SD2005-11: Evaluation of Crack-Free Bridge Decks

FINAL REPORT

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DISCLAIMER

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16. Abstract

Low cracking high performance concrete (LC-HPC) was developed in the pooled fund project TPF-5(051) at the University of Kansas. The objective of this project was to evaluate the proposed draft specifications of LC-HPC and its potential implementation in South Dakota to reduce cracking in bridge decks.

As a part of the evaluation, constructability issues were studied for test slabs, A45 concrete decks, and LC-HPC decks. The results of field and laboratory tests indicated that A45 concrete and LC-HPC produced comparable properties in wet and hardened conditions. Petrographic studies on cores cut from the bridge decks included in this study revealed that the average coarse aggregate content was about 40% with a volume of permeable voids of 10.6 to 12.5%.

A pair of bridges in Pennington County and another pair of bridges in Brule County were selected for the evaluation. One bridge deck in each pair was constructed using A45 concrete while the other was constructed with LC-HPC. Four sets of crack surveys were performed for each bridge deck (i) prior to the bridges being open to traffic, (ii) one year, (iii) two years and (iv) three years later. Crack densities were determined based on the crack maps corresponding to the four crack surveys for each deck. The crack densities revealed that the current SD-DOT A45 concrete decks performed nearly as well as or better than the proposed LC-HPC bridge deck. The crack densities of both decks were comparable to those obtained for several other South Dakota bridge decks and those available in published literature for other states.

The study also revealed that it is feasible to implement LC-HPC in a bridge deck field environment. However, bridge decks using LC-HPC concrete are likely to cost more than the identical bridge decks made from A45. Therefore, it is recommended that more studies be conducted before considering switching from South Dakota A45 concrete to LC-HPC specifications.

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CHAPTER 1 EXECUTIVE SUMMARY

1.1 Problem Description

Research performed at the University of Kansas under the Pooled Fund project TPF-5(051), *Construction of Crack-Free Concrete Bridge Decks* proposes "best practices" dealing with materials, production and placement of concrete, and the relevant construction procedures to produce crack-free bridge decks. The proposed "best practices" include using a mix design that has less cement paste, densely graded aggregate, and increased air content. Incorporating these "best practices" is believed to reduce shrinkage cracking that occurs in bridge decks as the concrete cures. Extended moist curing and stringent control of evaporation during placement are also believed to reduce cracking. As a part of this program, it was required to select and schedule bridges that were constructed using the proposed "best practices" and carry out detailed crack surveys on the bridge decks, before the bridges are open to traffic, one year, two years and three years after the construction. The crack surveys were to be performed using the protocol developed at the University of Kansas.

An evaluation of the proposed draft specifications (see Appendix A) for low-cracking high-performance concrete (LC-HPC) and its potential implementation for bridge decks was done as a part of the project. The primary focus of the project was the comparison of constructability and cracking behavior of the bridge decks made with LC-HPC concrete and those made with SDDOT's existing specifications using A45 concrete. Two pairs of new bridges were provided by SDDOT for this research. One bridge deck in each pair was constructed using the new LC-HPC specifications, and the other companion bridge deck was constructed using the existing A45 concrete specifications. The research bridges have the following design details:

- two single-span composite steel girder bridge, 208.5′ x 40′ roadway (structure numbers 52-435-289 and 52-435-288) as part of project IM90-2(134)59 PCN 4259, I-90 over East North Street in Pennington County;

- two three-span composite prestressed concrete girder bridges, both 173.6′ x 40′ roadway, (structure numbers 08-230-130 and 08-230-131) as part of IM-90-6(37)281 PCN 1245, I-90 over a county road in Brule County.

Due to the proximity of the bridge construction sites, the researchers from SDSMT studied the cracking behavior of the bridge decks located in Pennington County, and the researchers from SDSU proposed to study the cracking behavior of bridges built in Brule County. The general approach and methodology of research however was uniform and consistent between the two groups of researchers because this is a collaborative research project with close cooperation between the two universities.

This final report details the findings from the study. The actual research work was shared between SDSMT and SDSU. Each university worked in close coordination and collaboration with the other, but produced crack maps and performed analysis independently.

1.2 Research Objectives

The project had the following objectives:

1. Evaluate the constructability of the bridge decks with proposed Low-Cracking High Performance Concrete (LC-HPC) material and construction specifications proposed by Pooled Fund project TPF-5(051) as implemented by SDDOT special provisions.

2. Evaluate the performance of the bridge decks with respect to crack densities.

3. Evaluate the suitability of the draft Pooled Fund material and construction specifications for construction of crack-free bridge decks in South Dakota.

1.3 Description of Project Tasks

The research methodology followed for each project task is summarized in this section. The findings from each project task are briefly described. The detailed descriptions of each task are given in different chapters of this final report.

Task 1. Meet with the technical panel to review project scope, discuss construction issues, and present tentative work plan.

The researchers attended a planning meeting held at the SDDOT main office in Pierre, SD on May 16, 2005 where the researchers from SDSMT and SDSU met with the technical panel of this project. A technical panel meeting for the project took place between the researchers and the technical panel on July 13, 2005 in SDDOT main office in Pierre, SD. Several issues and tentative schedules for the bridge deck construction were discussed at the meeting.

Task 2. Conduct a literature search and compare the results of bridge construction practices and specifications in SDDOT with other states as they relate to crack-free bridge deck construction.

A detailed literature search was performed and is presented in Chapter 4. The search revealed that even after significant research effort and resources were devoted to solving the problem of bridge deck cracking, the cracking in bridge decks is still prevalent in old and recently constructed bridge decks (more so in newer bridges) within the United States and Canada, and the solution seems to be ever elusive. SD-DOT devoted significant efforts to minimize cracking in bridge decks in the State for the last several years. Success was reported in minimizing cracks 5 to 10 fold in a recent South Dakota project on optimized aggregate gradation for structural concrete.

The University of Kansas pooled fund project on minimizing cracking in bridge decks is one of the largest ongoing projects in the country. Through carefully laid out material specifications and construction methods, LC-HPC is reported to have reduced bridge deck cracking significantly in some Kansas bridge decks. An extensive list of publications dealing with bridge deck cracking and strategies to minimize cracking are also presented in Chapter 4.

A transportation agency questionnaire launched by SDSU researchers showed that roughly half of the respondents have performed research into reducing bridge deck cracking and have special techniques for placing low-cracking concrete. These agencies have reported cracking to be problematic and have used the following methods to reduce cracking: (i) 4 day moist cure and a waterproof membrane, (ii) silica fume and fly ash fillers with a 14 day moist cure, (iii) controlling weather conditions during placement, (iv) using different placement sequences, wet cure process, using set retarders, fiber reinforcing, and high performance concrete.

The questionnaire also determined how different agencies monitor and perform deck inspections. To repair cracks, the agencies used methods of sealing cracks on new decks and flood-coats on older decks, epoxy injections, rigid concrete overlays, and bituminous or penetrating sealers.

Trends in bridge structure type and cracking were also investigated through the questionnaire. Agencies reported LC-HPC to be having higher initial costs when compared to a conventional concrete due to research and maintenance costs. Complete details of the responses to the questionnaire are given in Chapter 4.

Task 3. Finalize a work plan incorporating previous SDDOT and other research results for

monitoring construction and evaluating performance of the experimental bridges.

This task was completed in a timely manner.

Task 4. Conduct shrinkage tests using SDDOT standard mix designs with fly ash and mix designs with reduced cement content and no fly ash, using South Dakota materials, and compare the results.

The test results indicate that the drying shrinkage of A45 concrete mixed in SDSMT laboratory and tested per ASTM C157 was about 240 x 10^{-6}, while that of LC-HPC was about 130 x 10^{-6}. Both shrinkage strains are very low and are within acceptable limits suggested by others, but LC-HPC showed much lower shrinkage strains at 60 days than the corresponding A45 concrete. This task is described in Chapter 6.

Task 5. Document the concrete physical properties (seven, fourteen, and twenty-eight day compressive strengths, air content, slump, and temperature) of SDDOT-furnished mix designs, for the control bridges and the LC-HPC concrete used on this project in the final report.

SDDOT engineers recorded properties of concrete for the four bridge decks and supplied these records to the researchers. The properties recorded were wet concrete properties measured at the location of the pours, compressive strength results as developed by the DOT personnel at different ages, and temperature measurements. Summaries of the records are provided in Chapter 7.

The strength development of A45 and LC-HPC concretes is reasonably close to each other. A45 concrete seemed to develop strength at a slower rate, but the strengths are within 200 psi at

28 days. The plots of test results demonstrated that there is no discernable difference between the compressive strength of A45 concrete and that of LC-HPC concrete at 28 days.

Task 6. Monitor construction of the control bridge decks and the LC-HPC bridge decks, including test slabs, and document the results. Submit an interim report, after construction of each of the four bridges, detailing constructability issues and suggestions of best practices for use on future bridges.

The constructability issues are discussed in detail in Chapter 8. Several problems were encountered during the construction of the test slabs, A45 control bridge decks and LC-HPC bridge decks.

Some of the major issues include maintenance of the specified wet concrete properties, placement of concrete, finishing, fogging, and placing curing material within 10 minutes of strike off. The contractor gave feedback suggesting that the construction using LC-HPC concrete cost them more than the corresponding construction using A45 concrete.

Task 7. Conduct an extensive crack survey of each of the four bridges prior to its being opened to traffic and annually for three years after opening to traffic. The crack surveys will be conducted according to the protocol developed as part of Pooled Fund TPF-5(051) Construction of Crack-Free Concrete Bridge Decks.

