarder films and fogging may reduce plastic shrinkage cracks when retarders are used in hot or cold weather.

There is no definite conclusion on the influence of accelerators on concrete cracking. It can increase shrinkage, early temperature rise, and early modulus of elasticity. It can also increase early strength and reduce plastic shrinkage cracking.

E. Shrinkage reducing agents

This is a new type of admixture in concrete. It is not included in ACI 212 report (1989).

Drying shrinkage in concrete is a complex process governed by several mechanisms including capillary stress, surface free energy, disjoining pressure, and movement of interlayer water (reviewed by Jennings and Xi 1992). When considering shrinkage in the 45-90% relative humidity range, capillary stress appears to be the predominant mechanism. When pore water evaporates from capillary pores in hardened concrete during drying, tension in the liquid is transferred to the capillary walls, resulting in the shrinkage of concrete. For a given pore size distribution, the internal stress generated upon evaporation is proportional to the surface tension of the pore water. Hence, the main function of most shrinkage reducing agents is to reduce surface tension in fresh concrete, such as lowering the surface tension of pore water. Another way to reduce shrinkage is to block the pathway for moisture exchange, i.e., to seal the pore system in concrete.

Much of the studies have been focused on laboratory evaluation of the properties of concrete with various types of shrinkage reducing agents (Tomita et al. 1986, Shoya et al. 1987, Shah et al. 1992). There have been little applications of shrinkage reducing agents in field concrete, and no applications on bridge decks. Additional research and field application experience are apparently needed for more widespread acceptance and utilization of shrinkage reducing agents.

F. Fiber reinforcement

The use of fiber reinforcement can reduce the crack width induced by various types of shrinkage and thermal deformation. However, it cannot reduce the overall shrinkage of concrete. Therefore, the main function of the randomly distributed small fibers in concrete is to turn large discrete cracks into many diffused and finer cracks. Of course, there are other advantages of using fiber reinforcement such as increasing the tensile strength and modulus of rapture, and improving the post-peak ductility and toughness.

Several types of fibers have been used in concrete: steel, glass, polyvinyl alcohol, cellulose and polypropylene. Research has shown different effectivenesses of the fibers in term of reducing crack width. Most recently, fiber hybridization is gaining much attention in the research community for fiber reinforced concrete. Sun et al. (2000) and Banthia (2000) conducted laboratory studies of the influence of hybrid fibers on the shrinkage and crack resistance of fiber reinforced concrete, in which steel fibers, polyvinyl alcohol fibers, and polypropylene fibers with different sizes and shapes were used together with expansive agents to improve cracking resistance of the concrete. The purpose of adding expansive agents was to improve the interface properties between the fibers and the surrounding cement paste. The performance of the concrete reinforced with hybrid fibers of different sizes and types was better than that with the fibers of mono type and size. It is believed that the fibers compensate each other at different scales (the size of steel fibers is much larger than that of polypropylene fibers), which improves crack resistance.

4.2 Design Factors

The literature reveals that the major design factors that contribute to early age cracking are restraint and girder type. There are also some other potential factors, such as dead-load deflection, concrete cover, span length and reinforcement.

4.2.1 Girder Restraints, Girder Types, Girder Size and Spacing

A concrete deck is usually made composite with the girder that results in restraint at the deck-girder interface. A strong restraint is necessary for the integrity of the girder-deck system, but it could induce deck cracking. Deck cracking could be eliminated or greatly reduced if there is no such restraint. Girder end conditions also affect deck cracking, restrained girder ends can lead to severe deck cracking. In regions over the bridge piers, the bottom of overhangs in bridge decks should have the same quantity of longitudinal reinforcement as the top to avoid severe shrinkage cracks that may develop. For decks with side-by-side girders, one may consider post-tensioning the slab in the transverse direction with unbounded tendons to reduce longitudinal shrinkage cracks in the slab and enhance the shear transfer between the girders.

Cracking occurs more frequently with steel girders than concrete girders. This is because the steel girders have different coefficient of thermal expansion from that of concrete deck, which leads to large volumetric mismatch upon temperature fluctuations. To reduce cracking of decks, reduce longitudinal restraint on bridge decks whenever possible.

Larger girder and closer spacing tend to be more prone to cracking (Rogalla et al. 1995). Therefore, use smaller girders with wider spacing as possible.

4.2.2 Deck Thickness and Thickness of Concrete Cover

Literature indicates that thinner decks tend to be more prone to cracking. However, there is no conclusive evidence. Deck thickness should not be less than 8.5 in. (Rogalla et al. 1995, French et al. 1999).

A thicker concrete cover tends to reduce settlement cracks. It is suggested to use concrete cover not less than 2 in.

4.2.3 Transverse Reinforcing Bars

Some concrete cracks usually occur right above the top transverse bars. To reduce this kind of cracking, avoid placing the top transverse bars and the bottom transverse bars on the same vertical plane, which will reduce the risk of forming full-depth cracks in the deck. Also place the top longitudinal bars above the transverse bars. The use of smaller bars with closer spacing can also improve the performance of concrete decks (Rogalla et al. 1995).

Shrinkage and temperature reinforcement must be placed in concrete deck properly. ACI 318-02 has specific requirements on this important issue (Suprenant 2002). The commentary for Section 7.12.1.2. (ACI 318-02) suggests that, to control cracking, it may be necessary to increase the amount of shrinkage and temperature reinforcement beyond the minimum amount required by the code (minimum ratio of 0.002 for Grade 40 or 50 deformed bars; 0.0018 for Grade 60 deformed bars). Gilbert (1992) indicates that the shrinkage and temperature reinforcement required for a fully restrained slab could be double that required by ACI 318. He showed that the Australian code requires two or three times more shrinkage and temperature reinforcement than the minimum required by ACI 318.

4.3 Construction Practices

Sometimes construction techniques employed by contractors during the construction process may significantly affect the early age cracking of concrete decks. Literature study points out some cases where concrete bridge decks built by certain contractors showed much higher incidence of cracking than those built by other contractors. This may be due to some improper construction practices by the contractors.

4.3.1 Concrete Placement Time, Finishing and Curing

The magnitude of deck cracking is greatly affected by concrete placement time. Late morning or early afternoon concrete placement in warm summer days should be avoided because it increases the temperature in concrete, especially during hydration.

Finishing procedures can affect deck cracking. Delayed finishing can make concrete prone to cracks (Rogalla et al. 1995, Schmitt and Darwin 1995). Ineffective curing was the most common reason suggested by the transportation agencies for excessive deck cracking. Concrete with high cement content and low water-to-cement ratio is more sensitive to curing conditions than that with low cement content and high water-to-cement ratio. For concrete with silica fume and/or fly ash, a 7-day continuous moist curing is recommended to reduce early age cracking. Apply fogging and moist curing as early as possible. Surface finishing and texturing should be completed as soon as possible to allow the final cure of the deck. Hand finishing should not be allowed except at the edge of the pavement.

To reduce early age cracking, use chemical evaporation retarder films and fogging when evaporation rate is high. Apply fogging and moist curing as early as possible. Water curing was recently suggested for better curing in a dry environment (Morin et al. 2002).

4.3.2 Vibration of Fresh Concrete

The literature study indicates that inadequate vibration is a major cause of cracking. Proper vibration methods will improve all the important properties of concrete in bridge decks. To avoid the problems caused by improper vibration, a new type of concrete has been developed, called self-compacting concrete. The first research into the new technology started in Japan in 1990s. The most significant advantage of the new concrete is its extreme workability, which allows the construction of high quality concrete without vibration (Ouchi et al. 2000). The high workability is achieved by using small aggregates and large amounts of admixtures (especially superplasticizers). The new technology is just starting to spread in the construction industry. Recently, the self-compacting concrete is further reinforced by various types of fibers for potential applications such as top thin layer on concrete bridge decks (Fischer and Li 2000, Li et al. 2000).

4.4 Environmental Conditions

Some environmental conditions may affect bridge deck cracking. Environmental conditions to be considered include wind, evaporation rate, relative humidity and air temperature.

4.4.1 Wind and Evaporation Rate

The presence of high wind increases the evaporation rate of water that may result in a condition under which the hydration process cannot proceed properly. This leads to substandard concrete properties. High evaporation rate is the main cause of plastic shrinkage cracks. Plastic shrinkage usually occurs when the surface evaporation rate exceeds the rate of concrete bleeding. To reduce concrete cracking, concrete placement during high-speed wind should be avoided.

Measure or estimate evaporation rate at the job site. For all decks, avoid concrete placement when the evaporation rate is above 0.20 lb./ft.^{2}/hr. (1.0 kg/m^2/hr) for normal concrete and 0.10 lb./ft.^{2}/hr. (0.50 kg/m^2/hr) for concrete with low water-to-cement ratio. The evaporation rate can be calculated using the chart developed by Lerch (1957) based on the measured wind velocity, concrete temperature, air temperature, and relative humidity. The method is adopted by ACI 305 (1991).

4.4.2 Relative Humidity

Without wind, the evaporation rate of moisture from fresh concrete could also be very high, which is directly due to the environmental relative humidity. Low environmental humidity will result in high drying shrinkage.

4.4.3 Air Temperature and Temperature Variation

The surrounding air temperature has a significant influence on deck cracking because it increases the rate of hydration and the evaporation rate of moisture from fresh concrete. The literature indicates that concrete should not be cast when air temperature is cooler than 7ºC (45ºF) and warmer than 27 ºC (80ºF) (Rogalla et al. 1995). Avoid large temperature variation (greater than 50ºF (10ºC)) on the day of concrete placement (Shing and Abu-Hejleh 1999). A recent study (Ozyildirim, 1998) revealed that concretes cured at higher temperatures have higher strength up to 28 days but lower strengths at one year. The permeability of the concrete with high temperature curing decreased with time.

Concrete mix temperature must be maintained above 50ºF (10ºC) for the first 72 hrs. and above 40ºF (4ºC) for the remaining curing period. Limit the maximum concrete temperature at placement to 80ºF (27ºC).

5. Research Activities in Colorado on the Deck Cracking Problem

5.1 Quick Research Study on Type IP Cement

As described in Section 2.1.1 for blended cements, application of Type IP cement on bridge deck overlay was studied in 1997 by Xi and his co-workers at the University of Colorado at Boulder. The Type IP cement was made of portland cement and calcined clay. The research results were summarized in a conference paper (Ababneh et al., 2000), and all details can be found in a research report (Xi et al. 1999). Regardless of the results of this preliminary study, Type IP concrete has not been further used in the construction of bridge decks in Colorado.

5.2 CDOT Research Report 99-8

In 1999, CDOT conducted a study entitled "Cracking in Bridge Decks: Causes and Mitigation." (Shing and Abu-Hejleh 1999). The study included the following work:

- Reviewed recent studies on the cause of bridge deck cracking; and identified material and design specifications, and construction practice that can help to reduce the severity of deck cracking.
- Performed an experimental study to compare the shrinkage properties of different concrete mixes that had been used by the Colorado Department of Transportation since 1971; and examined the influence of cement content and fly ash on the drying shrinkage of concrete.
- Reviewed current CDOT material and design specifications, and construction practice; and identified areas that need improvement.
- Surveyed seven newly constructed bridges to examine the extent of deck cracking (presented in Chapter 3).
- Identified important factors that influence deck cracking and development of recommendations that can be adopted by CDOT to reduce deck cracking.
- Identified concrete mixes that may be used to reduce the severity of deck cracking.

The mix design presented in Table 5.1 was recommended for consideration and further studies. This new mix was referred to as class DLS (class D with low shrinkage). CDOT Report 99-8 paved the way for the development of the two new concrete mixes for bridge decks in CDOT's specifications. It can be seen that the typical cement contents used by CDOT were 615~660 lb./cu. yd. for class D and 700 lb./cu. yd. for class DT, which are much higher than what is recommended in the literature reviewed in this study for the control of deck cracking. Furthermore, fly ash, when it is used in Colorado's bridge decks, is only 10% by weight of cement. In spite of the good performance of a number of such decks as shown in the aforementioned survey, this quantity may not be sufficient to lower the heat of hydration and early concrete strength to desirable levels, especially when silica fume is used. In Minnesota and Germany, fly ash is allowed up to 20% by weight.

After reviewing CDOT's "Standard Specifications for Road and Bridge Construction" and the recent updates of this document, the following recommendations are provided for CDOT to consider.

Table 5.1 Concrete Mix Design of Class DLS

Required 56-day Compressive Strength (psi)	4,500
Maximum Aggregate Size	0.75~1.00 in.
Cement Type	II
Cement Content	451 lb./cu.yd.
Silica Fume	27 lb./cu. yd.
Fly Ash-Class F	90 lb./cu. yd.
Maximum Water-to-Cement Ratio (based on the total cementitious materials)	0.38-0.47
Air Content (% by Volume)	7
Range of Slump (in.)	3.0~5.0

5.2.1 Materials Aspect

- Use Type II cement, and avoid finely ground cement and Type III cement in warm weather conditions. (*Both Type I and Type II are used by CDOT.*)

- Limit cement content to a maximum of 470 lb./yd^3 or lower if possible. However, cement content may have to be higher for thin overlays for a good workability and ease of surface finishing. (*CDOT uses 615~660 lb/yd^3 for class D and 700 lb/yd^3 for class DT.*)

- As long as the chloride diffusivity property permits, use a water-to-cement ratio not lower than 0.40; studies show that concrete with a low cement content and high w/c ratio has less cracking. A low w/c ratio can lead to less creep, more autogenous shrinkage, and more plastic shrinkage cracks. Silica fume and fly ash have a lower density than portland cement. Hence, the replacement of cement by silica fume and fly ash will increase the volume of cementitious materials. One may need to increase the w/c accordingly to account for this increase. However, an optimal w/c ratio has yet to be determined. (*Maximum permitted by CDOT is 0.44 for Class D and 0.35 for Class SF, silica fume concrete.*)

- Based on the experience in Germany, it is recommended to use Type F fly ash with a quantity that is 20% by weight of cement. In Colorado, one may consider using such an amount of fly ash in summer months; and in winter months, the use of fly ash can be optional depending on the air temperature. The amount of fly ash may be adjusted according to weather conditions. This, however, requires further studies. Furthermore, fly ash should be used with care in the dry weather conditions in Colorado and a good curing procedure is important for such concrete. (*CDOT uses 10% FA by weight of cement in class DGFA.*)

- Limit silica fume to 6% by weight of cement. (*CDOT uses 7.5% SF by weight of cement.*). Fast strength gain in deck concrete should be avoided. In Germany, deck concrete is not allowed to exceed 870 psi within the first 12 hours. This is achieved by using coarser cement and fly ash. In the U.S., such a limit might not be feasible because of the properties of the cements that are available. In spite of this, it is desirable to compare the early strengths and the rates of strength gain of concrete mixes made with different brands of Type II cement and fly ash that are available. If there is a noticeable variation in early strengths, then a reasonable upper limit should be established on the early strength. Furthermore, based on the results of the above study, reasonable bounds on the 7-day and 28-day strengths should be established, while allowing 56 days to arrive at the specified concrete strength. (*Currently, CDOT uses the 28-day strength only. Rate of strength gain is not controlled.*)

- Use the largest aggregate size possible and well-graded aggregate to minimize the cement paste volume. However, the maximum aggregate size should not exceed 1/3 of the deck thickness or 3/4 of the minimum clear bar spacing.

5.2.2 Design Factors

The design of bridge decks is often governed by the load carrying capacities. However, one should always consider the impact of design factors on the temperature and shrinkage cracks whenever possible. Some of the following recommendations are based on the literature survey, while others are based on the input from CDOT's Staff Bridge:

1. In regions over the bridge piers, the bottom of overhangs in bridge decks should have the same quantity of longitudinal reinforcement as the top to avoid severe shrinkage cracks that may develop.

2. For decks with side-by-side girders, one may consider post-tensioning the slab in the transverse direction with unbonded tendons to reduce longitudinal skrinkage cracks in the slab and enhance the shear transfer between the girders.

3. Use AASHTO/LRFD specifications to minimize the transverse reinforcement in decks; use smaller transverse bars which result in closer spacing of transverse reinforcement.

4. Use smaller girders.

5. Reduce longitudinal restraint on bridge decks whenever possible.

5.2.3 Construction Practice

Do not cast concrete decks when air temperature is less than 40^{o}F (7^{o}C) or over 80^{o}F (27^{o}C). Avoid large temperature variation (greater than 50^{o}F (10^{o}C)) on the day of concrete placement. This should be applied to all concrete decks. Cast concrete decks in early or mid-evening if the forecast temperature is 80^{o}F or above. Decks can be placed at night as long as they can be fogged for at least five hours before the air temperature goes beyond 80^o. (*For silica fume concrete overlay placement, CDOT requires that concrete deck surface temperature shall not fall below 40^{o}F. The maximum allowable air temperature is 80^{o}F for all concrete placements.*)

- Concrete mix temperature must be maintained above 50^{o}F (10^{o}C) for the first 72 hrs. and above 40^{o}F (4^{o}C) for the remaining curing period. Limit the maximum concrete temperature at placement to 80^{o}F (27^{o}C). (*CDOT currently specifies that concrete mix temperature must be maintained above 50^{o}F for the first 72 hrs. when the ambient temperature is below 35^{o}F and above 40^{o}F for the remaining curing period. Current CDOT limit on the maximum concrete temperature at placement is 90^{o}F.*)

- Measure or estimate evaporation rate at the job site. For all decks, avoid concrete placement when the evaporation rate is above 0.20 lb./ft.^{2}/hr. (1.0 kg/m^2/hr) for normal concrete and 0.10 lb./ft.^{2}/hr. (0.50 kg/m^2/hr) for concrete with low water-to-cement ratio. The evaporation rate can be calculated using the chart in Appendix D based on the measured wind velocity, concrete temperature, air temperature, and relative humidity. (*This is required by CDOT for silica fume concrete only.)*

- Apply fogging to all concrete decks without delay until the surface has been covered by the final cure. (*Required by CDOT for silica fume concrete only.*)

- For concrete with silica fume and/or fly ash, adopt a 7-day continuous moist curing to reduce early age cracking. Results of NCHRP Project 18-3 (NCHRP Report 410) indicate that silica fume has little influence on cracking provided that the concrete is properly cured for at least

7 days. Apply fogging and moist curing as early as possible. (*CDOT has a minimum of 5 days curing requirement for deck concrete.*)

- Surface finishing and texturing should be completed as soon as possible to allow the final cure of the deck. Hand finishing should not be allowed except at the edge of the pavement unless it is approved by the engineer.

- Seal all the cracks that develop in the first year after casting. Before crack sealing proceeds, conduct a crack survey and map the cracks in a manner presented in the preliminary crack survey in Appendix C. These tasks can be conducted by the contractors.

5.3 IBRC Research Study

Most recently, in a research project sponsored by FHWA under the Innovative Bridge Research and Construction (IBRC) program. The objective of this research study was to develop high performance (durable) concrete mix for CDOT that has low cracking and shrinkage potential and low permeability. Durability requirements specified by some of the DOTs are listed in Table 2.2. Since 1991, FHWA has established a High Performance Concrete Technology Delivery Team to help state DOTs to build more economical and durable bridges using high performance concrete. Details can be found at http://knowledge.fhwa.dot.gov/cops/hpcx.nsf/home.

More than 40 mix designs were tested and analyzed. This was a joint research effort by CDOT, CU-Boulder, and Lafarge. Based on test results of more than 40 concrete mixes, the following conclusions were made:

- The ratio of water to cementitious materials, w/(c+m), had the significant effect on chloride permeability, the permeability was proportional to the w/(c+m) ratios.
- Chloride permeability was not correlated to the total air entrainment. It was found that when air content was high, the permeability was usually low rather than high.
- The time for the first cracking to occur in the concrete ring test seemed not to be obviously related to the chloride permeability, but related to compressive strength. As the cement

content increased, the compressive strength of concrete increased, and the time for the first cracking to occur decreased.

- Class F fly ash is better than Class C fly ash in improving both the chloride permeability and cracking resistance of concrete.
- A proper increase in the content of coarse aggregate can improve the permeability, the cracking resistance, and 28-day strength. However, an increase in the proportion of an intermediate size of gravel does not improve the cracking resistance of concrete, nor the permeability. A larger size and higher proportion of gravel should be used.
- Longer curing time (12 days) seems to have an unfavorable effect on cracking resistance of concrete, but this needs to be confirmed by a more detailed experimental study.

Table 5.2 List of recommended mix designs

		II4-4	II 8	SFSP-F
Cement content (lb/yd^3)		465	485	490
Fly ash lb/yd^3 (wt.% of cement)		F116 (25)	F97 (20)	F98 (20)
Silica fume lb/yd^3 (wt.% of cement)		18.6 (4)	19.4 (4)	19.6 (4)
W/(C+M)		0.37	0.41	0.41
Sand (lb/yd^3)		1231	1398	1340
Gravel (lb/yd^3)		1780	1595	1595
HRWR (oz/100 lb cement)		11.91	11.14	5.13
Micro Air (oz/100 lb cement)		0.54	1.6	0.82
Retarder (oz/100 lb cement)		2.16	3.2	2.05
Slump (inch)		6.0	5.5	4.5
Air content (%)		5.5	8.5	7.0
Permeability at 28 days (Coulomb)		3290 2747	2941 3161	4392 4141
Permeability at 56 days (Coulomb)		2528 2005	1393 1609	2212 2346
First cracking (days)		18	14	15
Compressive strength (psi)	3 days	3487	2512	3105
	7 days	4363	3695	3583
	28 days	5645	4657	4634
	56 days	5661	5414	5541

The optimum ranges for primary concrete mix design parameters proposed in the project are:

The range for cement content	from 465 to 485 lb/yd^3
Water/cementitious ratio	from 0.37 to 0.41
Class F fly ash	from 20% to 25% (of cement content)
Silica fume	4% (of cement content)

Two mix designs (listed in Table 5.2) are recommended for use in the summer and in the winter, respectively. In the summer season, Mix II4-4 is preferable. It has a low cement content of 465 lb/yd^3 and a high fly ash content of 25 wt.% of cement. The water/cementitious ratio can be slightly increased if necessary to improve workability. In the winter season, Mix II8 is preferable. It has higher cement content and lower fly ash content than Mix II4-4. In Mix II8, gravel content could be increased to 1780 lb/yd^3 and w/c could be slightly reduced. In both mixes, Class F fly ash should be used.

For thin overlay concrete, Mix SFSP-F or Mix II4-4 or Mix II8 can be selected. If Mix II4-4 or Mix II8 is used for thin overlays, smaller aggregate should be used in the mix.

Table 5.3 Concrete mix design used in O'Fallon Park Bridge

Materials	Standards	Amount (per yd^3)
Cement Type I/II	AASHTO M85	470 lbs.
Fly ash, Class F	AASHTO M295	90 lbs.
Silica fume	AASHTO M307	25 lbs.
Sand	AASHTO M6	1250 lbs.
Gravel, size #57/67	AASHTO M80	1780 lbs.
Air entraining agent	AASHTO M154	2.9 ozs.
Water reducer, DC-55	AASHTO M194	70.2 ozs.
Water	AASHTO T26	244 lbs (29.3 gal.)

These mix designs were selected based on the following criteria

- 28-day Strength: 4500 to 5000 psi
- 56-day Strength: below 5500 psi

- 56-day Permeability: about 2000 coulombs or below
- Cracking time by ring test: 14 days or longer

Detailed experimental results for these mixes can be found in Report No. CDOT-DTD-R-2001-11 (Xi et al. 2001a). Mix II4-4 (and its deviations) was used (in the summer of 2002) in the construction of O'Fallon Park Bridge (IBRC Project #BRO-C110-015). The new concrete mix design can be seen in Table 5.3. The performance of the concrete will be monitored by CDOT and by the University of Colorado at Boulder.

Accompanying the use of this new mix, several new tests (Xi et al. 2001a) are conducted to verify the long term properties of the deck. These new tests are supplemental to the existing standards, and now include tests for 56-day and 90-day concrete compressive strength, chloride permeability (28 days and 56 days), shrinkage, creep, and ring cracking tests (for crack resistance). These tests are performed for each pour in the construction of the bridge.

5.4 Research Study on Fly Ash Micron-3

Boral Material Technologies Inc. (BMTI) has developed a new refined pozzolan, Boral Micron-3, which is an extremely fine, light-colored powder composed primarily of amophous calcium-silicates and aluminates. The mean particle diameter of Micron-3 typically ranges from 2.7 to 3.5 microns with over 90% of the material having a particle diameter less than 7 microns. Boral Micron-3 meets the Class F fly ash requirements of ASTM 618. Previous studies showed that Micron-3 can improve workability of fresh concrete and compressive strength of hardened concrete and reduce the permeability of the concrete. The concrete strength and permeability are comparable to the strength and permeability of the concrete with other highly reactive pozzolans such as silica fume.

Many concrete specimens were prepared with four different concrete mixes, namely, Reference mix, Silica fume mix, Micron A, and Micron B. Reference mix has no silica fume and no Micron-3. Silica fume mix has no Micron-3. Micron A and Micron B have no silica fume and have different dosages of Micron-3. The specimens were prepared by different curing conditions, and tested by several methods. Detailed results can be found in a report (Xi and Xie, 2001). The major conclusions were:

- Micron A has the lowest free shrinkage in most cases among the four mixes studied.
- The duration of curing has different effects on different mixes in terms of the drying shrinkages. Each mix has its own optimum curing time. Longer curing time tends to decrease the drying shrinkage and total shrinkage for all four mixes, but longer curing time does not help in reducing the autogenous shrinkage.
- There is no consistent relationship between the linear shrinkages and the cracking time measured by the ring test. There is a very good correlation between the initial cracking time and the 28-day compressive strength. The mixes with higher 28-day compressive strength have shorter initial cracking time.
- Micron B has the shortest initial cracking time among the four mixes studied.

6. Recommendations

The major factors contributing to deck cracking in new Colorado bridges were investigated. These include material, design, and construction factors. To accomplish this challenging task, an extensive literature review was performed; a survey among local agencies in Colorado was conducted and the survey results were analyzed; current CDOT specifications and practices were reviewed; information on the current practices of other surrounding DOTs were collected; and most importantly, nine newly constructed bridges were inspected. The inspection methods used were visual inspections (photos and crack mappings), sounding test by chain-dragging to detect concrete delamination, ultrasonic test for evaluating concrete quality and possible internal damages and cracks, and concrete coring to determine chloride permeability.

Based on the information collected from the CDOT database, presently, 18% of newly constructed bridge decks in Colorado have no early cracking problem, and the rest (82 %) have various degrees of cracking problems. According to an NCHRP Study (NCHRP Report 380) and various FHWA publications, the acceptable crack width from a corrosion and durability standpoint is between 0.004 in and 0.008 in. (0.1 and 0.2 mm). The widths of cracking observed on the inspected bridges in this report, however, vary from 0.01 to 0.10 in. (0.25 to 2.5 mm) in width. These cracks are usually severe, widespread, and spaced at a relatively uniform interval. Typically, they are oriented in the transverse and/or longitudinal directions. Occasionally, the cracks can form in random orientations. The cracks with widths larger than 0.004 to 0.008 in. have significant effects on permeability of concrete. Even with corrosion inhibitors applied in the concrete, the initiation of the corrosion of embedded rebars will be considerably accelerated.

Based on survey observations and laboratory test results, there seems to be a close relationship between the degree of cracking and the permeability index by the Rapid Chloride Permeability test. Higher permeability may be correlated to higher cracking tendancy. Therefore, a proper requirement on the permeability should be developed when new mix designs are specified.

6.1 Materials Aspects

1. Use Type I or Type II portland cement for bridge deck construction. Avoid finely ground cement and Type III cement. When Type I is used, an increased amount of fly ash can be added to replace portland cement in order to reduce the heat of hydration and strength development. Use Class F fly ash at 20% to 25% by weight of cement when Type I or Type II portland cement is used (*CDOT uses 10% FA in Class DGFA; no fly ash is used in other mix designs*).

2. Limit cement content to a maximum of 470 lb/yd^3 or lower if possible (*CDOT uses 615 to 660 lb/yd^3 for Class D and 700 lb/yd^3 for Class DT*).

3. Use a water/cement ratio of around 0.40 (Maximum permitted by CDOT is 0.44 for Class D and 0.35 for Class SF).

4. Limit silica fume to 5% by weight of cement in order to reduce permeability. Higher silica fume content will increase strength development and make the concrete mix stick (*CDOT uses 7.5% by weight of cement*).

5. Ground granulated blast-furnace slag (GGBFS) can be used together with fly ash and silica fume to improve durability of concrete (see Section 2.1.1.3 C).

6. In view of the importance of high early shrinkage strain, the rate of strength gain should be specified at 1, 3, 7, 28 and 56 days (see Section 2.1.1.C) (*CDOT uses the 28-day strength only*).

7. Permeability, drying shrinkage, and crack resistance tests should be considered as acceptance tests.

8. Use large sized and well-graded aggregate.

6.2 Design Factors

Considering deck cracking is a problem in Colorado, some design factors can be modified to help reduce the potential of deck cracking. It is important to point out that, differing from the recommendations for material factors, the effectiveness of the following recommendations for design factors have not been experimentally verified.

1. For decks with side by side girders, one may consider post-tensioning the slab in the transverse direction with unbonded tendons to enhance the shear transfer between girders and reduce longitudinal shrinkage cracks in the slab.

2. Use smaller size of reinforcement in the regions of negative moment.

3. Increase the minimum ratio for shrinkage and temperature reinforcement considering the arid environment of Colorado.

4. Use AASHTO/LRFD specifications to minimize the transverse reinforcement in decks; use smaller transverse bars. #5 rebar at 5.5" spacing is recommended.

5. Use smaller girders.

6. Reduce longitudinal restraint on bridge deck whenever possible. Restrained ends induce more cracking; therefore, reduce longitudinal restraints.

7. Concrete girders should be preferred for equivalent coefficients of thermal expansion.

8. Place the top transverse bars offset from the bottom transverse bars with regard to the vertical plane. This will reduce the risk of forming full-depth cracks in the deck.

9. Place the top longitudinal bars above the transverse bars.

10. Consider a minimum deck thickness of 8.5" (*CDOT currently uses a minimum deck thickness of 8.0"; 8.5" decks begin at an effective span of 10.25'*). Note that thin decks tend to crack more.

11. Consider thicker concrete (use >2.5"cover) (*CDOT uses top layer of reinforcing steel cover of 2.5" for deck with overlay, and 3" for bare deck*).

6.3 Construction Practices

Some construction practices can be improved to help reduce the potential of deck cracking. Again, it is important to point out that, differing from the recommendations for material factors, the effectiveness of the following recommendations for construction practices have not been experimentally verified.

1. Do not cast decks when air temperature is lower than 45 ºF or over 80 ºF. Avoid large temperature variation during concrete placement. (*CDOT requires that concrete deck surface temperature shall not fall below 40 ºF for silica fume concrete overlay placement; the maximum allowable air temperature is 90 ºF for all concrete placement*)

2. For all decks, avoid concrete placement when the evaporation rate is above 0.20 lb/ft^2hr for normal concrete and 0.10 lb/ft^2hr for concrete with low water/cement ratios. This will require on site measurement or estimate of the evaporation rate (*This is required by CDOT for silica fume concrete only*).

3. Apply immediate fogging to all concrete decks until the surface has been covered by the final cure (*Required by CDOT for silica fume concrete only*).

4. For concrete with silica fume and/or fly ash, adopt a 7-day continuous moist curing to reduce early age cracking (*CDOT has a minimum 5-day curing requirement for deck concrete*).

5. Surface finishing and texture should be completed as soon as possible to allow the final cure of the deck.

6. Seal all cracks that develop within the first year of casting. It should be noted that the critical crack width (without significant impact on durability of concrete) is between 0.004 in and 0.008 in. (0.1 and 0.2 mm), which are very small crack widths. In the field, a more practically feasible critical crack width should be established. Any crack wider than the critical width should be sealed.

7. Consideration of placement sequence recommended by NJ-DOT (NJDOT, 2000)
   - pouring complete deck at one time whenever feasible within the limitation of the maximum placement length based on the drying shringkage consideration.
   - placing each span in one placement for multispans composing of simple span.
   - dividing the deck longitudinally and making two placements for simple span bridge that cannot be placed in a single placement.
   - placing the center span segment first and making this placement as large as possible for simple span bridge that single placement cannot be made over full span length.
   - if multiple placement must be made and the bridge is continuous span, then place concrete in the center of positive moment region first and observe 72 hours delay between placement.
   - when deck construction joint are created, require priming existing interfaced surfaces with a primer/bonding agent prior to placement of new concrete

7. Implementation Update

7.1 Study Recommendations That Were Implemented

Several recommendations for changes in CDOT specifications have been implemented. The revised CDOT Section 601 from 2003 is presented in Appendix C. The revisions were mostly based on the recommendations of the previous chapter. Two new concrete classes are established in the revision for bare bridge decks: Class H for the deck and Class HT for the overlay that will replace the old Class SF. The old Class D mix will be utilized when asphalt overlay or membrane will cover the bridge deck. The old Class DT will be considered for rehabiliation and repair work only. The changes in Section 601 of CDOT's standard specifications include the following.

Definition of Class H and Class HT:

Class H concrete is used for bare concrete bridge decks that will not receive a waterproofing membrane. Additional requirements for Class H concrete are: An approved water reducing admixture shall be incorporated in the mix. Class H concrete shall contain a minimum of 55% AASHTO M 43 size No. 67 coarse aggregate. Class H concrete shall contain cementitious materials in the following ranges: 267 - 297 kg/m^3 (450 - 500 lbs/yd^3) Type II portland cement, 53 - 74 kg/m^3 (90 - 125 lbs/yd^3) fly ash and 12 - 18 kg/m^3 (20 - 30 lbs/yd^3) silica fume. The total content of Type II portland cement, fly ash and silica fume shall be 344 - 380 kg/m^3 (580 - 640 lbs/yd^3). Laboratory trial mix for Class H concrete must not exceed permeability of 2000 coulombs at 56 days (ASTM C 1202). Laboratory trial mix for Class H concrete must not exhibit a crack at or before 14 days in the cracking tendency test (AASHTO PP 34).

Class HT concrete is used as the top layer for bare concrete bridge decks that will not receive a waterproofing membrane. Additional requirements for Class HT concrete are: An approved water reducing admixture shall be incorporated in the mix. Class HT concrete shall contain a minimum of 50% AASHTO M 43 size No. 7 or No. 8 coarse aggregate. Class HT concrete shall contain cementitious materials in the following ranges: 267 - 297 kg/m3 (450 - 500 lbs/yd3)

Type II portland cement, 53 - 74 kg/m3 (90 - 125 lbs/yd3) fly ash and 12 - 18 kg/m3 (20 - 30 lbs/yd3) silica fume. The total content of Type II portland cement, fly ash and silica fume shall be 344 - 380 kg/m3 (580 - 640 lbs/yd3). Laboratory trial mix for Class HT concrete must not exceed permeability of 2000 coulombs at 56 days (ASTM C 1202). Laboratory trial mix for Class HT concrete must not exhibit a crack at or before 14 days in the cracking tendency test (AASHTO PP 34).

Bridge Deck Placing for Class H and Class HT:

Class H, and Class HT concrete shall be placed only when the concrete mix temperature is between 10^{o}C and 27^{o}C (50^{o}F and 80^{o}F) at the time of delivery. Class H and Class HT concrete shall not be placed in the bridge decks when air temperature exceeds 26^{o}C (80^{o}C) and/or the wind velocity exceeds 16 Km/h (10mph) as determined by digital thermometer and anemometer provided on site by the Contractor. If the Engineer can determine from the Contractor's data that the evaporation rate is les than 1.0 kg/m2/hr (0.20 lb/ft2/hr), in accordance with Figure 2.1.5 in ACI 305, then Class H and HT concrete may be placed under these conditions.

Curing Concrete Bridge Decks for Class H and Class HT:

For Class H and HT concrete the minimum curing period shall be 168 hours and from May 1 and until September 30 the water cure method as described below shall be used without the membrane forming curing compound.

Class H, Class HT and Class S50 concrete shall be cured as follows:

1. The concrete surface shall be kept moist at all times by fogging with approved atomizing nozzles until the surface has been covered by the final cure.

2. At lease two atomizing nozzles shall be in operation at all times. A fogging nozzle that has shown acceptable performance is FOGG-IT Waterfog, low volume (7.5 liters per minute),

manufactured by Fogg-it Nozzle Co. at P.O. Box 16053, San Francisco, California, 94116, or an approved equal.

3. From October 1 until April 30 continuous fogging will not be required if the evaporation rate is less than 0.50 kg/m2/hr (0.10 lb/ft2/hr). Ambient temperatures during initial curing shall be warm enough that the water from fogging does not freeze before insulating blankets are applied. The internal concrete temperature shall be determined by using thermocouples and a continuous recording device. The Contractor shall provide the thermocouples and a continuous recording device and install the thermocouples at locations designated by the engineer. The continuous recording device connected to the thermocouples shall be calibrated to provide accurate temperature readings. During the cure period the continuous recording device shall be visible, show visible readings, and the Contractor shall continuously monitor the concrete temperature and provide the recorded data to the engineer after the monitoring temperature for that placement is complete.

7.2 Study Recommendations That Were Not Implemented

Most of recommendations for material factors have been implemented in the recent research projects, such as the project of I-225 Parker Rd. interchange and the project of O'Fallon Park Bridge. The effectiveness of the recommendations has been verified by an experimetal study conducted in the University of Colorado at Boulder (Xi et al. 2001a). Among all recommendations for material factors, the use of GGBFS, the early age strength test (at 1, 3, and 7 days) and drying shrinkage test have not been considered in the current practice. The availability of GGBFS may be a problem in Colorado.

The recommendations for design factors have not been considered in the current practice. These recommendations should be considered together with structural design requirements, written as a special note, and be available for bridge design engineers.

Most of the recommendations for construction practices have been implemented. The placement sequences recommended by NJ-DOT have not been considered. More studies should be performed to verify the effectiveness of the placement sequences before actual implementation in Colorado.