The primary focus of this project was the comparison of cracking performance of LC-HPC bridge decks with that of standard SDDOT A45 concrete bridge decks. The following four sets of crack surveys were conducted for the control bridge decks and the LC-HPC bridge decks:

(i) prior to the bridges being open to traffic
(ii) one year after being open to traffic

(iii) two years after being open to traffic and

(iv) three years after being open to traffic

Fig. 1.1 shows the comparison of crack densities for the two types of bridge decks in Pennington County. The crack density of A45 bridge deck after three years of service was found to be 0.73 m/m^2 while the crack density of LC-HPC bridge deck after three years of service was found to be 0.85 m/m^2. These crack densities reveal that the current SD-DOT A45 concrete deck performed better than the proposed LC-HPC bridge deck in Pennington County.

Comparison of crack densities for the two types of bridge decks in Brule County is shown in Fig. 1.2. The figure shows that the crack density of A45 bridge deck after three years of service was practically the same as the crack density of LC-HPC bridge deck after about three years of service.

Complete details of the crack surveys and the crack maps for the four new bridges are given in Chapter 9.

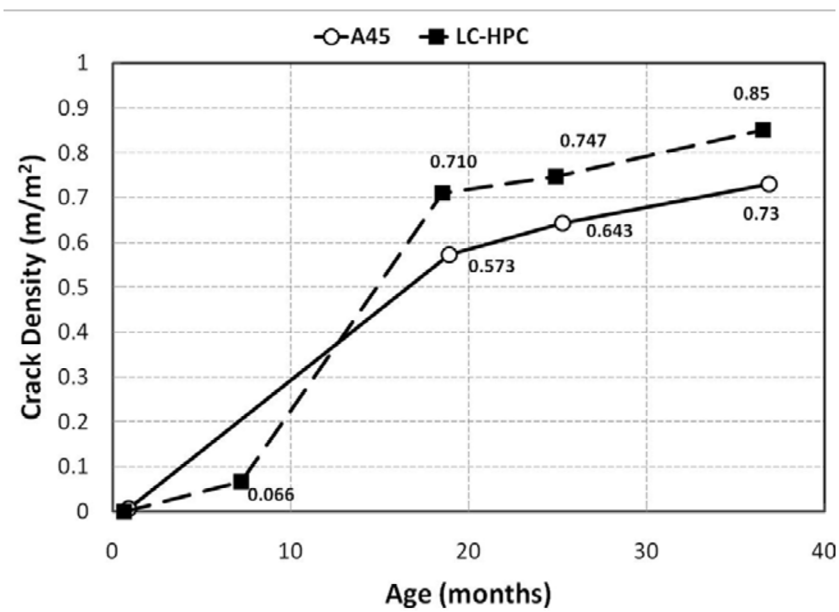


Fig. 1.1 Summary of Crack Densities for Pennington County Bridges

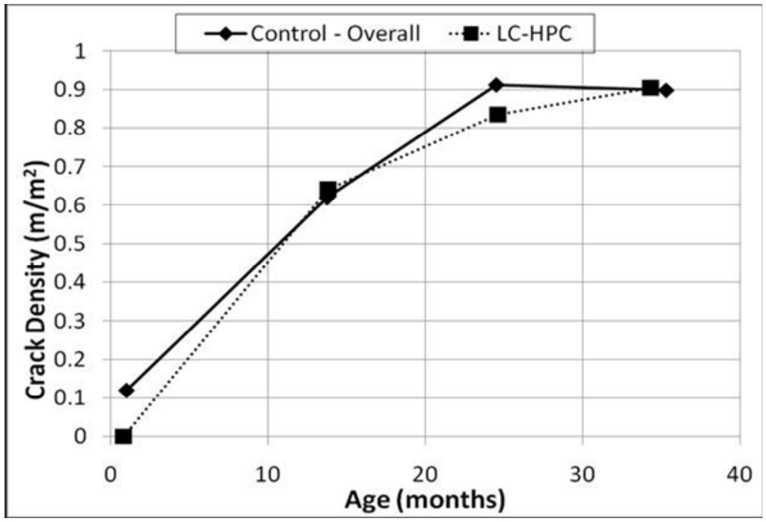


Fig. 1.2 Summary of Crack Densities for Brule County Bridges

Task 8. Conduct crack surveys on a minimum of two of each type of South Dakota bridges constructed prior to this research using: old mix design without addition of fly ash; more recent mix design with fly ash but without optimized aggregate; and current mix design with optimized aggregate and fly ash.

Details of the crack surveys conducted on existing bridge decks in South Dakota are given in Chapter 10. The results of the crack survey are summarized in Table 1.1. Steel girder bridges with optimized aggregate and fly ash cracked less than prestressed girder bridges with current SD-DOT mix with fly ash. Both types of bridges had higher crack intensities in negative moment regions and the interior spans.

Table 1.1 Summary of Crack Densities of Existing South Dakota Bridge Decks

Structure No.	Date Built	Girder Type	Deck Type	Deck Length (ft)	Deck width (ft)	Deck Thickness (in)	Skew Angle	Crack Density m/m^2
56-118-127	9/2003	Prestressed	Current SDDOT mix with flyash	332	36	8.00	15° RHF	0.72
56-118-128	9/2004	Prestressed	Current SDDOT mix with flyash	332	36	8.00	15° RHF	0.92
18-180-100	9/1988	Steel	Old SDDOT mix without flyash	307	40	8.25	0°	0.34
09-126-149	9/1999	Steel	Old SDDOT mix without flyash	268	36	8.75	0°	1.16
50-177-199	9/2004	Steel	Current SDDOT mix with flyash and optimized aggregate	374	56	8.00	0	0.40
50-178-199	9/2004	Steel	Current SDDOT mix with flyash and optimized aggregate	374	56	8.00	0	0.66

Task 9. Compare current SDDOT materials and construction specifications to the draft Pooled Fund material and construction specifications being developed for crack-free bridge decks with respect to constructability, South Dakota materials, and concrete plant capabilities and recommend best practices for future projects.

This task is addressed in several parts of the final report. However, the advantages of using LC-HPC concrete in South Dakota bridge decks have not been fully realized in this project. Therefore, there seems to be no merit at the moment in recommending best practices for future projects because it is unsubstantiated (from a crack density point of view) that LC-HPC is resulting in reduced cracking in bridge decks compared to South Dakota A45 concrete mix.

Task 10. Prepare a final report and executive summary of the prior research, research methodology, findings, conclusions and recommendations.

This task is satisfied through the publication of this report.

Task 11. Make an executive presentation to the SDDOT Research Review Board at the conclusion of the project.

The project has been successfully completed. The final presentation was given at the RRB meeting on December 16, 2010.

Supplemental Task S1. Perform Petrographic Analysis of Eight Concrete Cores Cut from Four Bridge Decks (two from each type)

Physical inspection of the concrete cores cut from the test slabs, A45 concrete deck, and LC-HPC deck showed no discernable difference between the microstructure of the cores. The details of the physical inspection are given in Chapter 12. Air voids, aggregate content, and distribution, and an estimate of density, absorption and void content were determined from a petrographic analysis conducted at SDSMT. The quartzite cores (Brule County) tend to have slightly greater air void content and slightly greater total aggregate content than the limestone cores (Pennington County). The concrete consolidation for all the bridge decks was very good. The average coarse aggregate content in the cores was found to be about 40%. The volume of permeable voids for these concretes ranged from 10.6 to 12.5%. Complete details of petrographic evaluation of the eight concrete cores are given in Appendix D.

1.4 Findings and Conclusions

This study compares the constructability of LC-HPC with A45 concrete for bridge deck applications in South Dakota. Wet concrete properties, compressive strength development, and drying shrinkage properties of the two types of concrete were studied and compared. The cores cut from the test slab, A45 concrete deck slab, and LC-HPC deck slab were physically and petrographically examined. Extensive crack surveys were conducted on the top surfaces of four bridge decks. Two bridge decks were located in Pennington County and two others were located in Brule County. One bridge deck from each pair was constructed from standard South

Dakota A45 mix concrete while the other was constructed with the new proposed LC-HPC concrete. The following conclusions are drawn from the study:

(a) It is feasible to make, place, consolidate and cure LC-HPC with all the relevant special requirements in a bridge deck placement environment. The wet concrete properties specified in the special specifications were achievable. The compressive strength and static modulus of LC-HPC were about the same as that of A45 concrete at 28 days. The drying shrinkage of LC-HPC made in the laboratory at 60 days was about 100×10^{-6} less than that of A45 concrete which by itself is very low.

(b) The cores cut from test slab, A45 deck and LC-HPC deck showed similar microstructure and physical properties, indicating that the physical structure of LC-HPC is similar to that of A45 concrete, and there was very good consolidation of concrete for all the decks. The average coarse aggregate content in the cores was found to be about 40%. The volume of permeable voids for these concretes ranged from 10.6 to 12.5%.

(c) The crack densities that were determined in this project indicate that currently used SD-DOT A45 concrete deck had a lower crack density of 0.73 m/m^2 after three years of service compared to the crack density of 0.85 m/m^2 for LC-HPC at about the same time of service in Pennington County. In Brule County, the crack density of A45 concrete bridge deck was 0.9 m/m^2 after three years of service. The crack density of LC-HPC bridge deck was also found to be 0.9 m/m^2 after three years of service. These crack densities reveal that South Dakota A45 mix performed nearly as well as or better than the proposed new LC-HPC mix for the four bridge decks evaluated in this project.

(d) The crack surveys conducted for the new and old bridge decks in South Dakota demonstrated that:

- LC-HPC had less early age (30 days) bridge deck cracking than A45 concrete.

- For both A45 and LC-HPC bridge decks, the crack densities of cracking increased with time.
- The existing steel girder bridges with optimized aggregate gradation concrete deck showed less cracking than prestressed girder bridges. Both types of bridges had higher crack densities in negative moment regions and interior spans.
- Other transportation agencies have investigated bridge deck cracking and have implemented different methods to control deck cracking. There seems to be no geographical trends for bridge deck cracking problems.

1.5 Recommendations

The procedures used in the construction of the LC-HPC bridge decks worked well. The following are suggested recommendations:

1) From the finding of this project, the researchers recommend that there is no tangible benefit in a switch from South Dakota A45 concrete mix to LC-HPC concrete for bridge decks in the State at the moment. More studies are required to demonstrate that there is beneficial effect due to the proposed switch because of the related cost implications.

2) When producing and placing LC-HPC for bridge decks, the contractor must be equipped to handle the mix and to have proper weather protection available during placement. Skewing the finishing bridges with respect to the actual deck layout is a preferred option. Vibrators may be mounted on the front of the finishing machine. Fogging needs to be modified to create a mist rather than fine water spray. All phases of the deck construction, i.e., placement, consolidating, finishing, and curing must be executed efficiently to meet the placement rates outlined in the specifications in order for LC-HPC to be placed successfully per the suggested specifications.

3) Concrete mix proportions should be designed for pumping or conveyor consistent with the placement method to be used at each site.

4) Care needs to be taken to design the vibrator mounting arrangement to prevent overlapping of vibration at the edge in future constructions.

5) Placement of concrete is to be avoided during severe windy conditions.

6) The data and the information from the existing bridges were limited. It is recommended that a more in-depth study, involving the same and other bridges, be performed to make more substantive claims regarding bridge deck cracking trends.

CHAPTER 2 INTRODUCTION

2.1 Problem Description

The need for the research arises from the SD-DOT involvement in TEA-21, "Innovative Bridge, Research and Construction Program (IBRC)". Research performed at the University of Kansas under the Pooled Fund project TPF-5(051), *Construction of Crack-Free Concrete Bridge Decks* proposes "best practices" dealing with materials, production and placement of concrete, and the relevant construction procedures to produce crack-free bridge decks. The proposed "best practices" include using a mix design that has less cement paste, densely graded aggregate, and increased air content. Incorporating these "best practices" is believed to reduce shrinkage cracking that occurs in bridge decks as the concrete cures. Extended moist curing and stringent control of evaporation during and immediately after placement are also believed to reduce cracking. Special provisions incorporating the additional requirements using the proposed "best practices" were developed by SD-DOT. The concrete mix using these special provisions is referred to as low cracking high performance concrete (LC-HPC). A copy of the specifications for LC-HPC developed by the Department is given in Appendix A. SD-DOT intended to evaluate LC-HPC in two bridge decks for potential implementation of LC-HPC in future projects. The evaluation includes constructability studies and the determination of the performance of LC-HPC bridge decks in terms of development of cracks over at least three years.

As a part of this program, it was required to select and schedule bridges that were constructed using the proposed "best practices" and carry out detailed crack surveys on the bridge decks. The crack surveys were to be performed using the protocol developed at the University of Kansas. A pair of identical bridges constructed in Pennington County and another pair of bridges constructed in Brule County were selected for the study. From each pair of bridge decks, one bridge deck was constructed using the SD-DOT A45 mix concrete and the other with the new LC-HPC mix concrete. Following is a brief description of the selected four bridges.

(a) Pennington County Bridges

Two bridges in Pennington County that are physically side by side were selected for the study. The two bridges are single-span composite steel girder bridges, with 208.5' x 40' roadway (structure numbers 52-435-289 and 52-435-288) as part of project IM90-2(134)59 PCN 4259, I-

90 over East North Street in Pennington County. One bridge is east bound and the other is west bound. The concrete mixes for these bridges used limestone aggregate.

(b) Brule County Bridges

Two three-span composite prestressed concrete girder bridges, both 173.6′ x 40′ roadway, (structure numbers 08-230-130 and 08-230-131) as part of IM-90-6(37)281 PCN 1245, I-90 over a county road were selected in Brule County.

Due to the proximity to the bridge construction sites from the respective university campus, the researchers from SDSMT studied the cracking behavior of the bridge decks located in Pennington County, and the researchers from SDSU studied the cracking behavior of bridges built in Brule County. The general approach and methodology of research however was uniform and consistent between the two groups of researchers because this is a collaborative research project with close cooperation and coordination between the two universities. The actual research work was shared between SDSMT and SDSU. Interim reports were submitted previously to document the findings from the research work.

The primary focus of the project was the study of constructability issues and the evaluation of the crack performance of the bridge decks constructed using LC-HPC concrete.

Constructibility issues were studied from the construction observations that were made during LC-HPC and the corresponding A45 concrete placement by representatives from South Dakota State University (SDSU), South Dakota School of Mines and Technology (SDSM&T), SDDOT engineers, and the contractors involved in the construction. These observations were used to evaluate the LC-HPC bridge deck construction process relative to A45 bridge deck placement.

The performance of the bridge decks was assessed by measuring and comparing crack densities in the bridge decks. Crack surveys (eight crack surveys in total for Pennington County bridges along with an additional repeat crack survey, and eight for Brule County bridges) were performed on the bridge decks using the protocol developed in the Pooled Fund study. These crack surveys correspond to the following four sets for each bridge deck:

(i) prior to the bridges being open to traffic (before grooving of the surface)

(ii) one year after being open to traffic

(iii) two years after being open to traffic and

(iv) three years after being open to traffic

Crack maps were developed from the crack surveys and the crack densities in m/m^2 were determined from the crack maps. These crack densities were used to compare the cracking performance of bridge decks.

Furthermore, a questionnaire was prepared and presented to all 50 American states, the District of Columbia, Puerto Rico, and all thirteen provinces and territories of Canada. The purpose of the questionnaire was to determine if any transportation agencies have attempted to reduce bridge deck cracking. There were nineteen participants (29%) and the results are summarized in this report.

Crack densities were also determined for six existing bridges without LC-HPC bridge decks. The existing bridges are of varying age, support type, and bridge deck types. Four bridges had steel girders and two had prestressed girders. For the six additional bridges, two bridge decks were placed using the old SDDOT mix design without flyash, two were placed with the SDDOT recent mix design with flyash, and two were placed with the SDDOT recent mix design with flyash and optimized aggregate.

SDDOT engineers measured and recorded fresh concrete properties in wet condition, and determined concrete compressive strength at different ages until 28 days. These test results were provided by SDDOT along with the temperature measurements for the LC-HPC bridge deck concrete in Pennington County.

Laboratory tests were also conducted at SDSMT to study the wet concrete properties and the drying shrinkage properties of A45 concrete and LC-HPC to compare the performance of the two concretes.

2.2 Research Objectives

The primary objectives of the project are as follows:

1. Evaluate the constructability of the bridge decks with proposed Low-Cracking High Performance Concrete (LC-HPC) material and construction specifications proposed by Pooled Fund project TPF-5(051) as implemented by SDDOT special provisions.

2. Evaluate the performance of the LC-HPC bridge decks with respect to crack densities relative to the currently used A45 bridge decks.

3. Evaluate the suitability of the draft Pooled Fund material and construction specifications for construction of crack-free bridge decks in South Dakota.

An additional objective was to determine the as-cast air voids, aggregate content and distribution, and an estimate of the density, absorption, and void content from a petrographic analysis and to compare these properties of LC-HPC concrete with those of A45 concrete.

Secondary objectives were to determine the suitability of the proposed LC-HPC mix designs, laboratory evaluation of properties of LC-HPC, properties of LC-HPC that was actually placed in bridge decks at sites, determination of concrete deck cracking performance of currently existing bridges in the State, and to collect and analyze the data from a transportation agency questionnaire.

2.3 Original Research Task List as Provided by SD-DOT

1) Meet with the technical panel to review project scope, discuss construction issues, and present tentative work plan.
2) Conduct a literature search and compare the results of bridge construction practices and specifications in SDDOT with other states as they relate to crack-free bridge deck construction.
3) Finalize a work plan incorporating previous SDDOT and other research results for monitoring construction and evaluating performance of the experimental bridges.
4) Conduct shrinkage tests using SDDOT standard mix designs with fly ash and mix designs with

reduced cement content and no fly ash, using South Dakota materials, and compare the results.

5) Document the concrete physical properties (seven, fourteen, and twenty-eight day compressive strengths, air content, slump, and temperature) of SDDOT-furnished mix designs, for the control bridges and the LC-HPC concrete used on this project in the final report.
6) Monitor construction of the control bridge decks and the LC-HPC bridge decks, including test slabs, and document the results. Submit an interim report, after construction of each of the four bridges, detailing constructability issues and suggestions of best practices for use on future bridges.
7) Conduct an extensive crack survey of each of the four bridges prior to its being opened to traffic and annually for three years after opening to traffic. The crack surveys will be conducted according to the protocol developed as part of Pooled Fund TPF-5(051) Construction of Crack-Free Concrete Bridge Decks.
8) Conduct crack surveys on a minimum of two of each type of South Dakota bridges constructed prior to this research using: old mix design without addition of fly ash; more recent mix design with fly ash but without optimized aggregate; and current mix design with optimized aggregate and fly ash.
9) Compare current SDDOT materials and construction specifications to the draft Pooled Fund material and construction specifications being developed for crack-free bridge decks with respect to constructability, South Dakota materials, and concrete plant capabilities and recommend best practices for future projects.
10) Prepare a final report and executive summary of the prior research, research methodology, findings, conclusions and recommendations.
11) Make an executive presentation to the SDDOT Research Review Board at the conclusion of the project.

A supplemental task was added to the project later. The task was to perform petrographic analysis to compare the as-cast air voids, aggregate content and distribution, and an estimate of the density, absorption, and void content of LC-HPC concrete with the corresponding values of A45 concrete obtained from the same study.

2.4 Structure of the Final Report

This final report details the findings from the study. Chapter 4 of the report presents results of a comprehensive literature search and a discussion on bridge construction practices and specifications in SDDOT and other states as they relate to crack-free bridge deck construction.

The results of the laboratory tests are presented in Chapter 6. The properties of field concrete as recorded by the Department are summarized in Chapter 7 along with temperature measurements.

Chapter 8 of this report provides details of the constructability studies for the test slabs, the A45 bridge decks and the LC-HPC bridge decks in Pennington and Brule counties.