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Appendix A. A part of CDOT 1999 specifications for construction of bridge decks with concrete mixes D and DT

DESCRIPTION

601.02 Classification. The classes of concrete shown in Table 601-1 shall be used when specified in the Contract.

TABLE 601-1 Concrete Table

Concrete Class	Required 28 Day Field Compressive Strength (MPa)	(1) Cement Content Minimum or Range (kg/m^3)	Air Content % Range (Total)	Additional Requirements
B	25 (3000 psi)	335 565 (lbs/cu yd)	5-8	(2) (4) (8) (10)
D	30 (4500 psi)	365 to 400 615 to 660 (lbs/cu yd)	5-8	(3) (5) (8) (10)
DT	30 (4500 psi)	415 700 (lbs/cu yd)	5-8	(3) (5) (8) (13)
P	30 (4200 psi)	335 565 (lbs/cu yd)	4-8	(7) (8)
S	(6)	400 660 (lbs/cu yd)	5-8	(3) (5) (8) (10)
S35	35 (5000 psi)	365 to 425 615 to 720 (lbs/cu yd)	5-8	(3) (8) (10) (11)
S40	40 (5800 psi)	365 to 450 615 to 760 (lbs/cu yd)	5-8	(3) (8) (10) (12)
BZ	30 (4000 psi)	365 610 (lbs/cu yd)	--	(9) (10)

(1) The cement content tolerance of + or - 1% specified in AASHTO M 157 will be allowed.
(2) Class D concrete may be substituted for Class B.
(3) Classes D, DT, S, S35, and S40 concrete require the use of an approved water reducing admixture.
(4) Class B concrete shall be used when Standard Plans specify Class A concrete.
(5) Bridge deck concrete shall have a maximum water/cement (w/c) ratio of 0.44. In determining the w/c ratio, the cement (c) shall be the sum of the weight of the cement and the weight of the fly ash.
(6) Strength for Class S concrete will be specified in the Contract.
(7) Class P pavement shall contain a minimum of 55% coarse aggregate. Coarse aggregate shall be No. 467 or No. 357 unless all transverse joints are doweled in which case No. 67 or No. 57 coarse aggregate is acceptable.
(8) The slump of the delivered concrete shall not exceed the slump of the approved concrete mix design by more than 38 mm (1½ inches).
(9) Concrete for caissons shall be Class BZ. Entrained air is not required unless specified in the Contract. High range water reducers may be added at the job site to obtain desired slump and retardation. Admixtures shall conform to subsection 711.03. Slump shall be a minimum of 125 mm (5 inches) and a maximum of 200 mm (8 inches).
(10) Superstructure concrete and Class BZ caisson concrete shall be made with 19 mm (3/4") nominal sized coarse aggregate: 100% passing the 25.0 mm (1") sieve and 90% to 100% passing the 19 mm (3/4") sieve. All other concrete shall have a nominal coarse aggregate size of 37.5 mm (1 ? ") or smaller: 100% passing the 50 mm (2") sieve and 95% to 100% passing the 37.5 mm (1 ? ") sieve. Bridge deck concrete shall contain a minimum of 55% of AASHTO Size No. 67 coarse aggregate.
(11) For Class S35 concrete the maximum water cement ratio shall be 0.42. The design cement content shall be selected from the range shown in the table.
(12) For Class S40 concrete the maximum water cement ratio shall be 0.40. The design cement content shall be selected from the range shown in the table.
(13) Class DT concrete shall contain a minimum of 50% coarse aggregate. The coarse Aggregate shall be AASHTO M 43, size No. 7 or size No. 8

MATERIALS

601.03 Materials shall meet the requirements specified in the following subsections:

Fine Aggregate	703.01
Coarse Aggregate	703.02
Portland Cement	701.01
Fly Ash	701.02
Water	712.01
Air Entraining Admixture	711.02
Chemical Admixtures	711.03
Curing Materials	711.01
Preformed Joint Material	705.01
Reinforcing Steel	709.01
Bearing Materials	705.06
Epoxy	712.11

Type I or II cement shall be used unless high early strength concrete or sulfate resisting concrete is called for on the plans or as otherwise permitted.

601.04 (unused)

CONSTRUCTION REQUIREMENTS

601.05 Proportioning. The Contractor shall submit design mix proportions, laboratory trial mix and aggregate data, for each class of concrete being placed on the project. Concrete shall not be placed on the project before the design mix proportions and data have been reviewed and approved by the Engineer. The test data shall show the mix design proportions, of all ingredients including cement, fly ash, aggregate, and additives, plus trial mix data including slump, air content, unit weight, yield, water/cement ratio, and 28 day compressive strength results as trialed under laboratory conditions. The test data submitted shall be based on tests conducted by the Contractor and shall not be based on tests conducted by the Department. The trial mix proportions must produce 28 day compressive strengths at least 115 percent of the required 28 day field compressive strengths. Each design shall establish the mix proportions and sources of all ingredients. Aggregate test data include gradations, percent passing 75 mm (No. 200) sieve, sand equivalent, fineness modules, specific gravities, absorptions, and LA Abrasion test results. The Contractor shall be responsible for the design mix proportions and all subsequent adjustments necessary to produce the specified concrete. The test data for Class P concrete shall also include 28 day flexural strength results from two beams broken in accordance with AASHTO T 97. The Department may run a trial mix to verify that the design mix meets the requirements of subsection 601.02.

The Contractor shall submit a new design mix that is based on the above requirements when a change occurs in the source, type, or proportions of cement, fly ash, or aggregate.

Yield shall be determined in accordance with AASHTO T 121 for each of the following:

(1) The design mix submitted by the Contractor shall be designed to yield 0.995 to 1.01 (26.87 to 27.27 cu. ft./ cu. yd. for English units) as determined by the Contractor.

(2) The trial mix conducted by the Contractor shall have a relative yield of 0.99 to 1.02 (26.73 to 27.54 cu. ft./ cu. yd.) as determined by the Contractor.

(3) For paving concrete where cubic meter (cubic yard) is a pay quantity the relative yield of the concrete produced on the project shall be 0.99 to 1.02 (26.73 to 27.54 cu. ft./cu. yd.). If the relative yield of the concrete produced does not conform to this range for two consecutive yield determinations, concrete production shall cease and the Contractor shall present a plan to correct the relative yield to the Engineer.

Review and approval of the design mix by the Engineer does not constitute acceptance of the concrete. Acceptance will be based solely on the test results of the concrete placed on the project.

The Contractor shall have the option of substituting approved fly ash for portland cement, up to a maximum of 20 percent by weight, in any class of concrete shown in Table 601-1, with the following exceptions:
(1) concrete used for bridge decks shall have a maximum substitution of 10 percent
(2) fly ash added to concrete pavements shall be added in accordance with subsection 412.04 which requires the fly ash to be in addition to the full weight of the cement as specified in Table 601-1.

Where the Contractor's voluntary use of fly ash results in any delay, necessary change in admixture quantities or source, or unsatisfactory work, the cost of such delays, changes or corrective actions shall be borne by the Contractor.

601.12 Placing Concrete.

(a) *General*. A preplacement conference shall be held with selected Contractor and Department personnel prior to placement of concrete bridge decks to discuss the method and sequence of placing concrete. Concrete shall not be placed until forms have been completed and materials required to be embedded in the concrete have been placed, and the Engineer has inspected the forms and materials. The forms shall be cleaned of all debris before concrete is placed.

The external surface of all concrete shall be thoroughly worked during the placing by means of tools of an approved type. The working shall be such as to force all coarse aggregate from the surface and to bring mortar against the forms to produce a smooth finish substantially free from water and air pockets, or honeycomb.

(b) *Hot Weather Limitations*. Placing of concrete during hot weather shall be limited by the temperature of the concrete at the time of placing. Mixed concrete which has a temperature of 32 degrees C (90 degrees F) or higher, shall not be placed.

The Contractor shall provide fogging equipment and keep the concrete surface moist at all times by fogging with an approved atomizing nozzle until the curing material is in place.

The aggregate stockpiles shall be kept moist at all times.

(c) *Cold Weather Limitations*. The mixed concrete temperature shall be between 10 and 32^0 C (50 and 90^0 F) at the time of placement. Water, aggregates, or both shall be heated when necessary under such control and in sufficient quantities to avoid fluctuations in the temperature of the concrete of more than 6^0 C (10^0 F) from batch to batch.

To avoid the possibility of flash set when the water is heated to a temperature in excess of 38^0 C (100^0 F), the water and the aggregates shall be charged into the mixer before the cement is added.

Heating equipment or methods which alter or prevent the entrainment of the required amount of air in the concrete shall not be used. The equipment shall be capable of heating the materials uniformly. Aggregates and water used for mixing shall not be heated to a temperature exceeding 65^0 C (15^0 F). Materials containing frost or lumps of frozen material shall not be used.

Stockpiled aggregates may be heated by the use of dry heat or steam. Aggregates shall not be heated directly by gas or oil flame or on sheet metal over fire.

When aggregates are heated in bins, steam-coil or water-coil heating, or other methods which will not be detrimental to the aggregates may be used. The use of live steam on or through binned aggregates will not be permitted.

Concrete shall not be placed on frozen ground.

(d) *Chutes and Troughs.* Concrete shall be placed so as to avoid segregation of the materials and the displacement of the reinforcement.

Concrete shall not be dropped more than 1.5 m (5 feet), unless confined by closed chutes or pipes. Care shall be taken to fill each part of the form by depositing the concrete as near final position as possible. The coarse aggregate shall be worked back from the forms and worked around the reinforcement without displacing the bars. After initial set of the concrete, the forms shall not be jarred and strain shall not be placed on the ends of projecting reinforcement.

Where steep slopes are required, the chutes shall be equipped with baffle boards or be in short lengths that reverse the direction of movement.

Concrete shall not be pumped through aluminum alloy pipe.

All chutes, troughs and pipes shall be kept clean and free from coatings of hardened concrete.

(e) *Vibrating.* Unless otherwise directed, the concrete shall be consolidated with suitable mechanical vibrators operating within the concrete. When required, vibrating shall be supplemented by hand spading with suitable tools to assure proper and adequate consolidation.

Vibrators shall be of a type and design approved by the Engineer. They shall be capable of frequencies of not less than 10,000 vibrations per minute, in air.

Vibrators shall be so manipulated as to work the concrete thoroughly around the reinforcement and imbedded fixtures and into corners and angles of the forms. Vibrators shall not be used as a means to cause concrete to flow or run into position in lieu of placing. The vibration at any point shall be of sufficient duration to accomplish consolidation, but shall not be prolonged to the point where segregation occurs.

(f) *Depositing Concrete Under Water.* Concrete, except for cofferdam seals, shall not be deposited under water, unless approved by the Engineer. If approved, care shall be exercised to prevent the formation of laitance. Concrete shall not be deposited until any laitance, which may have formed on concrete previously placed, has been removed. Pumping shall be discontinued while depositing foundation concrete if it results in a flow of water inside the forms. If concrete, except for cofferdam seals, is deposited under water, the proportion of cement used shall be increased at least 25 percent at the Contractor's expense. Concrete deposited under water shall be carefully placed in a compact mass in its final position by means of a tremie. The discharge or bottom end of the tremie shall be lowered to contact the foundation at the start of the concrete placement and shall be raised during the placement at a rate which will insure that the bottom or discharge end of the tremie is continuously embedded or buried in fresh concrete a minimum of 300 mm (12 inches). Air and water shall be excluded from the tremie pipe by keeping the pipe continuously filled. The continuity of the placement operation shall be maintained without breaking the seal between the concrete mass and the discharge end of the tremie until the lift is completed. The concrete placement shall not be disturbed after it has been deposited.

(g) *Placement.* Concrete shall be placed in horizontal layers not more than 450 mm (18 inches) thick except as hereinafter provided. When less than a complete layer is placed in one operation, it shall be terminated in a vertical bulkhead. Each layer shall be placed and consolidated before the preceding batch has taken initial set. Each layer shall be so consolidated as to avoid the formation of a construction joint with a preceding layer which has not taken initial set. Bridge deck concrete on superelevation or grade that exceeds 2 percent, shall be placed from the low point upward.

When the placing of concrete is temporarily discontinued, the concrete, after becoming firm enough to retain its form, shall be cleaned of laitance and other objectionable material to a sufficient depth to expose sound concrete. The top surfaces of concrete adjacent to the forms shall be smoothed with a trowel to minimize visible joints upon exposed faces. Work shall not be halted within 450 mm (18 inches) of the top of any face, unless provision has been made for a coping less than 450 mm (18 inches) thick, in which case the construction joint may be made at the under side of the coping.

Immediately after the work of placing concrete is halted, all accumulations of mortar splashed upon the reinforcement and surfaces of forms shall be removed before the concrete takes its initial set. Care shall be taken when cleaning reinforcing steel to prevent damage to or breakage of the concrete-steel bond.

Where Class DT concrete is used for patching, repair, or topping of existing concrete, the area that the Concrete Class DT contacts shall be prepared by shotblasting (3 - 5 mm deep [1/8 - 3/16" deep]) or rotomilling. If Class DT concrete is not placed within one week of the shotblasting or rotomilling the area shall then be sandblasted and cleaned of all sand, concrete fragments, dirt, and other foreign material within one week of placement. The area shall be moistened two to four hours before placement and shall be free of standing water at the time of placement.

(h) *Placing Sequence*. Unless otherwise shown on plans, or ordered, the concrete placing sequence shall be as follows:

Concrete in columns shall be placed in one continuous operation. The concrete in columns shall be allowed to set at least 12 hours before caps are placed. Each span of simple span concrete slab and girder bridges less than 10 m (30 feet) in length shall be placed in one continuous operation.

Concrete for simple or continuous girder spans greater than 10 m (30 feet) shall be placed in two operations; the first operation shall consist of placing the girder stems and any slab at the bottom of the stems, and the second operation shall consist of placing the top deck slab. The second pour shall not be made until the first pour has reached a compressive strength of twice the design unit stress shown on the plans.

Transverse construction joints shall be located as shown on the plans, or as approved.

Concrete slabs on simple span steel girder bridges shall be poured in one continuous operation for each span. If approval is given to place the deck of the entire structure, the Contractor shall use an approved retarder, when necessary, to retain the workability of the concrete and to obtain the desired finish.

Concrete slabs on continuous span steel girder bridges shall be placed in accordance with the placing sequence shown on the plans. The Contractor may place the deck of the entire structure in one operation, when approved. An approved retarder shall be used, when necessary, to retain the workability of the concrete and to obtain the desired finish. The leading edge of the freshly placed concrete shall be kept parallel to the substructure so that the girders will be loaded evenly during the placing and screeding operation.

(i) *Drainage and Weep Holes.* Drainage and weep holes shall be constructed at locations shown on the plans or as ordered. Ports or vents for equalizing hydrostatic pressure shall be placed below low water.

Forms for weep holes shall consist of approved form material. Wooden forms shall be removed after initial set of concrete has taken place.

Inlets of weep holes shall be surrounded with 0.03 m^3 (1 cubic foot) of filter material in a burlap sack, securely tied.

(j) *Construction Joints.* Construction joints shall be made only where located on the plans or shown in the placing schedule, unless otherwise approved.

All construction joints shall be cleaned of surface laitance, curing compound, and other foreign materials before fresh concrete is placed against the surface of the joint. Abrasive blast methods shall be used to clean construction joints between concrete girders and adjoining deck slabs. When the optional construction joints shown on the plans are used, any additional reinforcing steel shall be furnished and placed by the Contractor at no expense to the Department.

Surfaces on which concrete is to be placed shall be thoroughly moistened with water immediately before placing concrete.

Where construction joints are allowed on visible surfaces, chamfer strips attached to the forms or other approved methods shall be utilized to provide an even joint appearance.

When the plans show new concrete to be joined to existing concrete by means of bar reinforcing dowels placed in holes drilled in the existing concrete, the diameter of the holes shall be the minimum needed to place non-shrink grout or epoxy grout and the dowel. Immediately prior to placing the dowels, the holes shall be cleaned of dust and other foreign material and sufficient grout placed in the holes so that there are no voids in the drilled holes after the dowels are inserted.

(k) *Float Finish on Horizontal Surfaces*. All freshly placed concrete on horizontal surfaces shall be given a float finish except as otherwise provided in the plans. Bridge decks and bridge sidewalks shall be finished in accordance with subsection 601.15(c). A float finish shall be achieved by placing an excess of material in the form and removing or striking off the excess with a template, forcing the coarse aggregate below the mortar surface. Creation of concave surfaces shall be avoided. After the concrete has been struck off, the surface shall be thoroughly worked and floated with a suitable floating tool. Before the finish has set, the surface cement film shall be removed with a fine brush in order to have a fine grained, smooth but sanded texture.

(l) *Loading Piers and Abutments*. Superstructure dead loads shall not be applied until piers and abutments have attained a compressive strength of $0.8f'_c$.

The Contractor shall provide an as constructed survey of the abutments and piers prior to girder erection. The Contractor shall submit to the Engineer a copy of the survey notes detailing the girder seat elevations, anchor bolt locations and projections, and span distances from centerline of bearing to centerline of bearing. The survey notes shall indicate all adjustments necessary for bearing device dimensions other than those shown on the plans. The Contractor shall submit details for all adjustments to the Engineer for approval.

(m) *Opening to Traffic*. Concrete structures shall remain closed to traffic, and shall not carry Contractor's equipment, for 21 days after placement of the concrete deck is completed. The structure may be opened to traffic earlier if the concrete deck and all other concrete has attained the Field Compressive Strength given in Table 601-1. The minimum compressive strength shall be determined from test cylinders made and cured at the structure site in accordance with AASHTO T 23 and tested in accordance with AASHTO T 22.

In addition, for cast-in-place prestressed bridges, construction vehicles whose gross weight exceeds 900 kg (2,000 pounds), shall not be allowed on any span until prestressing steel for that span has been tensioned.

(n) *Epoxy Bonder*. An epoxy bonder meeting the requirements of subsection 712.11 shall be used where epoxy bonder is called for on the plans.

601.15 Bridge Deck Placing, Consolidating and Finishing.

(a) *Placing*. Concrete shall be placed in accordance with the requirements of subsection 601.12 except for the following:

Concrete shall be placed in such manner as to require as little rehandling as possible and at sufficient depth to provide some excess for screeding and finishing operations. The concrete shall be directed through suitable drop chutes to as near its final location as practicable. The pattern of placement shall be such that lateral flow will be minimized. Concrete shall be placed against concrete in place where practicable.

(b) *Consolidating*. Consolidation shall conform to subsection 601.12 (e) and to the following:

The Contractor shall provide suitable mechanical vibrators to melt down the batch at the point of discharge and to densify the concrete within the forms. The bond of fresh concrete to concrete previously placed shall be achieved by vibrating the new concrete together with the old. Immersion vibrators shall operate at a speed of not less than 10,000 vibrations per minute in air. Internal vibration shall be used along the edges of forms and in areas of congested reinforcing.

Plate vibrators, vibrating screeds, or rollers may be used for consolidating and finishing slabs with a nominal thickness of 150 mm (6 inches) or less. A combination of internal vibration and surface consolidation shall be used when the nominal slab thickness is greater than 150 mm (6 inches).