The details of the crack surveys conducted on the two bridge decks are presented in Chapter 9. The details of crack surveys conducted in this project on six existing bridge decks are presented in Chapter 10.

The observations on the cores cut from the test slab, A45 bridge deck and LC-HPC bridge deck are outlined in Chapter 12. The findings from petrographic studies of the cores cut from deck concrete for the two Pennington County bridges and the two Brule County bridges are included in Appendix D. The conclusions drawn from the study are given in Chapter 13 along with the relevant discussion and recommendations stemming from this project.

The special provisions developed by SD-DOT for LC-HPC are included in Appendix A, while Appendix B includes a copy of the mix proportions. A copy of the Pooled Fund project crack survey protocol is given in Appendix C. The complete petrographic report is included in Appendix D in its entirety.

CHAPTER 3 TASK DESCRIPTIONS

Task 1. Meet with the technical panel to review project scope, discuss construction issues, and present tentative work plan.

The researchers attended a planning meeting held at the SDDOT main office in Pierre, SD on May 16, 2005 where the researchers from SDSMT and SDSU met with the technical panel for the project. A technical panel meeting for the project also took place between the researchers and the technical panel on July 13, 2005 in SDDOT main office in Pierre, SD. Several issues and tentative schedules for bridge deck construction were discussed at the meeting.

CHAPTER 4 LITERATURE SEARCH AND SUMMARY OF BRIDGE CONSTRUCTION PRATICES AND SPECIFICATIONS

Task 2. Conduct a literature search and compare the results of bridge construction practices and specifications in SDDOT with other states as they relate to crack-free bridge deck construction.

4.1 Background

As highlighted in Transport Research Circular E-C 107, one of "*the age-old axioms in the concrete industry is that concrete cracks*". Relatively low tensile strength of concrete, is one of the dominant causes of cracking. Volumetric instability and the deleterious chemical reactions are considered to be the source of much of the cracking. Cracks provide access of corrosive solutions to the embedded steel reinforcement within concrete which in turn will cause degradation of steel bars.

Development of numerous cracks in bridge decks is a serious problem throughout the United States. Many bridge decks that were constructed in the last several years in various states have shown different levels and patterns of cracking. Several studies were conducted in the recent years to address the problem of cracks in bridge decks.

A list of common types of cracks is provided in Fig. 4.1 (reproduced from TR Circular E-C 107) based on when they appear in concrete, i.e., before hardening or after hardening. The figure shows the complex interrelation of crack development in concrete. Table 4.1 (from E-C 107) shows a summary of the type and form of cracks along with the primary causes with an approximate timeline for the appearance of these cracks.

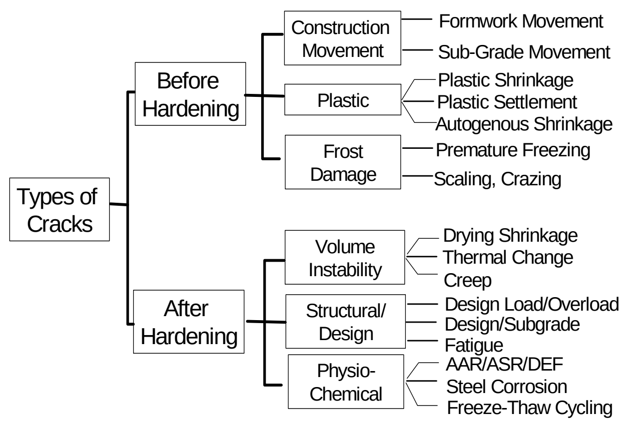


Fig. 4.1 Common Types of Cracking in Concrete Structures

(From TR Circular E-C 107)

Regardless of the type of bridge, the length and number of spans, and concrete mixture proportions, cracks invariably develop in all concrete deck slabs. Different crack types are classified in NCHRP Synthesis 333 as shown in Fig. 4.2. There can be several reasons for the development of cracks, but the transverse shrinkage cracks are believed to be one of the most dominant types of cracks in bridge decks due to the large surface area of the decks. Transverse shrinkage cracks can generally be observed at regular intervals of about 5 to 8 feet transverse to the longitudinal axis of a bridge (at a regular spacing parallel to the axis). Drying shrinkage effects are greatly influenced by concrete material properties, mixture proportions, material sources, cement content in concrete, end support restraints, type and extent of reinforcement, etc.

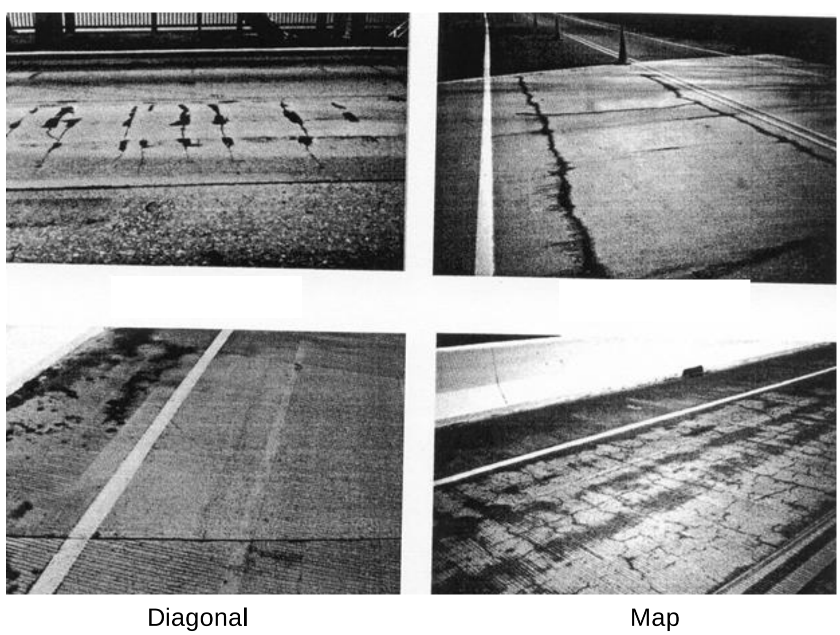


Fig. 4.2 Classification of Cracks (From: NCHRP Synthesis 333)

Transportation Research Circular E-C 107 provides several causes of crack formation in transportation structures, and their relation to the performance of concrete transportation structures. The circular also provides in-depth coverage of control of cracking and strategies for the prevention of cracking.

One of the few comprehensive resources that exclusively presents details of concrete bridge deck performance is NCHRP Synthesis 333. This report provides previous and current (at the time of its printing) practices used to improve the performance of bridge decks. The report mentions that the concern on cracking in bridge deck concrete increased in 1960s and the 1970s. Several strategies adopted in the last 40 years to minimize cracking include (i) increased clear concrete cover (ii) use of low slump, dense, low permeability concrete, and (iii) epoxy coated reinforcing bars. However, even after significant research and resources were devoted to solving the problem of bridge deck cracking, the cracking in bridge decks is still prevalent in old and recently constructed bridge decks, and the solution seems to be ever elusive.

Table 4.1 Classification of Cracks per Transportation Research Circular E-C107

Type of Cracking	**Form of Crack**	**Primary Cause**	**Time of Appearance**
Plastic settlement	over and aligned with reinforcement, subsidence under reinforcing bars	excessive bleeding, excessive vibrations	10 minutes to 3 hours
Plastic shrinkage	diagonal or random	rapid early drying	30 minutes to 6 hours
Thermal Contraction	Transverse	excess heat generation, excess temperature gradients	1 day to 2-3 weeks
Drying shrinkage	Transverse, pattern or map cracking	excessive mixture water, inefficient joints	few hours to months
Freezing and thawing	Parallel to the surface of concrete	lack of proper air-void system	more than a year
Corrosion of reinforcement	Over reinforcement	lack of cover, excess chloride	more than two years
Alkali-silica reaction	pattern and longitudinal cracks parallel to the least restrained side	reactive aggregate plus alkali	more than 5 years
Sulfate Attack	Pattern	Internal or external sulfates promoting the formation of ettringite	1 to 5 years

All cracks in concrete may not be deleterious. ACI Committee report 224 recommends that a minimum crack width of 0.007 inch for conditions such as those prevalent in South Dakota bridge decks can cause deterioration related to durability. Table 4.2 shows a summary of the classification of cracks based on crack widths as suggested in ACI 224 report.

Table 4.2 Allowable Crack Widths per ACI 224 Report

Exposure Condition	Maximum Allowable Crack Width
Dry Air	0.41 mm (0.016 in.)
Humidity, Moist Air, Soil	0.30 mm (0.012 in.)
Deicing Chemicals	0.18 mm (0.007 in.)
Sea Water	0.15 mm (0.006 in.)
Water Retaining Structures	0.10 mm (0.004 in.)

4.2 SD-DOT Efforts to Minimize Cracks in Bridge Decks

Of the several recent efforts by SD-DOT to minimize cracking in bridge deck concrete, a recent project on optimized aggregate gradation for structural concrete (Project # SD2002-02) completed by Dr. V. Ramakrishnan of SDSMT addressed the issue most directly. Use of concrete mixes based on high cement content and a gap graded (coarse and fine) aggregate was thought to be one of the causes for early transverse cracking seen in new bridge decks in the State. Use of a more well-graded aggregate permitted lower cement paste content which resulted in reduced shrinkage cracking. These findings were implemented in field studies. The crack surveys of the three new bridge decks constructed with optimized aggregate gradation concrete revealed that total number and total areas of cracks of the new bridges were significantly less than those for the older South Dakota bridge decks with standard A45 concrete that did not use optimized aggregate gradation concrete. The reduction was generally as much as 5 to 10 fold. The results of the crack surveys and the findings are included in the final report of the extension to this original project (SD2002-02E by Ramakrishnan and Patnaik, 2006).

The total number of cracks and crack areas of several bridge decks are also reported in the final report of SD2002-02 for thirteen bridges in the State. Six of the bridges surveyed were in East river and seven bridges surveyed were in West river areas of the State. Some of the details of these bridges are given in Tables 4.3 and 4.4.

The protocol developed in the Pooled Fund study requires that the crack densities in the current project be developed for the top surface of bridge decks only. The cracks included in the crack maps needed to be those cracks that were visible from waist height. The crack densities in the earlier project (SD2002-02) were expressed in terms of area of cracks (square foot) per 1000 square foot of bridge deck area. This was possible because the crack widths were recorded during the crack surveys in that project. However, the pooled study protocol specifically precluded measurement of crack widths and cracks not visible from waist height. In the context of the current study, the average crack density of these thirteen bridges that were surveyed in the State previously was calculated to be about 0.5 m/m^2 with average crack density of about 0.3 m/m^2 for East River bridges and 0.6 m/m^2 for West River bridges. These densities were based on the cracks that were recorded from the top surface of the bridge decks. The average crack width for the study is assumed to be 0.007 inch. These bridges were mostly constructed in 1980s and 1990s. Old bridge decks such as those surveyed in the State in the earlier project are known to have smaller crack densities than those constructed more recently. Density calculations in those crack surveys also included smaller cracks not visible from waist height, but the average values are expected to comparable.

Table 4.3 Details of the Crack Surveys for South Dakota Bridges

(Previously Reported under Project # SD2002-02)

S.No	Structure Number	No. of Cracks In the Curbs /Barriers	No. of Cracks In the Top Surface	No. of Cracks In the Bottom Surface	Total No. of Cracks	Year of Construction
	East River					
1	09-126-149	295	239	356	890	September-98
2	18-141-093	328	345	552	1225	February-95
3	08-080-112	617	269	683	1569	December-91
4	50-020-141	96	200	213	509	June-86
5	50-020-142	192	150	204	546	June-86
6	18-180-100	411	81	302	794	May-86
	West River					
1	52-415-285	215	146	652	1013	July-99
2	52-415-286	749	221	633	1603	July-99
3	47-215-363	127	24	249	400	January-97
4	24-248-119	166	14	182	362	December-96
5	41-095-059	331	25	439	795	March-95
6	10-103-367	261	87	300	648	December-90
7	10-114-411	416	68	290	774	October-89

Table reproduced from the Final Report of Project SD2002-02 (Courtesy: Dr. Rama)
Total number of cracks = 5533 (East River), 5595 (West River)

Table 4.4 Crack Densities for South Dakota Bridges

(Previously Reported under Project # SD2002-02)

S.No	Structure Number	Area of Cracks In the Top Surface (ft^2)	Area of Cracks In the Bottom Surface (ft^2)	Total Area of Cracks (ft^2)	Total Area of Bridge Deck (ft^2)
	East River				
1	09-126-149	0.050	0.100	0.150	9648
2	18-141-093	0.070	0.090	0.160	12220
3	08-080-112	0.080	0.180	0.260	10732
4	50-020-141	0.030	0.100	0.130	6048
5	50-020-142	0.040	0.090	0.130	6048
6	18-180-100	0.030	0.060	0.090	12296
	West River				
1	52-415-285	0.160	0.110	0.270	14920
2	52-415-286	0.360	0.100	0.460	14920
3	47-215-363	0.002	0.050	0.052	13388
4	24-248-119	0.001	0.040	0.041	15048
5	41-095-059	0.009	0.060	0.069	13390
6	10-103-367	0.080	0.040	0.120	19844
7	10-114-411	0.140	0.040	0.180	13543

Table reproduced from the Final Report of Project SD2002-02 (Courtesy: Dr. Rama)

4.3 Crack-Free Bridge Deck Studies at the University of Kansas (On-Going Efforts)

A major effort to minimize cracks in bridge decks was initiated in early 1990s by Dr. David Darwin and his team at the University of Kansas (KU). The project is ongoing under "Transportation Pooled Fund Study for the Construction of Crack-Free Bridge Decks". Complete details of the study and the background information on the ongoing pooled fund study project are given at the following website:

http://www2.ku.edu/~iri/projects/concrete/phase2.html

Phase I of the project is reported to be recently completed, and Phase II is currently ongoing. The current study is part of the pooled fund effort at KU. Several comprehensive reports are linked from the KU website. Some of the reports are listed below:

SM Report 78 “Cracking and Chloride Contents in Reinforced Concrete Bridge Decks"

SM Report 89 "Evaluating Free Shrinkage of Concrete for Control of Cracking in Bridge Decks

SM Report 92 “Development and Construction of Low-Cracking High-Performance Concrete (LC-HPC) Bridge Decks: Free Shrinkage, Mixture Optimization, and Concrete Production”

SM Report 94 "Development and Construction of Low-Cracking High-Performance Concrete (LC-HPC) Bridge Decks: Construction Methods, Specifications, and Resistance to Chloride Ion Penetration"

SM Report 97 "Lightweight Aggregates as an Internal Curing Agent for Low-Cracking High-Performance Concrete"

SM Report 98 "Effect of Materials and Curing Period on Shrinkage of Concrete"

The concrete mix details, material and construction specifications along with crack survey protocols used in this study were developed in the KU project. These specifications deal with materials, production and placement of concrete, and the relevant construction procedures to produce crack-free bridge decks. The project recommended using a mix design that has less cement paste, densely graded aggregate, and increased air content. Incorporating these “best practices” is believed to reduce shrinkage cracking that occurs in bridge decks as the concrete cures. Extended moist curing and stringent control of evaporation during and immediately after placement are also believed to reduce cracking. The special provisions developed by SD-DOT by adopting these requirements are based on the KU project. The concrete mix using these special provisions is referred to as low cracking high performance concrete (LC-HPC). Fig. 4.3 shows the crack densities of several bridge decks surveyed in the project. The figure is reproduced from the referred KU website. The figure shows that crack density increases with time for monolithic and control bridge decks. LC-HPC bridge decks are seen to produce greatly diminished crack densities. The data is available approximately for the last three years.

Measurements were taken on bridges in Kansas to study the penetration of chlorides to the level of the reinforcing steel. Their findings showed that dense, high quality concrete can significantly slow chloride penetration. Formation of cracks however reduces the advantage of dense low permeability high quality concrete. Therefore, the focus of the study was to reduce the crack densities in concrete bridge decks mainly through stringent control of the material properties of the concrete used in the bridge decks.

The study is expanding to include demonstration projects in several states. It is expected to provide more details in the near future. Results from this study are likely to be incorporated into the Kansas pooled fund study.

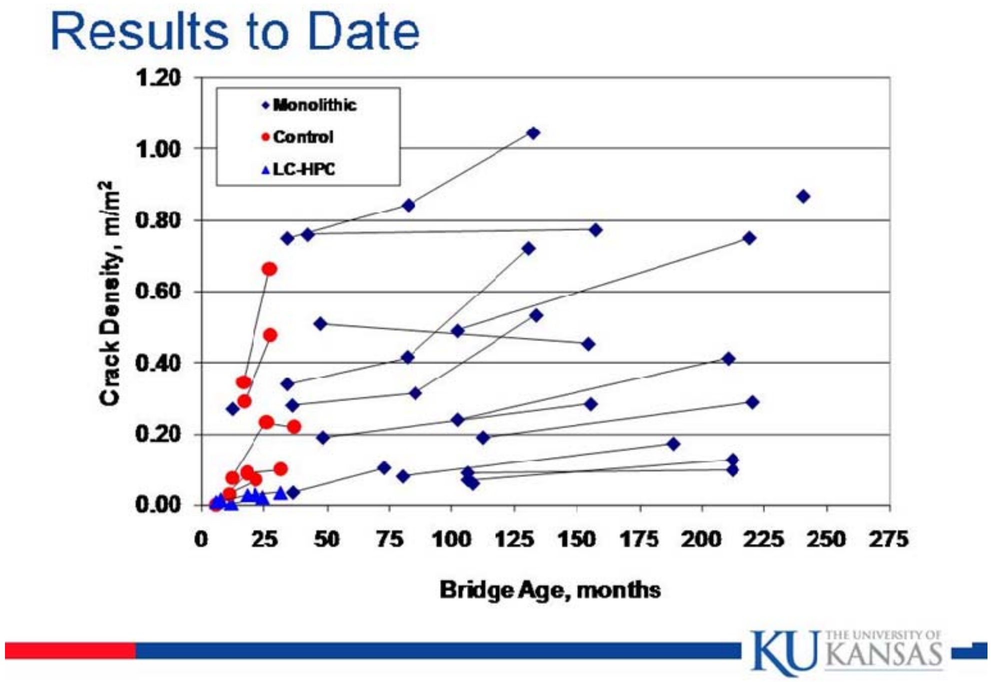


Fig. 4.3 Crack Densities of Bridge Decks Studied at KU

(From: http://www2.ku.edu/~iri/projects/concrete/phase2.html)

4.4 NYDOT Effort

A summary of knowledge gained by the New York State Department of Transportation (NYSDOT) Bridge Deck Task Force (BDTF) is presented in recent reports [Alampalli and Owens, 2000, Curtis and White, 2007]. The authors do not recommend any 'silver bullet' solution, but they discuss several recommendations to reduce the occurrence and magnitude of bridge deck cracking. They also discuss the factors that influence concrete deck cracking, and methods of treating existing cracks in bridge decks for single span structures. The report by Subramaniam and Agarwal (2009) studied in-situ properties of concrete of existing bridges, monitored strain and temperature data from newly constructed bridges, and evaluated local NY materials to study their influence on cracking.