(c) *Finishing*. Following consolidation, the concrete shall be struck off and finished by mechanical longitudinal floating, mechanical rolling, surface vibration, or a combination of any of these methods. Surface vibrators shall be of the low-frequency, high-amplitude type, operating at a speed of 3000 to 4500 vibrations per minute. If the vibrator speed is adjustable, maximum speed shall be used on the first pass and minimum on subsequent coverage.

The Contractor shall state at the pre-placement conference the make and type of deck finishing machine intended to be used. Deck finishing machines shall be supported beyond the edge of the bridge deck so that the greatest possible deck width will be finished by machine.

A paver's steel scraping straightedge or lute (100 mm [4 inch] maximum width) will be the only hand tool permitted on deck surfaces, except for a minimum use of metal hand floats and edgers along the forms and in areas where machine finishing cannot be effectively used. Only minimum hand finishing will be permitted and when the Engineer deems the slab surface is being overworked, all hand finishing will be stopped. If the surface of the deck slab becomes dry immediately following finishing operations, due to an excessive evaporation rate, it shall be covered with wet burlap or fogged with water covering the entire deck surface using pneumatic atomizing nozzles. The fog spray shall be just enough to retard surface evaporation and shall not change the water-cement ratio. During periods of excessive drying, a cover of wet burlap or plastic sheeting will be maintained on the slab at all times until final cure cover is placed. Monomolecular film coatings applied to the surface of the slab to retain moisture may be used provided they effectively retard surface evaporation and are adequately maintained throughout the finishing operation.

Bridge decks that will not be covered with waterproofing membrane shall receive a final finish as specified in the Contract.

(d) *Straightedge Testing and Surface Correction*. After the floating has been completed but while the concrete is still plastic, the Engineer may determine that the surface of the concrete should be tested for trueness. For this purpose the Contractor shall furnish and use an accurate 3 m (10 foot) straightedge or other approved device. Any depressions found shall be immediately filled with freshly mixed concrete, struck off, consolidated and refinished. High areas shall be cut down and refinished. Special attention shall be given to assure that the surface across joints meets the requirements for smoothness.

1. *Bridge Deck With Asphalt Riding Surface*. When the concrete is sufficiently hard, the pavement surface shall be retested with the 3 m (10 foot) straightedge or other approved device. Areas showing high spots of more than 3 mm (? inch) but not exceeding 13 mm in 3 m (? inch in ten feet), shall be marked. The marked area shall be immediately ground with an approved grinding tool so that the surface deviation will not be in excess of 3 mm in 3 m (? inch in ten feet). Where the deviation from the established cross section exceeds 13 mm in 3 m (? inch in ten feet), the area or section shall be removed and replaced at the expense of the Contractor, unless permitted to remain with modifications approved by the Engineer.

   Any area or section so removed shall not be less than 3 m (10 feet) in length nor less than the full width of the lane involved. When it is necessary to remove and replace a section of deck slab, any remaining portion of the slab adjacent to the formed joints that is less than 3 m (10 feet) in length, shall also be removed and replaced.

2. *Bridge Deck With Concrete Riding Surface*. Surface smoothness require-ments for a bridge deck built with a concrete riding surface shall conform to subsection 412.17.

(e) *Movable Bridge for the Inspectors*. A movable bridge or platform shall be provided by the Contractor and moved as directed to allow the inspectors to work over the freshly placed plastic concrete. The movable bridge shall be kept as close to the finishing screed as practical. The deck of the movable bridge shall be a minimum of 600 mm (24 inches) wide and no more than 600 mm (24 inches) above the surface of the concrete and shall be capable of supporting two inspectors. The Contractor shall provide additional movable bridges as appropriate for use by the Contractor's workers.

(f) *Concrete Bridge Sidewalks*. Bridge sidewalks shall receive a final transverse broom finish.

(g) If cracks in the deck concrete with a width of 0.9 mm (0.035") or greater occur within two weeks of placement, those cracks shall be repaired at the Contractor's expense. Cracks will be measured by the Engineer by insertion of a wire gauge at any time and temperature within the two weeks. The repair shall consist of filling the cracks with a low viscosity, two part, methacrylate or an approved equal. The repair shall be in accordance with the recommendations of the manufacturer of the crack filling material.

601.16 Curing Concrete Bridge Decks. The minimum curing time shall be five days. The concrete surface shall be kept moist at all times by fogging with an approved atomizing nozzle or applying a monomolecular film coating to retard evaporation until the curing material is in place.

When the ambient temperature is below 2^0 C (35^0 F), the Contractor shall maintain the concrete temperature above 10^0 C (50^0 F) during the curing period. It shall be the Contractor's responsibility to determine for himself the necessity for undertaking protective measures.

Concrete bridge decks, including bridge curbs and bridge sidewalks shall be cured as follows:

(a) Decks placed from May 1 to September 30 shall be cured by the membrane forming curing compound method followed by the water cure method as follows:

1. *Membrane Forming Curing Compound Method.* A volatile organic content (VOC) compliant curing compound conforming to AASHTO M 148, Type 2 shall be uniformly applied to the surface of the deck, curbs and sidewalks at the rate of 40 L/100 m^2 (1 gallon per 100 square feet). The curing compound shall be applied as a fine spray using power operated spraying equipment. The power operated spraying equipment shall be equipped with an operational pressure gage and a means of controlling the pressure. Before and during application the curing compound shall be kept thoroughly mixed by recirculation or a tank agitator. The application shall be within 6 m (20 feet) of the deck

finishing operation. When the finishing operation is discontinued, all finished concrete shall be coated with curing compound within ? hour. The curing compound shall be thoroughly mixed within one hour before use.

2. *Water Cure Method.* The water cure method shall be applied as soon as it can be without marring the surface and shall be continued for five days. The surface of the concrete, including bridge curbs and bridge sidewalks, shall be entirely covered with cotton, burlap, or combination polyethylene sheeting and burlap mats. Approved combinations of a barrier and a water retaining layer may be used. Prior to being placed, the mats shall be thoroughly saturated with water. The mats shall extend at least twice the thickness of the bridge deck beyond the edges of the slab and shall be weighted to remain in contact with the surface. The mats shall remain in contact and be kept wet for a minimum of five days after concrete placement.

(b) Decks placed between November 1 and March 31 shall be cured by application of a membrane forming curing compound followed by the blanket method as follows:

1. *Membrane Forming Curing Compound Method.* This method shall be applied in accordance with 601.16(a)1 above.

2. *Blanket Method.* Curing blankets with a minimum R-Value of 0.5 shall be placed on the deck as soon as they can be without marring the surface. Blankets shall be loosely laid (not stretched) and adjacent edges suitably overlapped with continuous weights along the lapped joints. The blankets shall remain in place for a minimum of five days after placement.

(c) Decks placed in April or October may be cured in accordance with either 601.16(a) or 601.16(b) above.

(d) For decks placed above an elevation of 2500 m (8,000 feet) above mean sea level, the Engineer may modify the time of year requirements for the cure methods defined in 601.16(a) and 601.16(b) above.

Appendix B. A part of CDOT 1999 standard special provision on projects that include concrete Class SF bridge deck overlays

Subsection 601.02 (see Appendix A) shall include the following:

Concrete Class SF shall conform to the following:

Class	Required 28 Day Field Compressive Strength, MPa (psi)	Minimum Cement kg/m^3 (lb/yd^3)	Silica Fume* (Percent of Mass [Weight] of Cement)	Maximum Water Cement Ratio	Total Air Content Range, %
SF	40 (5800)	390 (660)	7.5	0.35	4-8

*In addition to the cement.

Up to 10% by weight of the cement may be replaced by an approved fly ash.

The Contractor may use an approved mix utilizing a water reducing chemical admixture which conforms to AASHTO M 194 Type A, F, or G in the Concrete Class SF. The admixture shall be used in accordance with the recommendations of the manufacturer.

The coarse aggregate for Concrete Class SF shall be AASHTO M 43 size 67 and shall be a minimum of 50% of the total aggregate.

Silica fume admixture for Concrete Class SF shall conform to the following Table:

Silicon Dioxide (SiO_2), Min. %	Sulfur Trioxide (SO_3), Max. %	Fineness, Specific Surface, Min., m^2/kg
85	3	20,000

Prior to placement of Concrete Class SF the Contractor shall submit to the Engineer certified test reports stating that the silica fume admixture meets the specification requirements, supporting this statement with actual test results. The certification shall state the solids content if the silica fume admixture is furnished in a slurry.

Subsection 601.07 shall include the following:

The silica fume shall be added to the Concrete Class SF mix during initial batching for either truck or stationary (central) mixing.

Subsection 601.12 (b) shall include the following:

The Contractor shall determine if the weather conditions are acceptable for placement of concrete Class SF, by determining the overlay surface evaporation rate. The evaporation rate shall be obtained by measuring the relative humidity, the wind velocity, and the air temperature, all at or near the deck surface. A digital hygrometer/ thermometer provided by the Contractor shall be used to record the relative humidity and temperatures. Wind velocity near the concrete deck shall be measured with an anemometer provided by the Contractor. Using the appropriate parameters, the evaporation rate shall be determined using the Evaporation Nomograph in ACI Manual 305R-5, Figure 2.1.5. The maximum allowable or critical rate of evaporation will be 1.0 $kg/m^2/hr$ (0.20 $Lb/Ft^2/hr$). Concrete Class SF placement shall be discontinued when the critical evaporation rate is exceeded, unless the Contractor employs acceptable means and methods to lower the evaporation rate to below the maximum allowable value.

Subsection 601.12(c) shall include the following:

The concrete deck surface temperature shall not fall below 4 °C (40 °F) during the time of Concrete Class SF overlay placement.

(c) *Consolidation.* The Concrete Class SF shall be consolidated with either mechanical vibrators or a vibrating screed. If a vibrating screed is used for consolidation, the edges of the overlay shall be further densified with mechanical vibrators.

In subsection 601.16, first paragraph, delete the subsection title and replace it with the following:

601.16 Curing Concrete Bridge Decks and Bridge Deck Overlays.

Subsection 601.16 shall include the following:

(e) Concrete Class SF shall be continuously fogged as follows: The concrete surface shall be kept moist at all times by fogging with approved atomizing nozzles until the surface has been covered by the final cure (water cure or blanket method). At least two atomizing nozzles shall be in operation at all times. The fogging nozzles shall be FOGG-IT Waterfog, low volume (7.6 L [2 gallons] per minute), manufactured by Fogg-it Nozzle Co. at P.O. Box 16053, San Francisco, California, 94116, or an approved equal.

Appendix C. A part of the revised CDOT 2003 standard for construction of bridge decks with concrete Classes H, HT, D and DT

REVISION OF SECTION 601 STRUCTURAL CONCRETE

This is a standard special provision that revises or modifies CDOT's *Standard Specifications for Road and Bridge Construction*. It has gone through a formal review and approval process and has been issued by CDOT's Project Development Branch with formal instructions regarding its use on CDOT construction projects. It is to be used as written without change. Do not use modified versions of this special provision on CDOT construction projects, and do not use this special provision on CDOT projects in a manner other than that specified in the instructions unless such use is first approved by the Standards and Specifications Unit of the Project Development Branch. The instructions for use on CDOT construction projects appear below.

Other agencies that use the *Standard Specifications for Road and Bridge Construction* to administer construction projects may use this special provision as appropriate and at their own risk.

INSTRUCTIONS FOR USE ON CDOT CONSTRUCTION PROJECTS:

Use this standard special provision on projects that have any type of concrete construction.

March 6, 2003.

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Section 601 of the Standard Specifications is hereby revised for this project as follows:

Delete subsection 601.02 and replace with the following:

601.02 Classification. The classes of concrete shown in Table 601-1 shall be used when specified in the Contract.

Table 601-1 Concrete Table

Concrete Class	Required Field Compressive Strength (MPa)	Cement Content: Minimum or Range (kg/m^3)	Air Content: % Range (Total)	Water Cement Ratio: Maximum or Range
B	25 (3000 psi) at 28 days	335 (565 lbs/yd^3)	5 - 8	N/A
BZ	30 (4000 psi) at 28 days	362 (610 lbs/yd^3)	N/A	N/A
D	30 (4500 psi) at 28 days	365 to 392 (615 to 660 lbs/yd^3)	5 - 8	0.44
DT	30 (4500 psi) at 28 days	415 (700 lbs/yd3)	5 – 8	0.44
E	30 (4200 psi) at 28 days	392 (660 lbs/yd3)	4 – 8	0.44
H	30 (4500 psi) at 56 days	344 to 380 (580 to 640 lbs/yd3)	5 – 8	0.38 - 0.42
HT	30 (4500 psi) at 56 days	344 to 380 (580 to 640 lbs/yd3)	5 – 8	0.38 - 0.42
P	30 (4200 psi) at 28 days	392 (660 lbs/yd3)	4 – 8	0.44
S35	35 (5000 psi) at 28 days	365 to 427 (615 to 720 lbs/yd3)	5 – 8	0.42
S40	40 (5800 psi) at 28 days	365 to 451 (615 to 760 lbs/yd3)	5 – 8	0.40
S50	50 (7250 psi) at 28 days	365 to 475 (615 to 800 lbs/yd3)	5 – 8	0.38

Class B concrete is an air entrained concrete for general use. Class D or H concrete may be substituted for Class B concrete. Additional requirements for Class B concrete are: Class B concrete shall have a nominal coarse aggregate size of 37.5 mm (1½") or smaller, i.e., 100% passing the 50 mm (2") sieve and 90% to 100% passing the 37.5 mm (1½") sieve. Approved fly ash may be substituted for portland cement up to a maximum of 20% Class C or 30% Class F by weight.

Class BZ concrete is concrete for drilled piers. Additional requirements for class BZ concrete are: Entrained air is not required unless specified in the Contract. High range water reducers may be added at the job site to obtain desired slump and retardation. Slump shall be a minimum of 125 mm (5") and a maximum of 200 mm (8"). Class BZ caisson concrete shall be made with 19 mm ($^3/_4$") nominal sized coarse aggregate, i.e., 100% passing the 25.0 mm (1") sieve and 90% to 100% passing the 19 mm ($^3/_4$")

sieve. Approved fly ash may be substituted for portland cement up to a maximum of 20% Class C or 30% Class F by weight.

Class D concrete is a dense medium strength structural concrete. Class H may be substituted for Class D concrete. Additional requirements for Class D concrete are: An approved water reducing admixture shall be incorporated in the mix. Class D concrete shall be made with 19 mm (3/4") nominal sized coarse aggregate, i.e., 100% passing the 25.0 mm (1") sieve and 90% to 100% passing the 19 mm ($^3/_4$") sieve. When placed in a bridge deck, Class D concrete shall contain a minimum of 55% AASHTO M 43 size No. 67 coarse aggregate. Approved fly ash may be substituted for portland cement up to a maximum of 20% Class C or 30% Class F by weight.

Class DT concrete may be used for deck resurfacing and repairs. Class HT may be substituted for Class DT concrete. Additional requirements for Class DT concrete are: An approved water reducing admixture shall be incorporated in the mix. Class DT concrete shall contain a minimum of 50% AASHTO M 43 size No. 7 or No. 8 coarse aggregate. Approved fly ash may be substituted for portland cement up to a maximum of 20% Class C or 30% Class F by weight.

Class E concrete may be used for fast track pavements needing early strength in order to open a pavement to service soon after placement. Additional requirements for Class E concrete are: Type III cement may be used. Class E concrete shall contain a minimum of 55% AASHTO M 43 size No. 357 or No. 467 coarse aggregate. If all transverse joints are doweled, then Class E concrete shall contain a minimum of 55% AASHTO M 43 sizes No. 57, No. 67, No. 357, or No. 467 coarse aggregate. In addition to the compressive strength requirements in Table 601-1 and unless stated otherwise on the plans, Class E concrete shall achieve a field compressive strength of 17 MPa (2500 psi) within 12 hours. Laboratory trial mix for Class E concrete must produce an average 28 day flexural strength of at least 4482 kPa (650 psi). Approved fly ash may be substituted for portland cement up to a maximum of 30% Class F by weight.

Class H concrete is used for bare concrete bridge decks that will not receive a waterproofing membrane. Additional requirements for Class H concrete are: An approved water reducing admixture shall be incorporated in the mix. Class H concrete shall contain a minimum of 55% AASHTO M 43 size No. 67 coarse aggregate. Class H concrete shall contain cementitious materials in the following ranges: 267 - 297 kg/m3 (450 - 500 lbs/yd3) Type II portland cement, 53 - 74 kg/m3 (90 - 125 lbs/yd3) flyash and 12 - 18 kg/m3 (20 - 30 lbs/yd3) silica fume. The total content of Type II portland cement, flyash and silica fume shall be 344 - 380 kg/m3 (580 - 640 lbs/yd3). Laboratory trial mix for Class H concrete must not exceed permeability of 2000 coulombs at 56 days (ASTM C 1202). Laboratory trial mix for Class H concrete must not exhibit a crack at or before 14 days in the cracking tendency test (AASHTO PP 34).

Class HT concrete is used as the top layer for bare concrete bridge decks that will not receive a waterproofing membrane. Additional requirements for Class HT concrete are: An approved water reducing admixture shall be incorporated in the mix. Class HT concrete shall contain a minimum of 50% AASHTO M 43 size No. 7 or No. 8 coarse aggregate. Class HT concrete shall contain cementitious materials in the following ranges: 267 - 297 kg/m3 (450 - 500 lbs/yd3) Type II portland cement, 53 - 74 kg/m3 (90 - 125 lbs/yd3) flyash and 12 - 18 kg/m3 (20 - 30 lbs/yd3) silica fume. The total content of Type II portland cement, flyash and silica fume shall be 344 - 380 kg/m3 (580 - 640 lbs/yd3). Laboratory trial mix for Class HT concrete must not exceed permeability of 2000 coulombs at 56 days (ASTM C 1202). Laboratory trial mix for Class HT concrete must not exhibit a crack at or before 14 days in the cracking tendency test (AASHTO PP 34).

Class P concrete is used in pavements. Additional requirements for Class P concrete are: Class P concrete shall contain a minimum of 55% AASHTO M 43 size No. 357 or No. 467 coarse aggregate. If all transverse joints are doweled, then Class P concrete shall contain a minimum of 55% AASHTO M 43 sizes No. 57, No. 67, No. 357, or No. 467 coarse aggregate. Laboratory trial mix for Class P concrete must produce an average 28 day flexural strength of at least 4482 kPa (650 psi). Class P concrete shall contain 70% to 80% portland cement and 20% to 30% Class F fly ash in the total mass (weight) of cement plus fly ash. Unless acceptance is based on flexural strength, the total mass (weight) of cement plus Class F fly ash shall not be less than 392 kg/m3 (660 lbs/yd3). If acceptance is based on flexural strength, the total mass (weight) of cement plus Class F fly ash shall not be less than 309 kg/m3 (520 lbs/yd3).

Class S35 concrete is a dense high strength structural concrete. Additional requirements for Class S35 concrete are: An approved water reducing admixture shall be incorporated in the mix. Class S35 concrete shall be made with 19 mm (3/4") nominal sized coarse aggregate, i.e., 100% passing the 25.0 mm (1") sieve and 90% to 100% passing the 19 mm (3/4") sieve. When placed in a bridge deck, Class S35 concrete shall contain a minimum of 55% AASHTO M 43 size No. 67 coarse aggregate. Approved fly ash may be substituted for portland cement up to a maximum of 20% Class C or 30% Class F by weight.

Class S40 concrete is a dense high strength structural concrete. Additional requirements for Class S40 concrete are: An approved water reducing admixture shall be incorporated in the mix. Class S40 concrete shall be made with 19 mm (3/4") nominal sized coarse aggregate, i.e., 100% passing the 25.0 mm (1") sieve and 90% to 100% passing the 19 mm (3/4") sieve. When placed in a bridge deck, Class S40 concrete shall contain a minimum of 55% AASHTO M 43 size No. 67 coarse aggregate. Approved fly ash may be substituted for portland cement up to a maximum of 20% Class C or 30% Class F by weight.

Class S50 concrete is a dense high strength structural concrete. Additional requirements for Class S50 concrete are: An approved water reducing admixture shall be incorporated in the mix. Class S50 concrete shall be made with 19 mm (3/4") nominal sized coarse aggregate, i.e., 100% passing the 25.0 mm (1") sieve and 90% to 100% passing the 19 mm (3/4") sieve. When placed in a bridge deck, Class S50 concrete shall contain a minimum of 55% AASHTO M 43 size No. 67 coarse aggregate. Approved fly ash may be substituted for portland cement up to a maximum of 20% Class C or 30% Class F by weight. Laboratory trial mix for Class S50 concrete must not exhibit a crack at or before 14 days in the cracking tendency test (AASHTO PP 34).

Subsection 601.03 shall include the following:

Silica fume admixture shall conform to the requirements of subsection 701.03.

Calcium chloride shall not be used in any concrete unless otherwise specified.

Delete subsection 601.05 and replace with the following:

601.05 Proportioning. The Contractor shall submit a Concrete Mix Design Report consisting of design mix proportions, laboratory trial mix and aggregate data for each class of concrete being placed on the project. Concrete shall not be placed on the project before the Concrete Mix Design Report has been reviewed and approved by the Engineer. The Concrete Mix Design cannot be approved when the laboratory trial mix and aggregate data are the results from tests performed more than a year in the past. The design mix proportions shall show the weights and sources of all ingredients including cement, fly ash, aggregates, water, additives and the water cement ratio (w/c). When determining the w/c, cement (c) shall be the sum of the weight of the cement, the weight of the fly ash and the weight of silica fume.

The laboratory trial mix data shall include results of the following:

(a) AASHTO T 119 Slump of Hydraulic Cement Concrete.
(b) AASHTO T 121 Mass per Cubic Meter (Cubic Foot), Yield, and Air Content (Gravimetric) of Concrete. Air content from AASHTO T 152 Air Content of Freshly Mixed Concrete by the Pressure Method may be used in lieu of the air content by the gravimetric method in AASHTO T 121.

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(c) AASHTO T 22 Compressive Strength of Cylindrical Concrete Specimens shall be performed with at least two specimens at 7 days and three specimens at 28 days. Three additional specimens tested at 56 days shall be required for Class H and HT concrete.
(d) Class H and HT concrete shall include a measurement of permeability by ASTM C 1202 Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration. The concrete test specimens shall be two 2 inch thick disks sawed from the centers of two molded 4 inch diameter cylinders cured no more than 56 days in accordance with ASTM C 192 Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory.
(e) Class H, HT and S50 concrete shall include a measurement of cracking by AASHTO PP 34 Standard Practice for Estimating the Cracking Tendency of Concrete. The ring shall be cured in an indoor room with the temperature maintained 18°C - 24°C (65°F - 75°F) and relative humidity not exceeding 40%.
(f) Class E and P concrete shall include AASHTO T 97 Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading) performed with two specimens at 7 days and four specimens at 28 days.
(g) Class E concrete shall include a report of maturity relationships in accordance with ASTM C 1074 with the following additions or modifications. The Contractor shall provide a multi-channel maturity meter and all necessary wire and connectors. The Contractor shall be responsible for the placement and maintenance of the maturity meter and wire. Placement shall be as directed by the Engineer.

    1. The cylinders used to establish the compressive strength vs. maturity relationship shall be cast and cured in the field in conditions similar to the project.
    2. These cylinders shall be tested in pairs at times which yield compressive strengths three sets of which are at or below 17 MPa (2500 psi) and one of which is above 17 MPa (2500 psi).
    3. Testing to determine datum temperature or activation energy will not be required.
    4. A test slab shall be cast at the same time and location as the cylinders. The test slab shall have a length and width of 2 m x 2 m (6 feet x 6 feet) and a thickness equal to the pavement design thickness. The maturity of the test slab, when used in the compressive strength vs. maturity relationship from the cylinders, shall indicate that a compressive strength of 17 MPa (2500 psi) is achieved in the required time. Slab maturity will be determined with two probes located in the slab approximately 300 mm and 600 mm (1 and 2 feet) from the edge. The test slab shall be covered with a blanket similar to the one to be used on the pavement.

Except for class BZ concrete, the maximum slump of the delivered concrete shall be the slump of the approved concrete mix design plus 38 mm (1½”). Except for class H and HT concrete, the laboratory trial mix must produce an average 28 day compressive strength at least 115 percent of the required 28 day field compressive strength. The laboratory trial mix for Class H or HT concrete must produce an average 56 day compressive strength at least 115 percent of the required 56 day field compressive strength.

The laboratory trial mix shall have a relative yield of 0.99 to 1.02. When Portland Cement Concrete Pavement is paid with a volumetric pay quantity, the relative yield of the concrete produced on the project shall be 0.99 to 1.02. If the relative yield of the produced concrete does not conform to this range for two consecutive yield determinations, concrete production shall cease and the Contractor shall present a plan to correct the relative yield to the Engineer.

Aggregate data shall include the results of the following:

(1) AASHTO T 11 Materials Finer Than 75 um (No. 200) Sieve in Mineral Aggregates by Washing.
(2) AASHTO T 19 Unit Weight and Voids in Aggregate.
(3) AASHTO T 21 Organic Impurities in Fine Aggregate for Concrete.
(4) AASHTO T 27 Sieve Analysis of Fine and Coarse Aggregates.
(5) AASHTO T 84 Specific Gravity and Absorption of Fine Aggregate.

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(6) AASHTO T 85 Specific Gravity and Absorption of Coarse Aggregate.
(7) AASHTO T 96 Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.
(8) AASHTO T 104 Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate.
(9) AASHTO T 176 Plastic Fines in Graded Aggregates and Soils by use of the Sand Equivalent Test
(10) ASTM C 535 Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
(11) ASTM C 1260 Potential Alkali Reactivity of Aggregates (Mortar-Bar Method)

Any aggregate with an expansion of 0.10 percent or more at 16 days after casting as determined by ASTM C 1260 shall not be used unless mitigative measures are included in the mix design and subsequent results of CPL 4202 with the design mix proportions show an expansion not exceeding 0.10 percent at 16 days after casting. The Concrete Mix Design Report shall state what mitigative measures were included in the concrete mix design and include results for ASTM C 1260 and CPL 4202.

The Concrete Mix Design Report shall include Certified Test Reports showing that the cement, flyash and silica fume admixture meet the specification requirements and supporting this statement with actual test results. The certification for silica fume shall state the solids content if the silica fume admixture is furnished as slurry.

Where the Contractor's use of fly ash results in any delay, necessary change in admixture quantities or source, or unsatisfactory work, the cost of such delays, changes or corrective actions shall be borne by the Contractor.

The Contractor shall submit a new Concrete Mix Design Report meeting the above requirements when a change occurs in the source, type, or proportions of cement, fly ash, or aggregate. Unless otherwise permitted by the Engineer, the product of only one type of portland cement from one mill of any one brand shall be used in a concrete mix design.

Review and approval of the Concrete Mix Design by the Engineer does not constitute acceptance of the concrete. Acceptance will be based solely on the test results of concrete placed on the project.

Subsection 601.07 shall include the following:

Silica fume shall be added to the mix during initial batching.

Subsection 601.12 shall include the following:

At the pre-placement conference, the Contractor shall present a concrete winter protection plan for acceptance by the Engineer. The accepted concrete winter protection plan shall contain information on the number and type of heat sources to be used, a sketch detailing the enclosure materials, and all other pertinent information. Sufficient equipment shall be supplied to continuously maintain the specified temperature uniformly in all parts of the enclosure. Insulated blankets on top of the bridge deck and freely circulated artificial heat below the deck will be permitted.

Subsection 601.12(c) shall include the following:

Before concrete placement, all ice, snow, and frost shall be completely removed from within formwork. Salt shall not be used to thaw ice, snow, or frost.

Delete subsection 601.13 and replace with the following:

601.13 Curing Concrete Other Than Bridge Decks. When the ambient temperature is below 2°C

(35°F) the Contractor shall maintain the concrete temperature above 10° C (50°F) during the curing period. It shall be the Contractor's responsibility to determine for himself the necessity for undertaking protective measures.

The minimum curing period shall be determined by one of the following methods. The Engineer shall review for adequacy, the Contractor's determination of the curing period.

(1) The minimum curing period shall be 120 hours

(2) The minimum curing period shall be from the time the concrete has been placed until the concrete has met a compressive strength of 80 percent of the required field compressive strength. The Contractor shall cast information cylinders on the final portion of a placement and stored as close to the structure as possible. The information cylinders shall receive similar thermal protection as the structure. The contractor shall be responsible for the protection of the information cylinders. In-place strength shall be determined by at least two cylinders. If the information cylinders are destroyed in the field, the minimum curing period shall be 120 hours.

(3) The minimum curing period shall be from the time the concrete has been placed until the concrete has met a compressive strength of 80 percent of the required field compressive strength. The Contractor shall develop a maturity relationship for the concrete mix design in accordance with ASTM C 1074. The Contractor shall provide the maturity meter and all necessary thermocouples, thermometers, wires and connectors. The Contractor shall be responsible for the placement, protection and maintenance of the maturity meters and associated equipment. Locations where the maturity meters are placed shall be protected in the same manner as the rest of the structure. The Contractor shall install the thermocouples at locations designated by the Engineer. The Contractor shall monitor the temperature at intervals acceptable to the Engineer.

Maturity meters, thermocouples and information cylinders will not be measured or paid for separately, but shall be included in the work.

Enclosures with artificial heat sources will be permitted. If enclosures are used the Contractor shall monitor the structural integrity of the enclosure. Artificial heat sources shall not be placed in such a manner as to endanger formwork or expose any area of concrete to drying due to excessive temperatures. At the end of the curing period, the protection shall remain in place until it can be removed without permitting the concrete temperature to fall more than 28°C (50°F) in a 24-hour period. Sudden changes of concrete temperature shall be prevented.

Immediately after placing fresh concrete, all concrete shall be cured by one of the following methods. The Engineer shall review for adequacy, the curing method proposed by the Contractor.

(a) *Water Method.* All surfaces other than slabs shall be protected from the sun and the whole structure shall be kept wet throughout the curing period. Surfaces requiring a Class 2 finish may have the covering temporarily removed for finishing, but the covering must be restored as soon as possible. All concrete slabs shall be covered as soon as possible with suitable material so that concrete is kept thoroughly wet for at least five days. The concrete surface shall be kept moist at all times by fogging with an atomizing nozzle until the covering is placed.

(b) *Membrane Forming Curing Compound Method.* Curing compound may be applied only to those surfaces, which are to receive a Class I or Class 4 final finish. A volatile organic content (VOC) compliant curing compound conforming to AASHTO M 148, Type 2 shall be used on surfaces where curing compound is allowed, except that Type 1 curing compound shall be used on exposed aggregate or colored concrete, or when directed by the Engineer. Curing compound shall not be used on construction joints. The rate of application of curing compound will be in accordance with the manufacturer's recommendation, but shall not be more than 7 m^2/L (300 ft^2/g). All concrete cured by this method shall receive two applications of the curing compound. The first coat shall be applied immediately after stripping of forms and acceptance of the concrete finish. If the surface is dry, the concrete shall be thoroughly wet with water and the curing compound applied just as the surface film of water disappears. The second application shall be applied after the first application has set. During curing operations, any unsprayed surfaces shall be kept wet with water. The coating shall be protected against marring for a period of at least 10 days after application. Any coating marred, or otherwise disturbed, shall be given an additional coating. Should the surface coating be subjected continuously to injury, the Engineer may require that water curing, as described in subsection 601.13(a) be applied at once. When using a curing compound, the compound shall be thoroughly mixed within an hour before use. If the use of a curing compound results in a streaked or blotchy appearance, its use shall be discontinued. Water curing, as described in subsection 601.13 (a), shall then be applied until the cause of the defective appearance is corrected.

(c) *Form Method.* Concrete shall be protected by forms during the curing period Forms shall be kept moist, when necessary, during the curing period to insure the concrete surface remains wet.

(d) *Blanket Method.* Electrically heated curing blankets or insulation blankets may be used in cold weather to maintain specified curing temperature and to retain moisture in concrete. Blankets shall be lapped at least 200 mm (8 inches) and shall be free of holes. Blankets shall be secured at laps and edges to prevent moisture from escaping.

The following procedures shall be followed if the temperature of the concrete structure falls below 0ºC (32°F) before the concrete reaches 80 percent of the required field compressive strength:

(1) The Contractor will take cores at locations designated by the Engineer.
(2) The Engineer will take immediate possession of the cores and submit the cores to a petrographer for examination in accordance with ASTM C 856.
(3) All costs associated with coring, transmittal of cores, and petrographic examination shall be born by the Contractor regardless of the outcome of the petrographic examination.
(4) Concrete damaged by frost as determined by the petrographic examination shall be removed and replaced at the Contractor's expense.

Delete subsection 601.15 and 601. 16 and replace with the following:

601.15 Bridge Deck Placing, Consolidating and Finishing. The Contractor shall prepare a written Quality Control Plan (QCP) which defines the quality control measures the Contractor will use to ensure the placing, consolidating, and finishing, curing and weather protection of the bridge deck conforms to the Contract requirements. The Contractor may refer to the Structural Concrete Pre-Pour Conference Agenda in the department's Construction Manual for examples of items that should be included in the QCP. It shall also identify the Contractor's method for ensuring that the provisions of the QCP are met. The Contractor shall submit the QCP to the Engineer for written approval before the pre-pour conference.

A Pre-Placement Conference shall be held at a time mutually agreed upon before the initial placement of Class SF, Class H, Class HT or Class S50 concrete. Representatives of the ready mix producer and the Contractor shall meet with the Engineer to discuss the following topics:

(1) Concrete Mix Ingredients and Proportions (cement content, effect of admixtures, etc.)
(2) Work Schedule
(3) Applicable Specifications and Special Notes
(4) Delivery Details
(5) Planned Construction Joint Locations
(6) Role of All Personnel
(7) Construction Details - surface preparation, finish, joint locations, etc.

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(8) Testing Requirements
(9) Acceptance Criteria
(10) Contingency Plans for Wind, Rain, Breakdown, etc.
(11) Curing Details

(a) *Surface Preparation.* Tops of girders, precast deck panels, pier caps, and abutments that will come into contact with bridge deck concrete shall be heated to raise the temperature above 2°C (35°F) prior to concrete placement. The proposed preheating method is subject to approval by the Engineer.

Prior to placement of a Class HT concrete overlay, the deck shall be prepared as follows:

1. Newly Placed Decks or Existing Decks That Have Been Used as the Final Driving Surface. The deck shall be shot blasted in preparation for a mechanically bonded surface. Shot blasting shall remove the upper surface of the deck down to the coarse aggregate, which requires removing approximately 3 to 5 mm (1/8 to 3/16 inch) of the concrete.

2. Existing Decks Covered with One or More Layers of Bituminous Pavement. The deck shall be planed in accordance with subsection 202.09 to remove all overlying bituminous pavement, bridge deck membrane, and the upper 6.5 mm (1/4 inch) of the deck concrete.

If Class HT concrete is not placed within one week of shot blasting or planing, the area shall then be sandblasted and cleaned of all sand, concrete fragments, dirt, and other foreign material within one week before placement. The area shall be moistened at least two hours before placement in order that the substrate concrete is saturated. The substrate concrete shall be allowed to dry and shall be saturated surface dry and free of visible water at the time of placement.

(b) *Test Slab*. At least 7 days prior to initial placement of Class SF, Class H, Class HT or Class S50 concrete on or in a deck, the Contractor shall have prepared, placed, and cured one test slab of at least 3 m^3 (4 Cu. Yd.) to verify mix design, demonstrate the ability to perform placement, finishing & curing operations, and to check quality control. The test slab shall be approximately the same thickness as the concrete to be placed. Additional test slabs shall be placed as necessary to verify changes in design or procedures at the contractor's expense. Test slabs that are placed as acceptable work in segments of sidewalks, or as approach slabs, or other locations acceptable to the Engineer, will be paid for as the pay item for that element of the contract.

(c) *Placing*. Concrete shall be placed in accordance with the requirements of subsection 601.12 except for the following:

Concrete shall be placed in such manner as to require as little rehandling as possible and at sufficient depth to provide adequate material for screeding and finishing operations. The concrete shall be discharged as near its final location as practicable. The pattern of placement shall be such that lateral flow will be minimized. Concrete shall be placed against the leading edge of fresh concrete where practicable.

Class H, Class HT and Class S50 concrete shall be placed only when the concrete mix temperature is between 10°C and 27°C (50°F and 80°F) at the time of delivery. Class H, Class HT and Class S50 concrete shall not be placed in or on bridge decks when the air temperature exceeds 26°C (80°F) and/or the wind velocity exceeds 16 Km/h (10 mph) as determined by a digital thermometer and anemometer provided on site by the Contractor. If the Engineer can determine from the Contractor's data that the evaporation rate is less than 1.0 kg/m2/hr (0.20 lb/ft2/hr), in accordance with figure 2.1.5 in ACI 305, then Class H and HT concrete may be placed under these conditions.

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Longitudinal joints for a Class HT concrete overlay will be allowed only at the locations of lane lines and must be approved by the Engineer.

Transverse joints may be utilized when the Engineer determines that the work is not progressing in a satisfactory manner, or when required by change in weather conditions. The Engineer may approve transverse joint locations to accommodate phased overlay construction.

(d) *Consolidating*. Consolidation shall conform to subsection 601.12(e) and to the following:

The Contractor shall provide suitable mechanical vibrators to disperse the batch at the point of discharge and to densify the concrete within the forms. The bond of fresh concrete to concrete previously placed shall be achieved by vibrating the new concrete together with the old. Immersion vibrators shall operate at a speed of at least 10,000 vibrations per minute in air. Internal vibration may be used along the edges of forms and in areas of congested reinforcing. A combination of immersion vibration and surface consolidation shall be used.

(e) *Finishing*. Following consolidation, the concrete shall be struck off and finished by mechanical longitudinal floating, mechanical rolling, surface vibration, or a combination of any of these methods. Surface vibrators shall be of the low frequency, high-amplitude type, operating at a speed of 3000 to 4500 vibrations per minute.