4.5 Other Reference Material

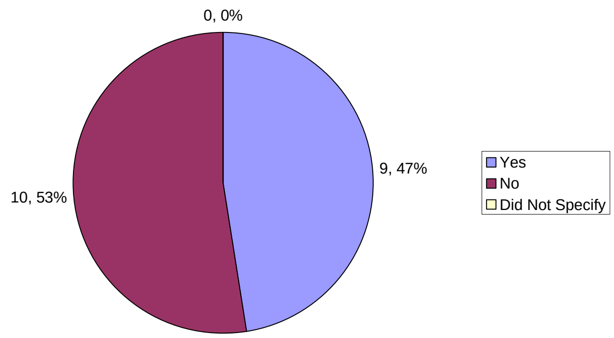
There is a significant amount of literature published by several other researchers and agencies. However, all these publications are not being discussed in this report for brevity. The retrieval details of the relevant and useful publications are given in the bibliography at the end of this section (see Section 4.7).

4.6 Bridge Construction Practices and Specifications in Other States

Through specific research efforts to understand the cracking behavior of concrete bridge decks, several states developed their own construction practices and material specifications in order to reduce cracking in concrete decks. These practices and specifications are summarized by the SDSU research team. This summary was developed by sending a questionnaire to all the state DOTs in the U.S. and also to DOTs in Canada. The response received from the questionnaire was summarized, compiled and presented by the SDSU team.

The questionnaire was prepared to determine the severity of bridge deck cracking and to determine the state-of-practice regarding attempts to reduce bridge deck cracking. The questionnaire was kept short to promote participation in the survey. The questionnaire was sent to a total of 65 transportation agencies: all 50 American states, the District of Columbia, Puerto Rico, and all thirteen Canadian provinces and territories. Only nineteen (29%) of the agencies responded. The results of the questionnaire are summarized in this section.

The first objective was to determine which states are using or have tried to develop techniques for placing low-cracking concrete bridge decks. Figure 4.4 shows the agencies that have developed specifications for reduced cracking concrete. Approximately half, 9 (47%), of the 19 respondents have developed specifications. The other 10 (53%) have not developed specifications. Looking at the map in Figure 4.4, there does not appear to be any geographical or climatic trends.



Yes	No	Did Not Specify
Alberta	Arkansas	
Iowa	Connecticut	
Kansas	Idaho	
Maryland	Louisiana	
Missouri	Maine	
New Jersey	North Dakota	
Ontario	Nova Scotia	
Oregon	Oklahoma	
Wisconsin	Puerto Rico	
	Virginia	

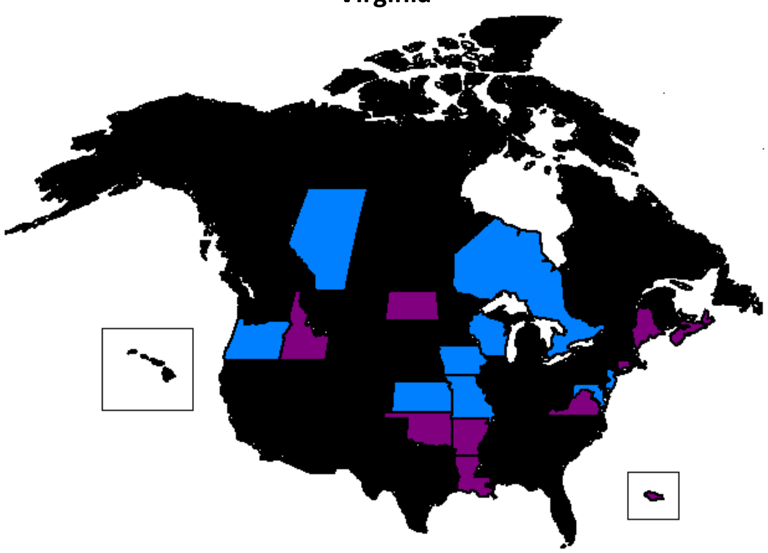
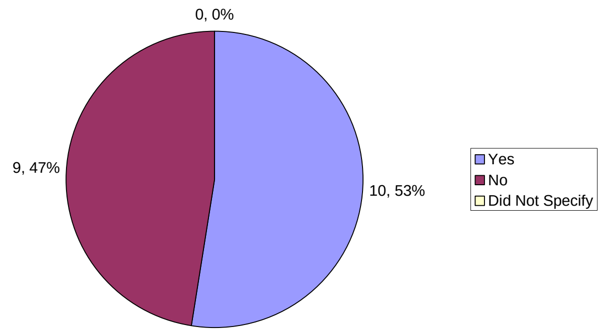


Fig 4.4 Developments of Specifications for Reducing Cracking in Concrete Bridge Decks (out of 19)

Figure 4.5 shows the agencies that have performed research attempting to reduce bridge deck cracking. Of the 19 respondents, 10 (53%) responded that their agencies have performed some form of research in this area and the other 9 (47%) have not performed any research. Again, there does not appear to be and geographic or climatic trend. Of the 10 agencies that reported performing research to reduce cracking, only 4 (40%) reported that their agency published a formal report, shown in Figure 4.6. These states are Arkansas, Virginia, New Jersey, and Maine. The other 6 (60%) did not prepare a report of their findings.



Yes	No	Did Not Specify
Arkansas	Alberta	
Idaho	Connecticut	
Kansas	Iowa	
Maine	Louisiana	
Maryland	North Dakota	
Missouri	Nova Scotia	
New Jersey	Ontario	
Oklahoma	Oregon	
Virginia	Puerto Rico	
Wisconsin		

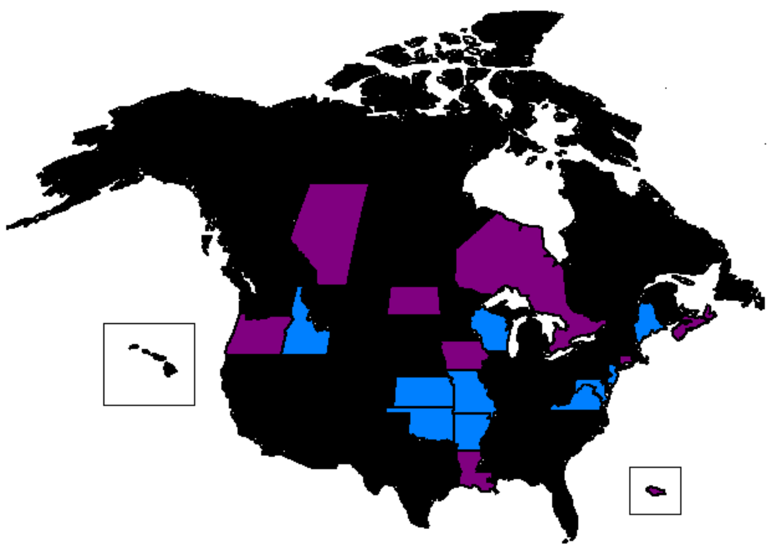
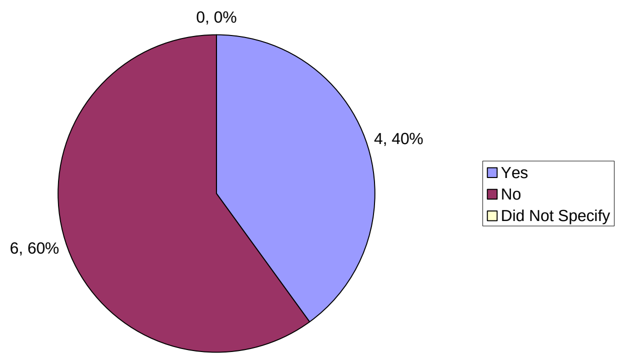


Fig. 4.5 Research Performed into Reducing the Cracking of Concrete Bridge Decks (out of 19)



Yes	No	Did Not Specify
Arkansas	**Idaho**	
Maine	**Kansas**	
New Jersey	**Maryland**	
Virginia	**Missouri**	
	Oklahoma	
	Wisconsin	

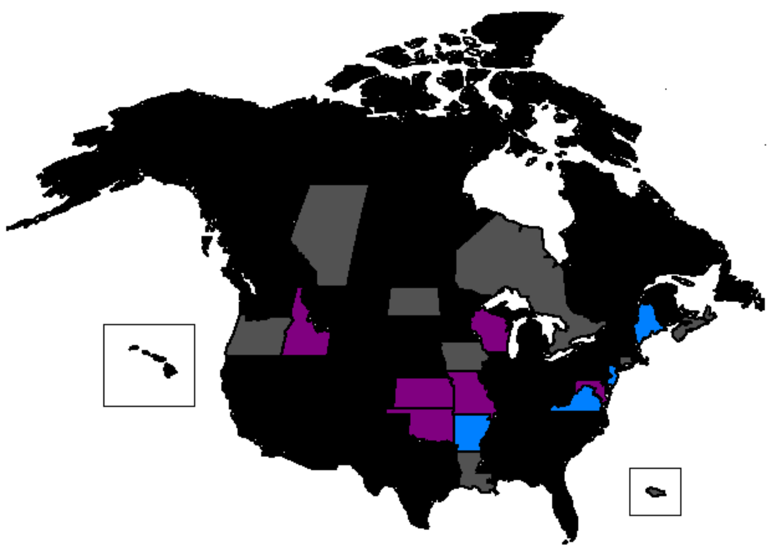
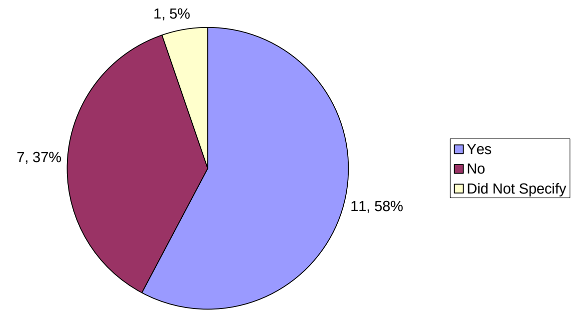


Fig. 4.6 Performed Research and Published a Report of the Results (out of 10)

The second objective was to try and quantify how problematic bridge deck cracking is. The agencies that reported bridge deck cracking as a problem are shown in Figure 4.7. Of the 19 respondents, 11 (58%) reported that bridge decks cracking is a problem for their agencies. Oregon noted that bridge decks experience mostly shrinkage cracking. Missouri and Wisconsin noticed that cracking has become an increased problem in the last 10 to 15 years. Only 7 (37%) reported that cracking is not a problem and one (5%) agency, Idaho, did not respond to this question. Idaho, however, does have long-term durability issues with its bridge decks caused by transverse cracking. An explanation for the responses that cracking is not a problem is that cracking is unavoidable and therefore two agencies depend on waterproofing membranes (Ontario and Maine). Ontario has noticed that cracking is more evident in high-performance concrete (HPC). North Dakota has cracking, but was unsure of the definition of 'serious' cracking. No significant geographic or climatic trends are apparent from the map attached to Figure 4.7.