A paver's steel scraping straightedge or lute, 100 mm (4 inch) maximum width, shall be the only hand tool permitted on deck surfaces, except for a minimum use of hand floats and edgers along the forms and in areas where machine finishing cannot be effectively used. Only minimum hand finishing will be permitted. If the surface of the deck slab becomes dry immediately following finishing operations, due to an excessive evaporation rate, it shall be covered with wet burlap or fogged with water covering the entire deck surface using pneumatic atomizing nozzles. The fog spray shall be just enough to retard surface evaporation and shall not change the water-cement ratio. During periods of excessive drying, a cover of wet burlap or plastic sheeting shall be maintained on the slab at all times until final cure is placed. Monomolecular film coatings applied to the surface of the slab to retain moisture may be used provided they effectively retard surface evaporation and are adequately maintained until the final cure is placed.

Surfaces of bridge decks and bridge approach slabs that will be the final riding surface shall be finished as follows:

1. For the final finish a seamless strip of plastic turf shall be dragged longitudinally over the full width of bridge deck after a seamless strip of burlap or other approved fabric has been dragged longitudinally over the full width of bridge deck to produce a uniform surface of gritty texture.

   The drags shall be mounted on a bridge other than the bridge to be furnished for department use. The dimensions of the drags shall be such that a strip of material at least 1 m (3 feet) wide is in contact with the full width of pavement surface while each drag is used. The drags shall consist of sufficient material and be maintained in such a condition that the resultant surface finish is of uniform appearance and reasonably free from grooves over 2 mm (1/16 inch) in depth. Where more than one layer of burlap drag is required, the bottom layer shall be approximately 150 mm (6 inches) wider than the layer above. Drags shall be maintained clean and free from encrusted mortar. Drags that cannot be cleaned shall be discarded and new drags installed.

2. Texturing. When posted speeds are 65 km/h (40 mph) or higher, the finish shall be a grooved finish conforming to the following:

   After the Engineer has accepted the finished surface, and after concrete has cured for at least seven days, the bridge deck surface shall be textured by grooving with a mechanized saw (sawed grooves). Grooving shall be done prior to the application of the concrete sealer. Only multi-blade saw cutting equipment furnished with circular blades may be used. Single blade equipment may be authorized by the Engineer where multi-blade assemblies do not allow sawing a distance one foot from obstructions.

The grooving shall be rectangular and conform to the following:

Depth: 3 mm ± 1 mm (? inch ± 1/32 inch)
Width: 3 mm ± 1 mm (? inch ± 1/32 inch)
Spacing: 20 mm ± 1 mm (¾ inch ± 1/32 inch) center to center

Grooves shall be longitudinal and parallel to the centerline of the roadway. Overlapping of grooves by succeeding passes will not be permitted. The grooves shall terminate 0.45 m (1.5 feet) from the face of curb or bridge rail on each side of the overlaid bridge deck.

Grooving to bridge joint system. For joint systems that are perpendicular to the roadway centerline, grooving shall extend to 225 mm ± 75 mm (9 inches ± 3 inches) from the armor of the joint.

For the joint systems that are not perpendicular to the centerline of the roadway, grooving shall remain parallel to the centerline and shall not be nearer than 150 mm (6 inches) to the joint armor nor farther than 1.2 m (4 feet) from the joint armor. The distance between grooves, from one side to other of the joint system, shall not exceed 1.5 m (5 feet).
The Contractor shall maintain the grooving equipment so that aggregate particles or cement build-up on the saws is promptly cleared or cleaned so that the grooves are neat, true and in conformance with the specified dimensions.

(f) *Surface Smoothness.*

1. All Bridge Deck Surfaces. Acceptability of the deck surface will be determined as follows: The Contractor shall furnish a 3 m (10 foot) straightedge or other approved device. When the concrete is sufficiently hard, the Contractor shall test the bridge deck surface with the 3 m (10 foot) straightedge or other approved device. Areas showing high spots of more than 3 mm ($^{1}/_{8}$ inch) but not exceeding 12 mm (½ inch) in 3 m (10 feet) shall be marked. The marked area shall be immediately ground with an approved grinding tool so that the surface deviation will not be in excess of 3 mm ($^{1}/_{8}$ inch) 3 m (10 feet). Grinding shall not reduce the concrete cover on reinforcing steel to less than 45 mm ($1^{3}/_{4}$ inches), (70 mm [$2^{3}/_{4}$ inches] for bare decks without an overlay). Decks that require additional corrective action shall be corrected with a concrete overlay approved by the Engineer.

2. All Bridge Deck Final Riding Surfaces. Bare deck, or any concrete overlayed final surface is subject to an incentive payment. The Contractor shall provide the Engineer with the following for incentive payment only: The longitudinal finished surface smoothness of structures and approach slabs including concrete deck and any overlaid surface shall be tested with the profilograph method in accordance with subsection 105.031(b). Bridge Deck shall be subject to an incentive payment in accordance with the following Table 601-3. Incentive Payments will be based on the Lane Profile Index (LPI) before diamond grinding of bumps or any corrective work has been done.

TABLE 601-3
BRIDGE DECK SMOOTHNESS (INCHES/MILE)

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2.5 mm (0.1 INCH) BLANKING BAND

PAVEMENT SMOOTHNESS CATEGORY[1]	INCENTIVE PAYMENTS				CORRECTIVE WORK REQUIRED
	LPI (in./mi.)	Concrete $/sq.yd.	LPI (mm/km)	Concrete $/m^2	
ALL BRIDGE DECKS	#12 or 12.1-15 15.1-18 18.1-22 22.1-25	$1.20 $0.90 $0.60 $0.30 $0.00	#190 191-235 236-285 286-345 346-395	$1.45 $1.08 $0.72 $0.36 $0.00	In accordance with subsection 601.15(d)1

[1] This category will be used only on new construction or complete reconstruction of bridge deck.

(g) *Movable Bridges.* Movable bridges or platforms shall be provided by the Contractor and moved as directed to allow the inspectors to work over the freshly placed plastic concrete. A movable bridge shall be kept as close to the finishing screed as practical. The deck of the movable bridges shall be a minimum of 600 mm (24 inches) wide and no more than 600 mm (24 inches) above the surface of the concrete and shall be capable of supporting two people. The Contractor shall provide additional movable bridges as appropriate for the work.

(h) *Concrete Bridge Sidewalks.* Bridge sidewalks shall receive a final transverse broom finish.

(i) If cracks in the deck concrete with a width of 0.9 mm (0.035 inches) or greater occur within two weeks of placement, those cracks shall be repaired at the Contractor's expense. Cracks will be measured by the Engineer by insertion of a wire gauge at any time and temperature within the two weeks. The repair shall consist of filling the cracks with a low viscosity, two part, methacrylate or an approved equal. The repair shall be in accordance with the recommendations of the manufacturer of the crack filling material.

601.16 Curing Concrete Bridge Decks. Except for Class H and HT concrete, the minimum curing period shall be 120 hours. The concrete surface shall be kept moist at all times by fogging with an approved atomizing nozzle or applying a monomolecular film coating to retard evaporation until the curing material is in place.

For Class H and HT concrete the minimum curing period shall be 168 hours and from May 1 and until September 30 the water cure method as described below shall be used without the membrane forming curing compound.

Concrete bridge decks, including bridge curbs and bridge sidewalks shall be cured as follows:

(a) Decks placed from May 1 to September 30 shall be cured by the membrane forming curing compound method followed by the water cure method as follows:

1. Membrane Forming Curing Compound Method. A volatile organic content (VOC) compliant curing compound conforming to AASHTO M 148, Type 2 shall be uniformly applied to the surface of the deck, curbs and sidewalks at the rate of 40 L/100 m^2 (1 gallon per 100 square feet). The curing compound shall be applied as a fine spray using power operated spraying equipment. The power operated spraying equipment shall be equipped with an operational pressure gage and a means of controlling the pressure. Before and during application the curing compound shall be kept thoroughly mixed by recirculation or a tank agitator. The application shall be within 6 m (20 feet) of the deck finishing operation. When the finishing operation is discontinued, all finished concrete shall be coated with curing compound within ½ hour. The curing compound shall be thoroughly mixed within one hour before use.

2. Water Cure Method. The water cure method shall be applied as soon as it can be without marring the surface and shall be continued for five days. The surface of the concrete, including bridge curbs and bridge sidewalks, shall be entirely covered with cotton, burlap, or combination polyethylene sheeting and burlap mats. Approved combinations of a barrier and a water retaining layer may be used. Prior to being placed, the mats shall be thoroughly saturated with water. The mats shall extend at least twice the thickness of the bridge deck beyond the edges of the slab and shall be weighted to remain in contact with the surface. The mats shall remain in contact and be kept wet for a minimum of five days after concrete placement.

(b) Decks placed between November 1 and March 31 shall be cured by application of a membrane forming curing compound followed by the blanket method as follows:

1. Membrane Forming Curing Compound Method. This method shall be applied in accordance with 601.16(a)1 above.

2. Blanket Method. Curing blankets with a minimum RValue of 0.5 shall be placed on the deck as soon as they can be without marring the surface. Blankets shall be loosely laid (not stretched) and adjacent edges suitably overlapped with continuous weights along the lapped joints. The blankets shall remain in place for a minimum of five days after placement.

(c) Decks placed in April or October may be cured in accordance with either subsection 601.16(a) or 601.16(b) above.

(d) For decks placed above an elevation of 2500 m (8,000 feet) above mean sea level, the Engineer may modify the time of year requirements for the cure methods defined in subsection 601.16(a) and 601.16(b) above.

(e) Class H, Class HT and Class S50 concrete shall be cured as follows:

1. The concrete surface shall be kept moist at all times by fogging with approved atomizing nozzles until the surface has been covered by the final cure.

2. At lease two atomizing nozzles shall be in operation at all times. A fogging nozzle that has shown acceptable performance is FOGG-IT Waterfog, low volume (7.5 liters per minute), manufactured by Fogg-it Nozzle Co. at P.O. Box 16053, San Francisco, California, 94116, or an approved equal.

3. From October 1 and until April 30 continuous fogging will not be required if the evaporation rate is less than 0.50 kg/m2/hr (0.10 lb/ft2/hr). Ambient temperatures during initial curing shall be warm enough that the water from fogging does not freeze before insulating blankets are applied. The internal concrete temperature shall be determined by using thermocouples and a continuous recording device. The Contractor shall provide the thermocouples and a continuous recording device and install the thermocouples at locations designated by the Engineer. The continuous recording device connected to the thermocouple shall be calibrated to provide accurate temperature readings. During the cure period the continuous recording device shall be visible, show visible readings, and the Contractor shall continuously monitor the concrete temperature and provide the recorded data to the engineer after the monitoring of temperature for that placement is complete.

(f) When the ambient temperature is below 2°C (35°F), the Contractor shall maintain the internal concrete temperature above 10°C (50°F) during the curing period, except the last 48 hours of the curing period the internal concrete temperature may be kept above 4°C (40°F).

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Internal concrete temperature shall be determined by using thermocouples. Thermocouple wire, connectors, and hand held thermometer will be supplied by the Engineer. The Contractor shall install the thermocouples at locations designated by the Engineer.

During the curing period, the Contractor shall monitor the enclosure at intervals acceptable to the Engineer. The Contractor shall monitor concrete temperature, and the structural integrity of the enclosure. Artificial heat sources shall not be placed in such a manner as to endanger formwork or expose any area of concrete to drying due to excessive temperatures.

During the curing period, for each day that the internal concrete temperature falls below the specified temperature, the protection shall remain in place and one extra day of curing time above 4°C (40°F) shall be added to the original days of protection.

If the internal concrete temperature at any location in the bridge deck concrete falls below 0°C (32°F) during the first 24 hours of the curing period, the Engineer may direct the Contractor to core the areas in question at the locations indicated by the Engineer. The Engineer will take immediate possession of the cores. The Engineer will submit the cores to a petrographer for examination in accordance with ASTM C 856. Concrete damaged by frost, as determined by the petrographer, shall be removed and replaced at the Contractor's expense. All costs associated with coring, transmittal of cores, and petrographic examination shall be born by the Contractor regardless of the outcome of the petrographic examination.

At the end of the protection period, the protection shall remain in place until it can be removed without permitting the concrete temperature to fall more than 28°C (50°F) in a 24-hour period. Sudden changes of temperature shall be prevented.

Subsection 601.17 shall include the following:

After the curing period for Class HT concrete has elapsed, the overlay shall be "sounded" by the Contractor in accordance with ASTM D 4580 Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding to determine if the Class HT concrete has bonded to the bridge deck. In areas where the Class HT concrete has not bonded to the bridge deck, it shall be removed and replaced at the Contractors expense.

Class HT concrete overlays shall not be opened to traffic, including construction traffic, for at least 14 days after placement. At the Engineer's discretion, the overlay may be opened to construction traffic sooner than 14 days but not until after the curing period has elapsed and the average strength of two field cured cylinders has reached 30 MPa (4500 psi). The field cured cylinders shall be made in accordance with AASHTO T 23 Making and Curing Concrete Test Specimens in the Field.

Subsection 601.18 shall include the following:

Bridge Deck Finish (Sawed Grooves) will be measured by the square meter (square yard). The area includes the length of the bridge and approach slabs, with deductions for areas occupied by expansion devices as specified, multiplied by the width of the roadway between the faces of curb or bridge rail on each side, less 0.9 m (3.0 feet). Bridge Deck Finish (Sawed Grooves) will not be remeasured but will be the quantity shown on the plans. Exceptions for each structure will be: (1) when field changes are ordered, or (2) when it is determined that there are discrepancies on the plans in an amount of plus or minus two percent of the plan quantity for the structure.

Subsection 601.19, 2nd paragraph shall include the following:

Pay Item	**Pay Unit**
Bridge Deck Finish (Sawed Grooves)	Square Yard
Bridge Deck Finish (Sawed Grooves)	Square Meter

Appendix D. Basics of the ultrasonic test and interpretation of the test data

D.1 Basic Principle

A transducer and a receiver are used to conduct the ultrasonic test. As shown in Fig. D.1, the transducer can be placed at a location T_x on a concrete surface, and the receiver can be placed at the location R_x. The distance between the two points is X_1. Ultrasonic signals are generated by the transducer, traveling through the concrete, and received by the receiver. Then, the same method can be repeated by placing the receiver at distances X_2, X_3, X_4 … Ultrasonic testing results shown in Fig. D.1 are the time required (in microseconds) for ultrasonic signals to travel between the transducer and receiver versus the distances between the transducer and receiver.

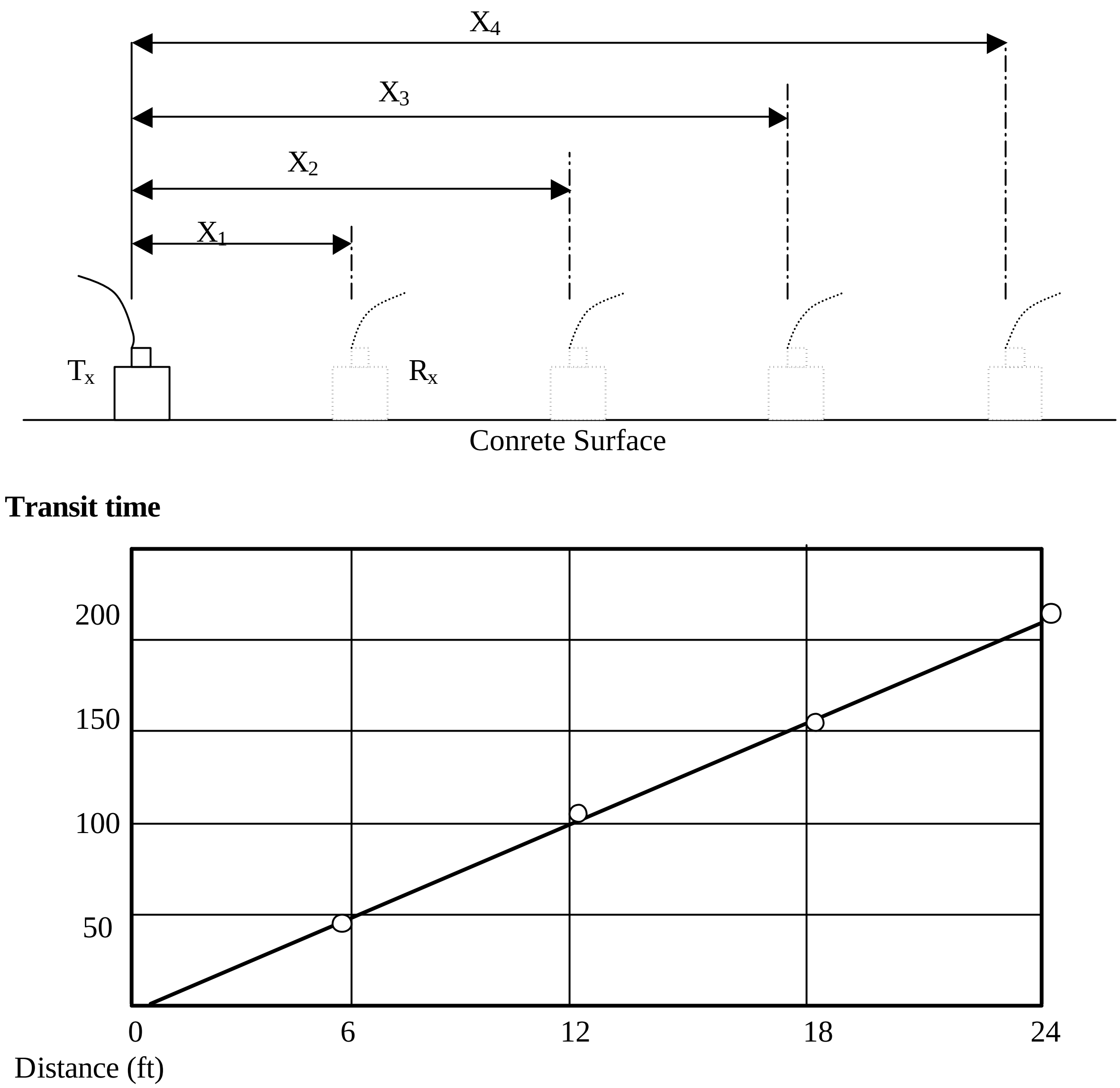

Fig. D.1 Signal arrival times vs. the distances between transducer and receiver

When the signals travel in concrete with good and uniform quality, the arrival times of the signal between the points should be proportional to the distances, since the signal velocity is a constant. Therefore, the slope of the curve (i.e., the velocity) is a constant, as shown in Fig. D.1. When the signals travel in concrete with voids or cracks, which act as barriers to the signals, the signals have to travel around the barriers. As a result, the arrival times will not be proportional to the distances, as shown in Fig. D.2. Therefore, if the points in an ultrasonic graph do not form a straight line or there is a discontinuity in the graph (see the dashed line in Fig. D.2), it is an indication that the density of the concrete is not a constant, which could be due to existing voids and/or cracks between the transducer/receiver positions.

The other indication of damage is an abrupt change in slope (as shown in Fig. D.3). This change in slope is a result of a slower pulse velocity near the surface than the velocity deeper within the concrete. The slower pulse velocity implies a layer of inferior low quality concrete. This layer could form as a result of damage by fire, frost, sulfate attack, etc.

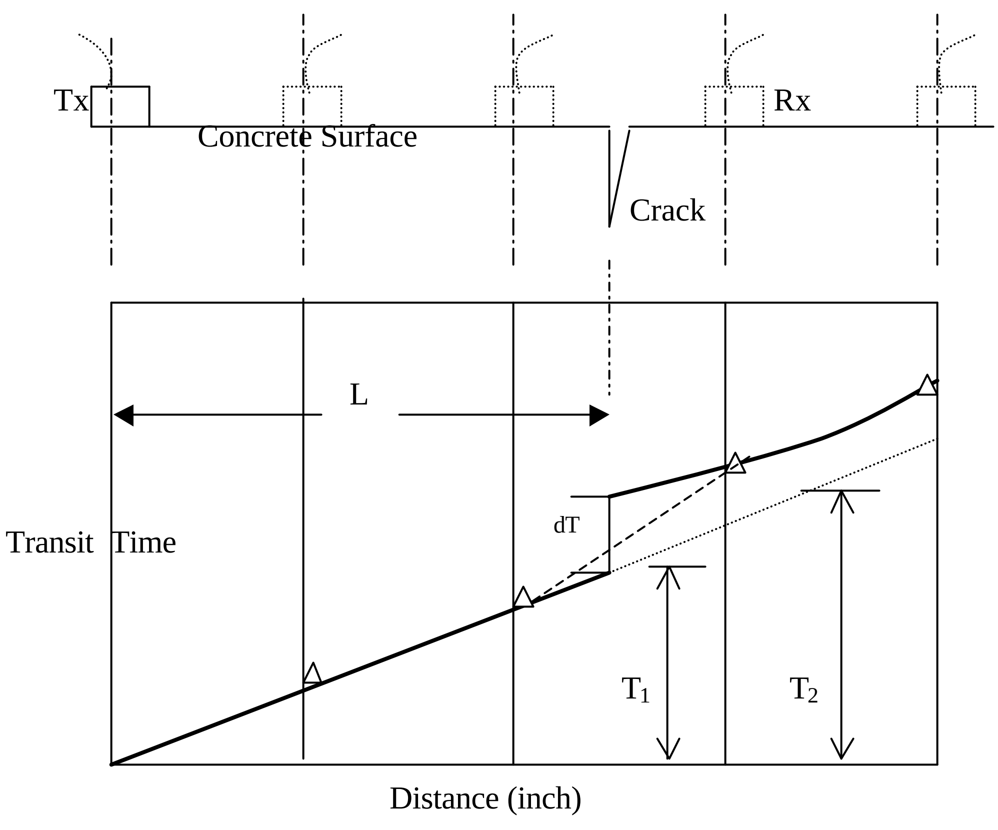

Figure D.2 Locating the cracks in concrete

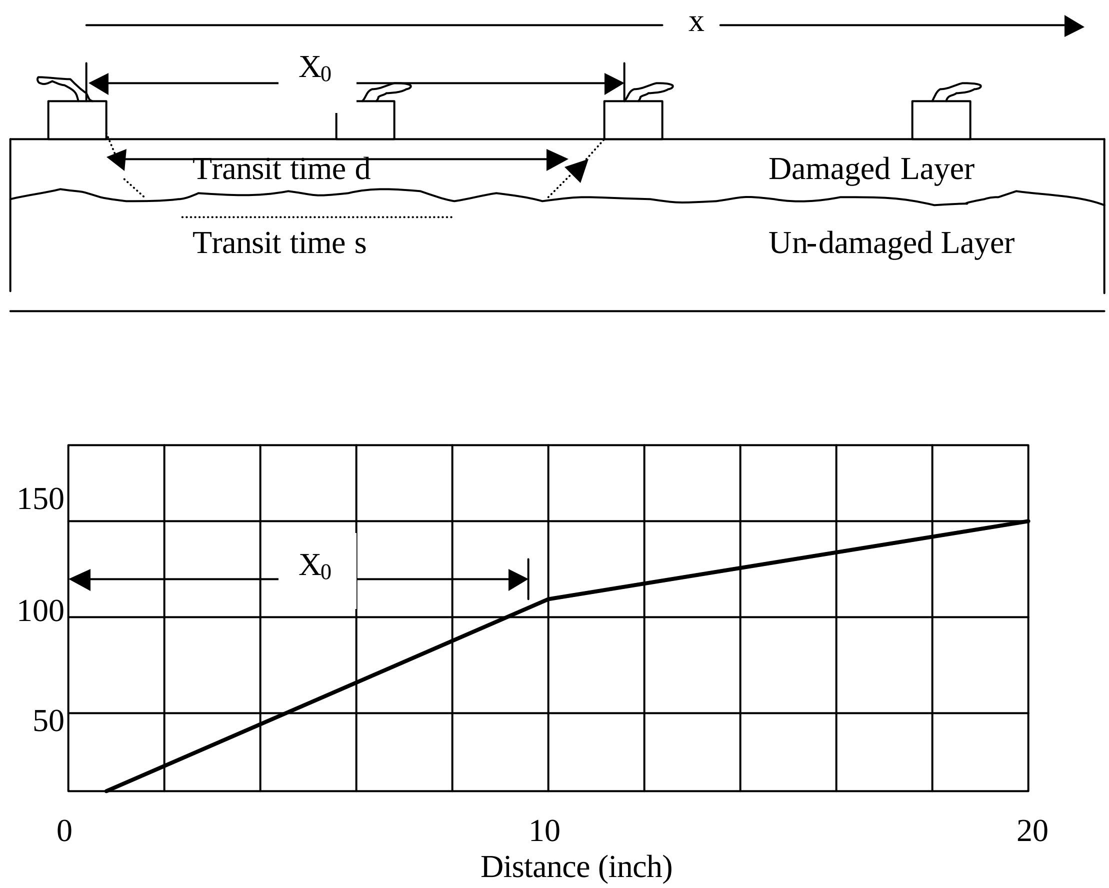

Fig. D.3 Investigation of surface layer damage in concrete

D.2 Demonstration of the Principle of Ultrasound Sound Test

Sample of the ultrasound results for the SH71/24 structure are provided below. Two locations with six different measurements around the damaged/cracked concrete were selected for the ultrasonic test as shown in Figure D.4. The results in terms of transit time (microseconds) versus distance (ft) were presented in Figures D.5 through D.10. Lines were constructed between coordinates of signal travel time and distance to determine the linearity between data points. From these figures, the typical damage encountered is cracking (partial or full depth). It can be seen that the cracks around receivers R_3 and R_4 (shown in Figure D.5) are in severe condition. This is evident by the significant discontinuity in the line.

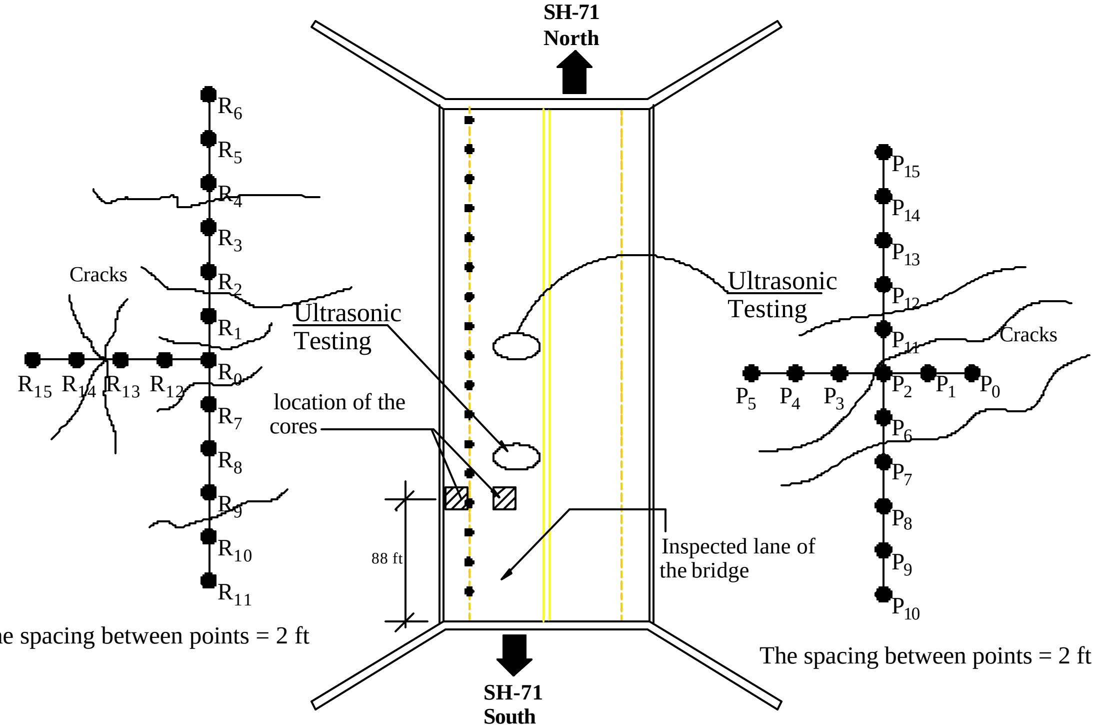

Fig. D.4: Sketch for the top view of SH-71 and US24 bridge.

Table D.1: Measured travel time along line R_0-R_6 (see Figure D.4).

Starting Point	**End Point**	**Distance (ft)**	**Measured travel time (micro-seconds)**
R_0	R_0	0	0
R_0	R_1	2	240
R_0	R_2	4	473
R_0	R_3	6	772
R_0	R_4	8	1180
R_0	R_5	10	1290
R_0	R_6	12	1632

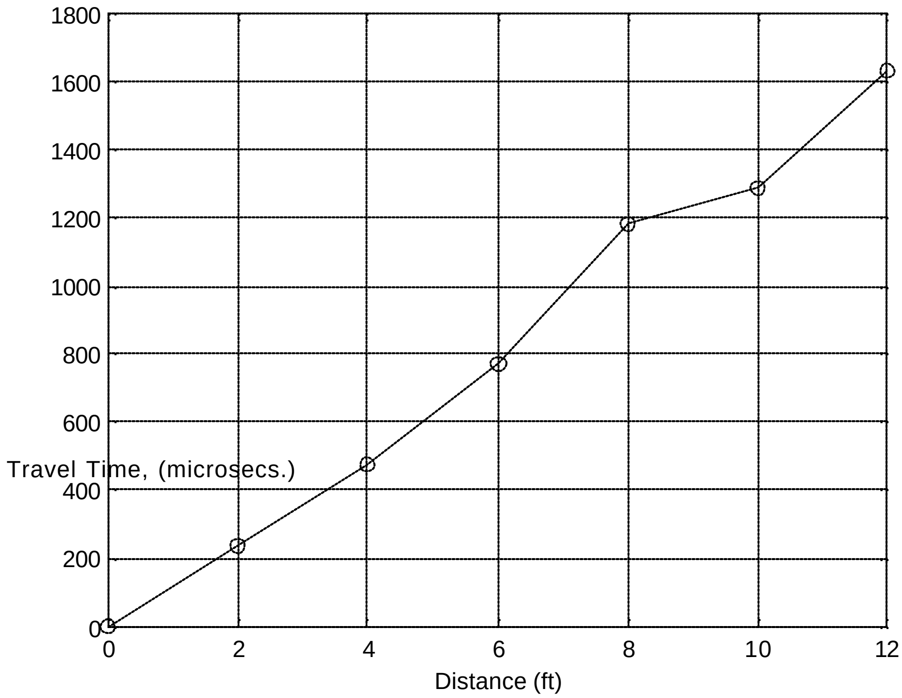

Fig. D.5: Travel time versus distance along line passing point R_0 through R_6 (as shown in Figure D.4)

Table D.2: Measured travel time along line R_0-R_{11} (see Figure D.4).

Starting Point	**End Point**	**Distance (ft)**	**Measured travel time (micro-seconds)**
R_0	R_0	0	0
R_0	R_7	2	197
R_0	R_8	4	512
R_0	R_9	6	773
R_0	R_{10}	8	1043
R_0	R_{11}	10	1310

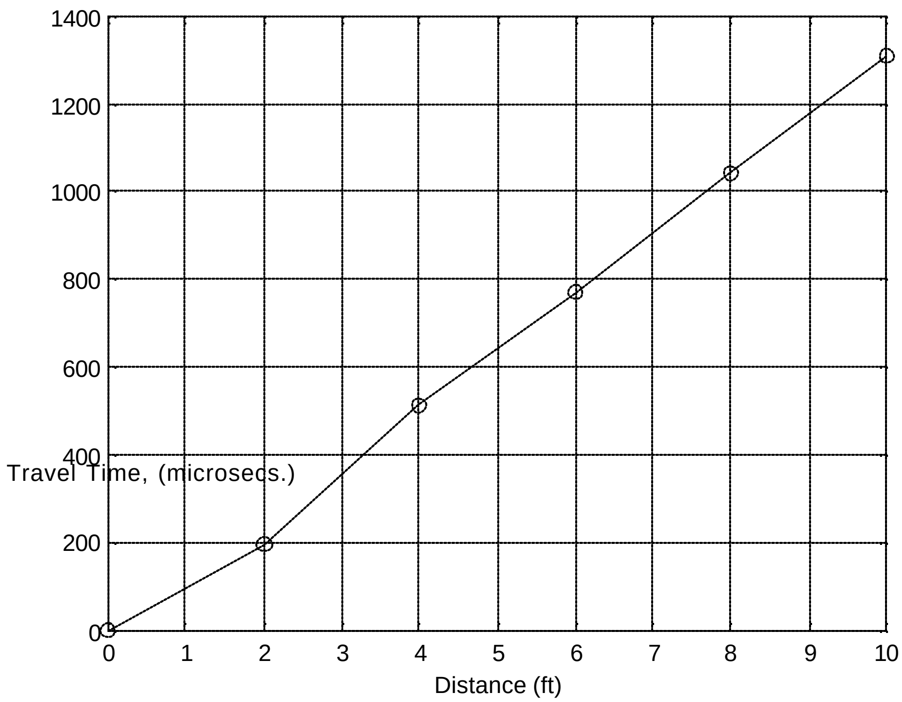

Fig. D.6: Travel time versus distance along line passing point R_0 through R_{11} (as shown in Figure D.4)

Table D.3: Measured travel time along line R_0-R_{15} (see Figure D.4).

Starting Point	**End Point**	**Distance (ft)**	**Measured travel time (micro-seconds)**
R_0	R_0	0	0
R_0	R_{12}	2	184
R_0	R_{13}	4	521
R_0	R_{14}	6	791
R_0	R_{15}	8	1147

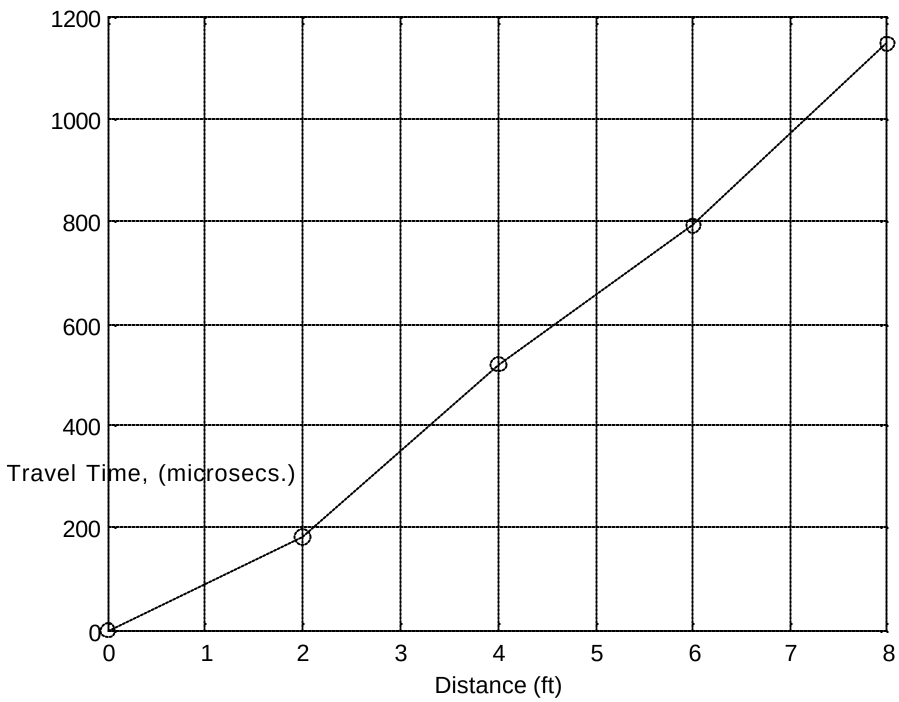

Fig. D.7: Travel time versus distance along line passing point R_0 through R_{15} (as shown in Figure D.4)

Table D.4: Measured travel time along line P_0-P_5 (see Figure D.4).

Starting Point	**End Point**	**Distance (ft)**	**Measured travel time (micro-seconds)**
P_0	P_0	0	0
P_0	P_1	2	243
P_0	P_2	4	499
P_0	P_3	6	760
P_0	P_4	8	1016
P_0	P_5	10	1286

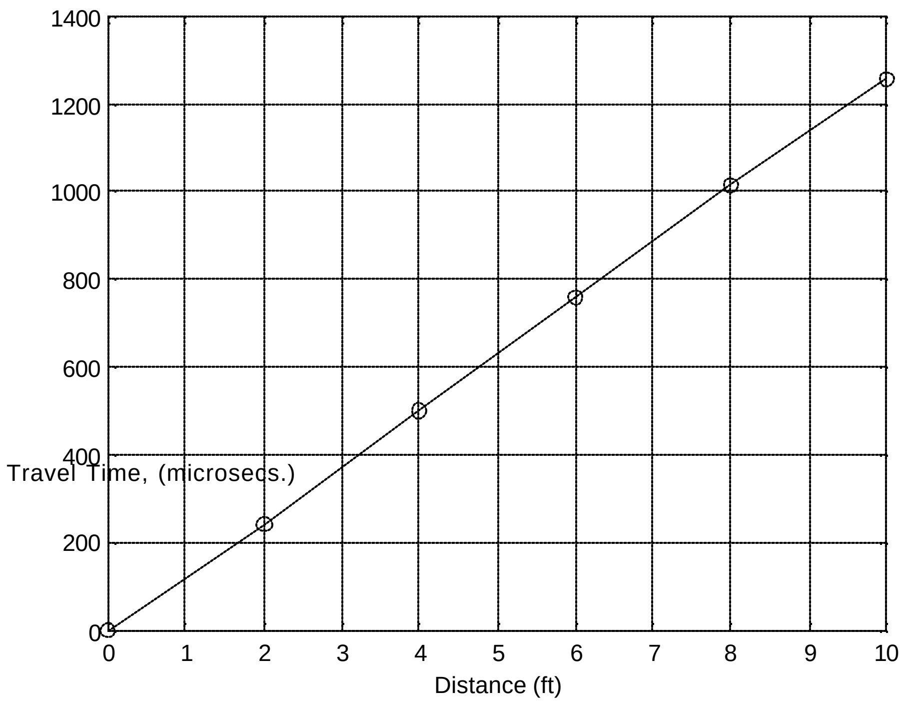

Fig. D.8:Travel time versus distance along line passing point P_0 through P_5 (as shown in Figure D.4)

Table D.5: Measured travel time along line P_2-P_{10} (see Figure D.4).