Yes	No	Did Not Specify
Alberta	**North Dakota**	**Nova Scotia**
Arkansas	**Iowa**	
Idaho	**Ontario**	
Kansas	**Louisiana**	
Missouri	**Maine**	
New Jersey	**Connecticut**	
Oklahoma	**Maryland**	
Oregon		
Puerto Rico		
Virginia		
Wisconsin		

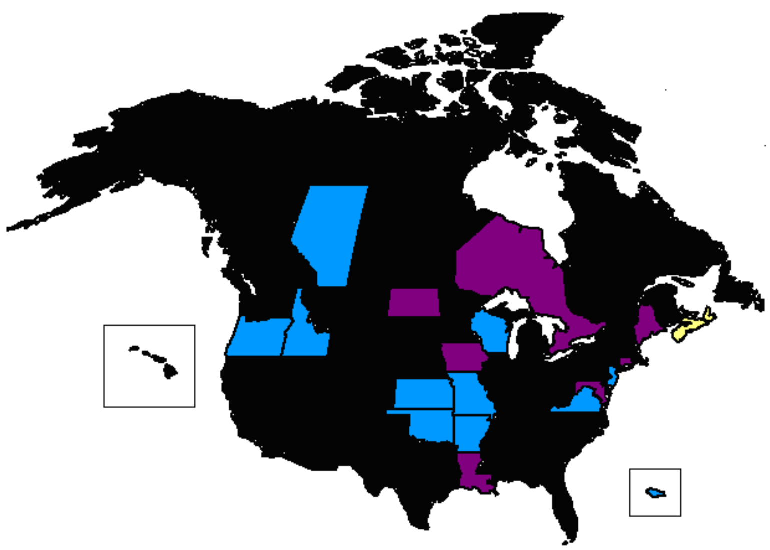
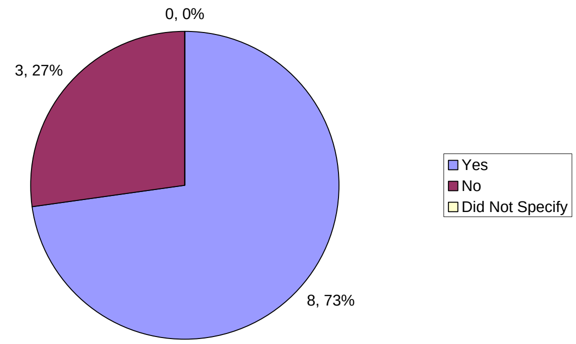


Fig. 4.7 Reported Bridge Deck Cracking as a Problem (out of 19)

Figure 4.8 shows the agencies that reported bridge deck cracking as a problem and performed research attempting to reduce cracking. Of the 11 agencies that reported cracking as a problem, 8 (73%) have performed research while only 3 (27%) have not (Alberta, Oregon, and Puerto Rico). Similarly,

Figure 4.9 shows agencies that reported cracking was not a problem, but had performed research attempting to reduce cracking anyway. Of the 7 agencies that reported cracking was not a problem, 2 (29%) had also performed research (Maryland and Maine), while the other 5 (71%) did not perform research. Again, Maine considers cracking to be unavoidable and reported using waterproof membranes on its bridge decks.



Yes	No	Did Not Specify
Arkansas	**Alberta**	
Idaho	**Oregon**	
Kansas	**Puerto Rico**	
Missouri		
New Jersey		
Oklahoma		
Virginia		
Wisconsin		

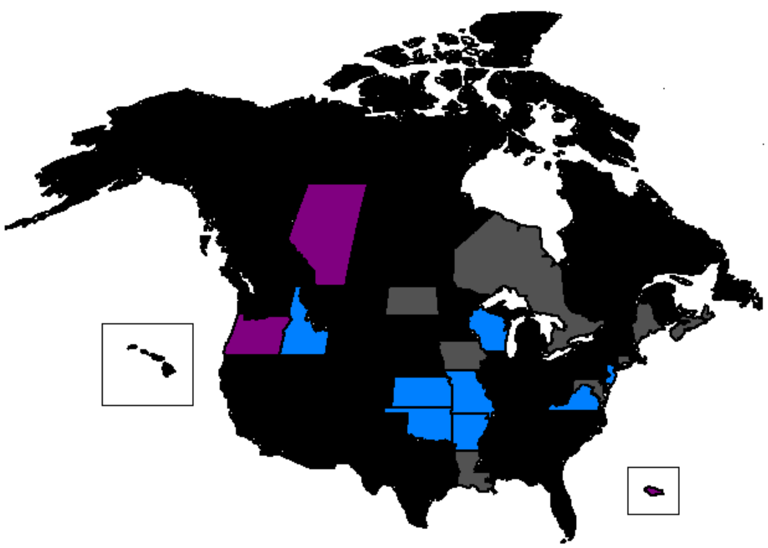
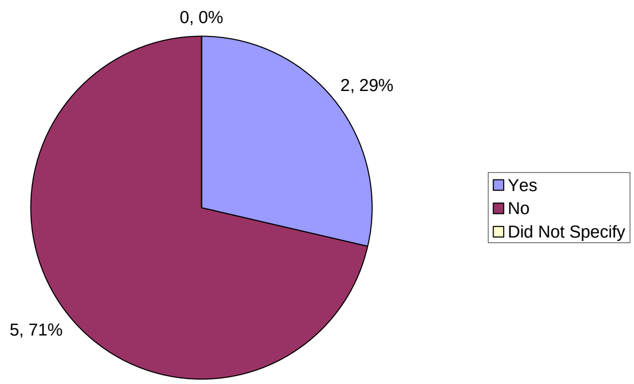


Fig. 4.8 Reported Bridge Deck Cracking as a Problem and have Performed Research into Crack Reduction (out of 11)



Yes	No	Did Not Specify
Maine	North Dakota	
Maryland	Ontario	
	Iowa	
	Louisiana	
	Connecticut	

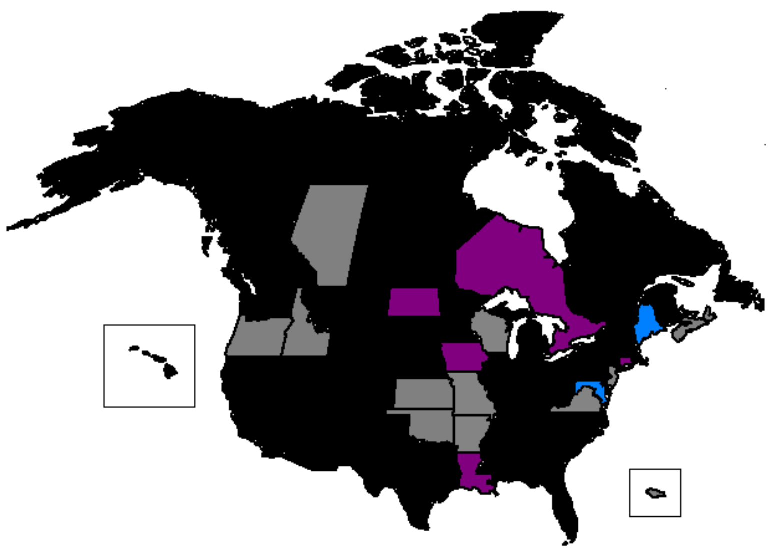
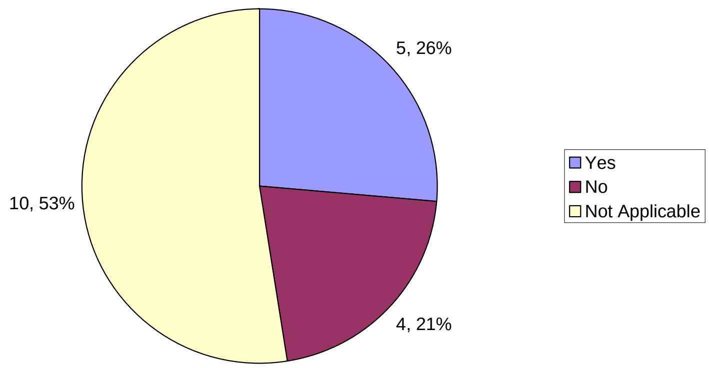


Fig. 4.9 Reported Bridge Deck Cracking was not a Problem and have Performed Research into Crack Reduction (out of 7)

The third objective of this questionnaire was to determine which agencies achieved success in reducing bridge deck cracking. Figure 4.10 shows which agencies reported success with their implemented measures. Of the 19 respondents, 5 (26%) reported that they had success with reducing bridge deck cracking, 4 (21%) did not have success, and 10 (53%) responded that this was not applicable to their agency. The agencies that reported success reducing cracking are Ontario, Alberta, Kansas, Iowa, and Maryland. Each agency implemented different techniques to help reduce cracking. Alberta incorporated the use of silica fume and fly ash, as well as extending the moist cure to 14 days. Iowa had success by implementing an evaporation rate control procedure, the use of mineral admixtures, set-retarder and water reducer, and also required an immediate wet cure. Kansas has had success by controlling environmental conditions, using different placing sequences, and requiring a wet cure. Maryland reported success by using HPC and fiber reinforcement. Ontario has had success using a four-day moist cure and then a waterproof membrane. Maine reported that their efforts have not been successful and therefore use waterproof membranes to reduce the concern of cracking. Wisconsin reported that they had had some success, but not enough. Missouri responded that this was not applicable because they are not sure of the effects that curing changes have made yet.