Starting Point	**End Point**	**Distance (ft)**	**Measured travel time (micro-seconds)**
P_2	P_2	0	0
P_2	P_6	2	275
P_2	P_7	4	562
P_2	P_8	6	822
P_2	P_9	8	1097
P_2	P_{10}	10	1346

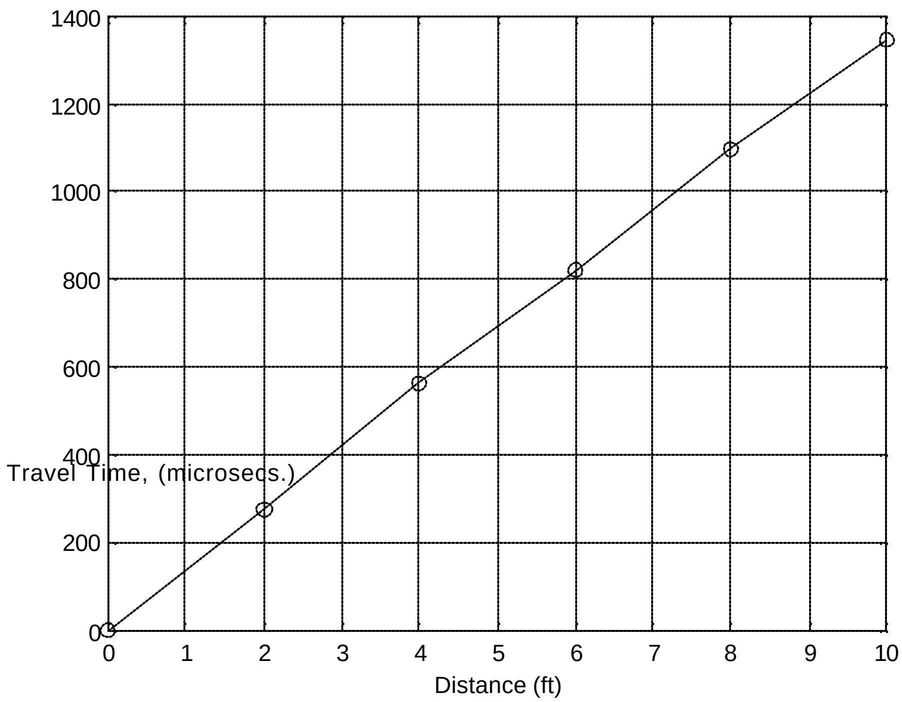

Fig. D.9: Travel time versus distance along line passing point P_2 through P_{10} (as shown in Figure D.4)

Table D.6: Measured travel time along line P_2-P_{15} (see Figure D.4).

Starting Point	**End Point**	**Distance (ft)**	**Measured travel time (micro-seconds)**
P_2	P_2	0	0
P_2	P_{11}	2	261
P_2	P_{12}	4	517
P_2	P_{13}	6	762
P_2	P_{14}	8	1080
P_2	P_{15}	10	1336

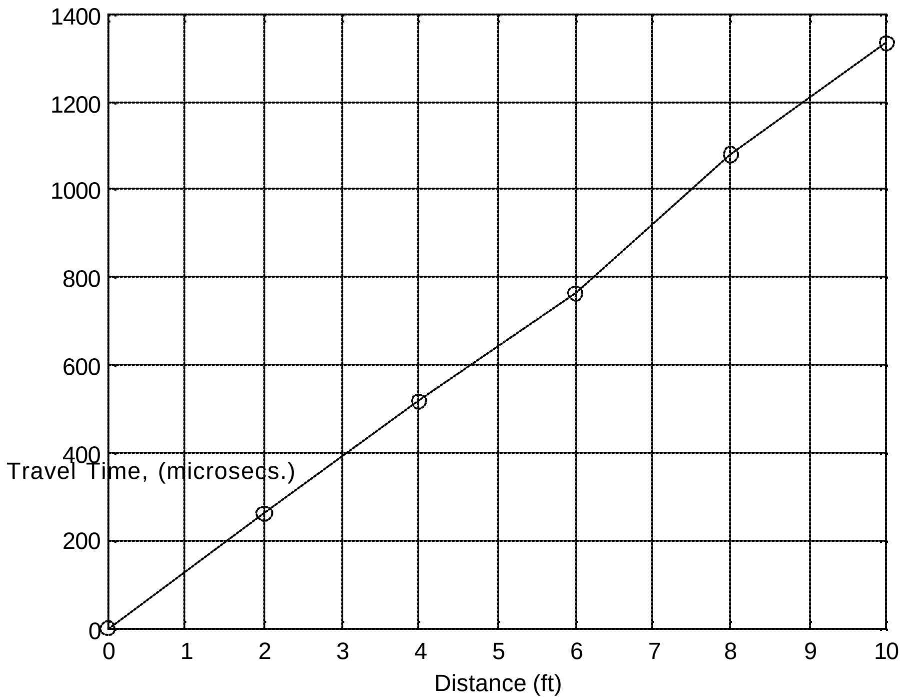

Fig. D.10: Travel time versus distance along line passing point P_2 through P_{15} (as shown in Figure D.4)

Appendix E. Crack Surveying Results

This appendix is not available online, due to its large size

Appendix F. Forensic Investigation of the Bridge Deck Cracking Problem at I-70 Structure over Clear Creek at Hidden Valley

F.1 Background

Several structures on I-70 at MP 243 (the Hidden Valley interchange) have a large number of cracks in their decks. Upon a request by the Colorado Department of Transportation (CDOT), the University of Colorado at Boulder (CU-Boulder) performed a quick study to investigate the causes of the early age deck cracking.

Figure F.1 F-15-BZ – the on-ramp from the frontage road to eastbound I-70

The bridge deck is about one year old and supported by prestressed concrete I girders. F-15-BZ – the on-ramp from the frontage road to eastbound I-70 is temporary closed to traffic, for examination in an effort to determine the cause of the cracks (see Fig. F.1). A field trip to the bridge was arranged on June 19, 2000 by CDOT Research Branch and Region 6. The research team from CU-Boulder inspected the transverse cracks on the bridge decks, took field notes and measurements on the cracks, discussed with field engineers about the construction process of the bridge, and collected some structural drawings. Then, the concrete materials used for the construction of the bridge decks were shipped to CU-Boulder for lab study. A detailed experimental study on the concrete materials was performed at CU-Boulder and the structural drawings were briefly reviewed.

F.2 Field observation

Transverse cracks were found in concrete decks throughout the entire bridge. Higher crack concentration occurred on both end spans of the bridge relative to those on the mid-span of the bridge. The distance between cracks are approximately 2.5 ft at the end span (see Fig. F.2) and

3.5 ft at mid-span (see Fig. F.3). At this stage, most of crack widths are less than 0.060 inches (1.5 mm), and most of the transverse cracks span throughout the deck. There were no longitudinal cracks observed.

Figure F.2 Crack pattern at the end span (2.5 ft spacing)

Figure F.3 Crack pattern at the mid-span (3.5 ft spacing)

F.3 Experimental study on properties of the concrete

F.3.1 Materials used for the bridge deck

Construction records show that the concrete deck was poured in the summer of 1999 from 7/26/99 till 7/28/99 (Sunny, temp. range: 51-91 F). There was no significant visible camber in the top of the deck. The following is information on concrete materials used for the bridge deck:

- Class: D/FA
- Field Strength: 4500 psi
- Lab Design Strength: 5175 psi
- Physical properties of trial batch
  Slump: 3.5 in.
  Air: 5.70 %
  W/C: 0.40
  Unit weight: 142.3 lbs/ft^3
  Yield: 1.01
- Compressive strength (psi) of trial batch:
  7 day: 4830
  28 days: 6130
- Concrete mix proportions (1 CUBIC YARD SSD BATCH WEIGHTS)

Table F.1 Concrete mix design

	Weight	Supplier	Type
Cement	595 lb	MOUNTAIN	I/IILA
Flyash	65 lb	BORAL	C
Coarse agg.	1780 lb	MOUNTAIN AGG, EMPIRE #67	
Sand	1147 lb	MOUNTAIN AGG, EMPIRE #6	
AEA	5.3 oz	MB MICRO AIR	
WRA	19.8 oz	MB PROKRETE N	
Water	270 lb	-	

F.3.2 Materials used in the laboratory study

The materials used in this experimental study, including cement, sand, gravel, fly ash, silica fume, and admixtures, were provided by CDOT. The materials are the same as those used for the construction at the Hidden Valley Bridge. We used the same mix design as the one used in the construction of the bridge. The amount of water was adjusted based on the absorption test data of the aggregates in our lab.

F.3.3 Specimen preparation and test results

Slump test and air content test were performed during the cast of concrete specimens. Then, the following tests were performed:

(1) Compressive strength at 3 days, 7 days, 28 days, and 56 days. 4” by 8” cylinders were used for the compressive strength test. Two cylinders for each test on 3 days, 7 days, 28 days, and 56 days.

(2) Rapid chloride permeability test at 28 days and 56 days. 4” by 2” cylindrical specimens were used for the permeability test. Two specimens for each test at 28 days and 56 days.

(3) Drying shrinkage test. Two concrete prisms of 3” by 3” by 12” were made for the drying shrinkage test. After 7 days of curing in a fog room, the prisms were removed from the fog room and placed in the lab (temperature = 22 °C and relative humidity = 35%). Shortening of the prisms due to drying shrinkage was then measured.

(4) Crack resistance test (Ring test). Two concrete rings of 6” height with outer diameter 18” and inner diameter 12” were made for the ring test. After one day of curing under room temperature, the mold was removed and the concrete ring was placed in the lab (temperature = 22 °C and relative humidity = 35%) until the first crack was observed.

Table F.2 lists all test data obtained in the Materials Laboratory at the University of Colorado at Boulder. The results of the drying shrinkage test are shown in Fig. F.4a.

Table F.2 Test data

Slump (inch)	Air (%)	Compressive Strength (psi)				Permeability (coulomb)		Initial Cracking (days)
		3d	7d	28d	56d	28d	56d	
3	5	3873	4586	5740	6528	7189	4573	14
3.5*	5.7*		4830*	6130*				

* Test data from the original trial batch by the concrete supplier.

F.3.4 Discussions on the test results

(1) Compressive strength

From the test results, it can be seen that the strength data obtained in our lab agree quite well with the data from the concrete supplier. The strength development looks good and the designed strength (5175 psi) is achieved.

(2) Chloride permeability

The test data of chloride permeability are high at both 28 days and 56 days. The high permeability of the concrete indicates low resistance to chloride penetration, implying possible earlier onset of steel corrosion in the concrete pavement in the future. In general, high permeability does not constitute a direct cause to early cracking in the concrete. However, high permeability is due mainly to high porosity in the cement paste, which is a control factor for concrete drying shrinkage, and the shrinkage could be a major cause of the cracking. At 56 days, the permeability is slightly above 4000 coulomb, which may be considered to be an indication for possible high drying shrinkage of the concrete.

(3) Drying shrinkage

As shown in Fig. F.4a, the drying shrinkage of the concrete has been measured up to 56 days. The shrinkage strain can generally be studied from three aspects. The first aspect is the maximum shrinkage strain at a certain age of the concrete. The second is the shrinkage rate at the age (i.e., the slope of the shrinkage curve). The third is the rate of shrinkage development at different ages.

More importantly, Fig. F.4a shows that the concrete used in the Hidden Valley Bridge has very large early age shrinkage, much higher than the shrinkage of the bridge deck concrete currently developed in our lab (see Fig. F.4b) for another project sponsored by CDOT. Comparing Figs. F.4a and F.4b, one can see that the Hidden Valley concrete has a drying shrinkage about 550 $\mu\varepsilon$ after 28 days of exposure to the environment, while the bridge deck concretes show about 250 $\mu\varepsilon$ at the same age. The early age shrinkage of the Hidden Valley concrete is doubled! Moreover, there is a sharp change in the shrinkage rate at about 14 days of exposure in Fig. F.4a.

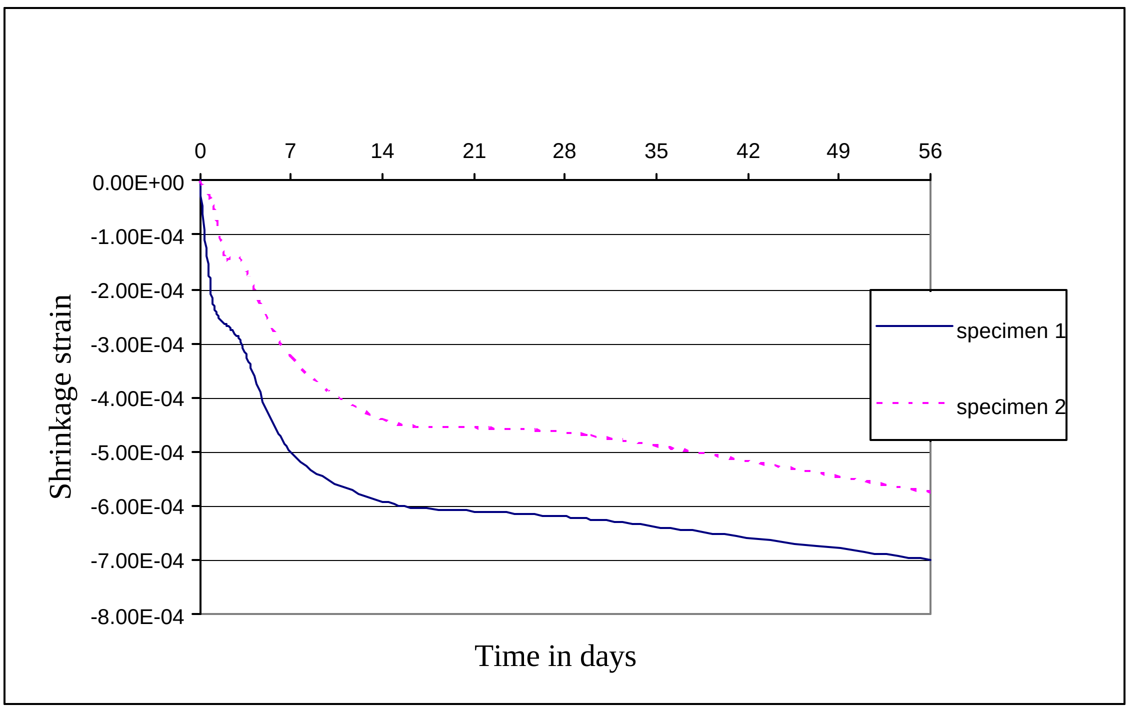

Figure F.4a Drying shrinkage data of the Hidden Valley concrete

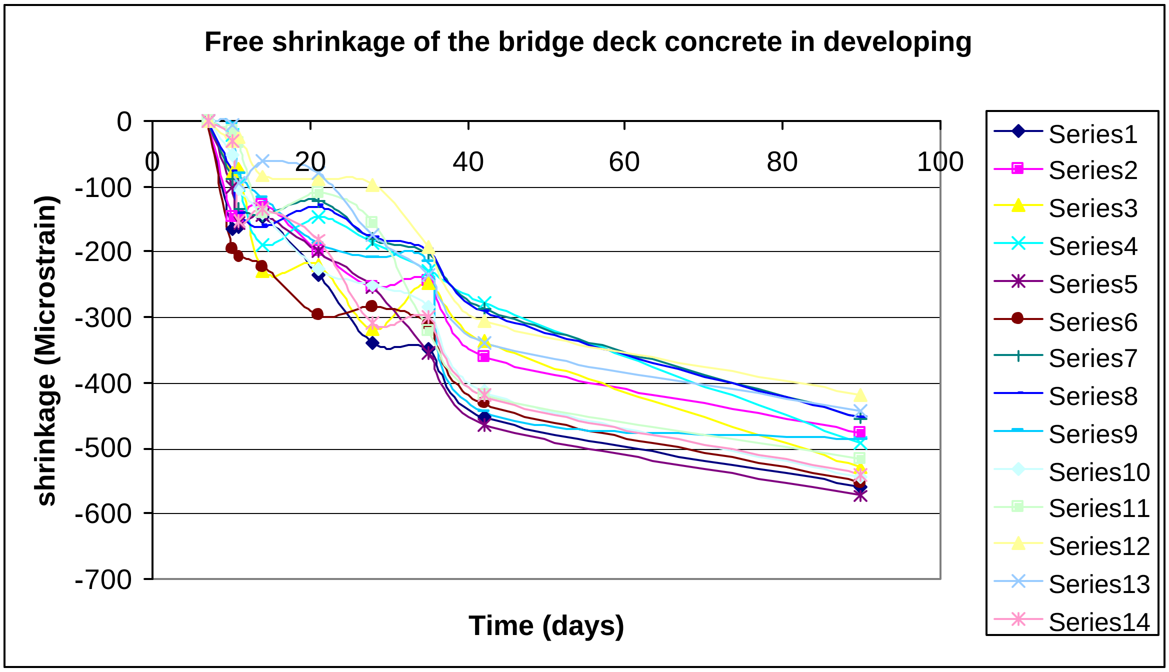

Figure F.4b Drying shrinkage data of recently developed bridge deck concrete

The shrinkage rate before 14 days is much higher than after 14 days. In both Fig. F.4a and Fig. F.4b, the shrinkage data were taken after 7 days curing. In Fig. F4a, the day after the seven-day curing was considered as the first day, while in Fig. F.4b, the plot starts from the first day of curing.

The high shrinkage rate and high shrinkage value at early age may be an important reason for the Hidden Valley concrete to develop severe early age cracking. By a further observation on the strength development of the Hidden Valley concrete, it is noticed that the Hidden Valley concrete exhibited a high early strength (possibly high final strength, too), 6130 psi at 28 day (tested by the concrete supplier) and 5760 psi at 28 days in our lab. High early strength means more C-S-H gel in the concrete matrix, which would result in high shrinkage in a dry condition.

(4) Ring test

The ring test provides combined information from two different aspects. One is the amount of restrained shrinkage of the concrete, and the other is the resistance to cracking of the concrete. The concrete showed an initial cracking at 14 days. Compared with test data in the literature, and compared with other test data obtained in our lab from other research projects, the Hidden Valley concrete shows early cracking, but not very early (other concrete mixes of low cracking potential in our lab have shown initial cracking times ranging from 11days to 20 days).

F.4 Conclusions and recommendations

Based on available results obtained from this quick study, we can provide some explanations on the causes of the transverse cracking at the Hidden Valley Bridge. The following is a brief summary of our finding and recommendations for further research, listed in the order of importance.

The drying shrinkage and shrinkage rate of the concrete used in Hidden Valley are very high at early ages, compared with other concrete mixes developed in our lab. The stress due to the drying shrinkage of the concrete overlay surpassed the tensile strength of the concrete. This may be the primary cause for the deck cracking. In a circumstance of high wind speed, for instance, it is quite possible for the concrete to develop very high drying shrinkage at its early age. Once the cracks form, they accelerate the moisture diffusion and further increase the drying shrinkage.

The real concrete mixture used in the construction may be different from the original mix design used in our lab. The difference can be determined by taking concrete cores from the bridge deck, conducting the same experimental study, and comparing the test results with our lab test data.