Yes	No	Not Applicable
Alberta	Maine	Arkansas
Iowa	New Jersey	Connecticut
Kansas	Puerto Rico	Idaho
Maryland	Wisconsin	Louisiana
Ontario		Missouri
		North Dakota
		Nova Scotia
		Oklahoma
		Oregon
		Virginia

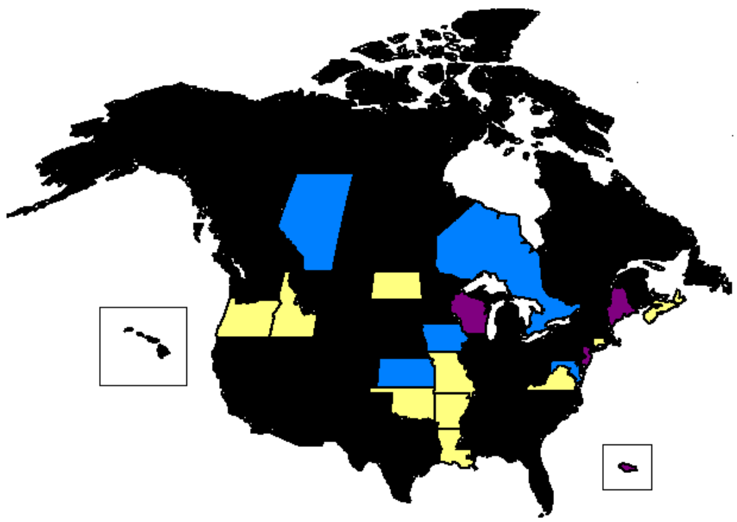
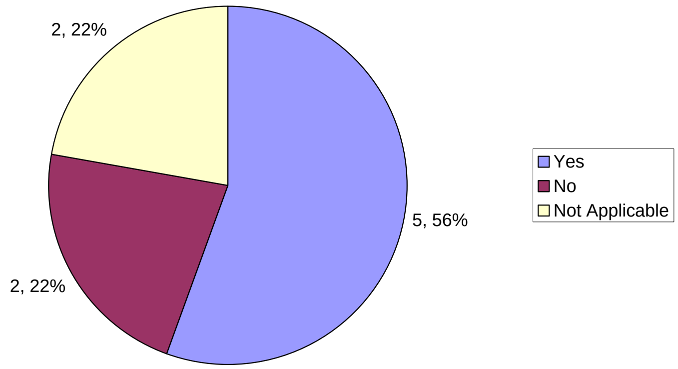


Fig. 4.10 Reported Success with Implemented Measures in Reducing Bridge Deck Cracking (out of 19)

Figure 4.11 shows which agencies have developed specifications to reduce bridge deck cracking and reported success with their implemented measures. Of the 9 agencies that developed specifications to reduce cracking, 5 (56%) reported they had success with their implemented measures, 2 (22%) did not have success, and 2 (22%) reported that this was not applicable. The agencies that reported success with their implemented measures from Figure 4.7 above are the same agencies that reported success in Figure 4.11 (Ontario, Alberta, Kansas, Iowa, and Maryland). The two agencies that reported that their specifications had not been successful in reducing cracking are New Jersey and Wisconsin. Similar to Figure 4.11, Figure 4.12 shows agencies that performed research attempting to reduce bridge deck cracking and reported success with their implemented measures. Of the 10 agencies that performed research, only 2 (20%) reported success with their implanted measures (Kansas and Maryland), 3 (30%) reported that they had not had success (Wisconsin, New Jersey, and Maine), and 5 (50%) reported that this was not applicable.



Yes	No	Not Applicable
Alberta	New Jersey	Missouri
Iowa	Wisconsin	Oregon
Kansas		
Maryland		
Ontario		

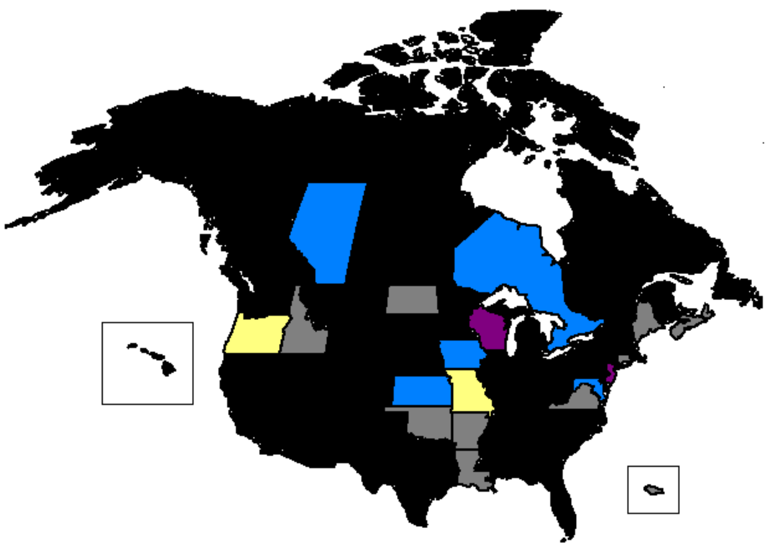
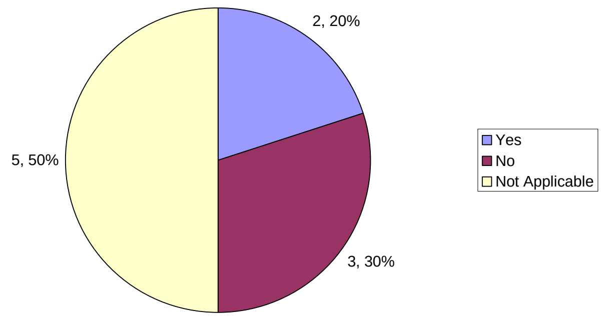


Fig. 4.11 Developed Specifications for Reduced Bridge Deck Cracking And Reported Success with Implemented Measures (out of 9)



Yes	No	Not Applicable
Kansas	Maine	Arkansas
Maryland	New Jersey	Idaho
	Wisconsin	Missouri
		Oklahoma
		Virginia

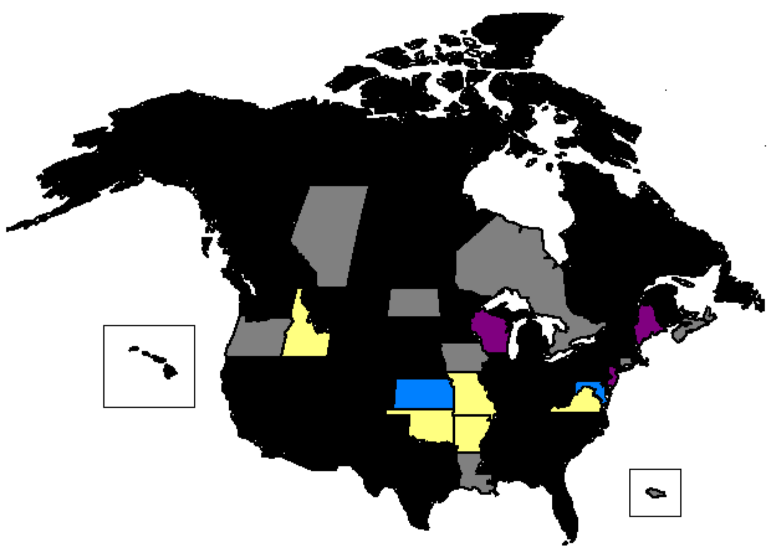
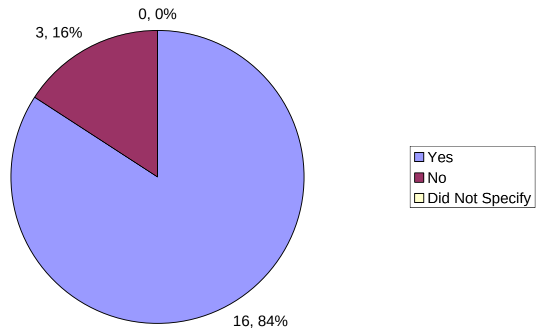


Fig. 4.12 Performed Research into Reducing Bridge Deck Cracking and Reported Success with Implemented Measures (out of 10)

The fourth objective of this questionnaire was to try and determine what durability issues agencies have with bridge decks. The agencies were asked what the expected service life of their concrete bridge decks was. The average lifespan is 49 years with a range from 20 to 100 years. All three of the responding Canadian provinces reported an expected lifespan of 75 years, while the Midwest states reported a range of 30 to 50 years. Figure 4.13 shows which agencies reported that de-icing chemicals or salts are used on their bridge deck surfaces. Of the 19 respondents, 16 (84%) reported using de-icing chemicals, and only 3 (16%) reported that de-icing chemicals are not used. The three agencies that reported de-icing chemicals are not used are Louisiana, Puerto Rico, and Ontario. Ontario does use de-icing chemicals, but it says that since its bridge decks have waterproof membranes, there is no possibility of chlorine contacting the reinforcement. The map attached to Figure 4.13 shows that mostly states in northern climates or mountainous terrain use de-icing chemicals. Arkansas reported that de-icing chemicals are used mostly in the northern regions of the state. Maine uses mostly a salt-brine solution. Oregon's use of de-icing chemicals is limited to the parts of the state that experience significant snow fall.



Yes	Yes (cont)	No
Alberta	Missouri	Louisiana
Arkansas	New Jersey	Ontario
Connecticut	North Dakota	Puerto Rico
Idaho	Nova Scotia	
Iowa	Oklahoma	
Kansas	Oregon	
Maine	Virginia	
Maryland	Wisconsin	

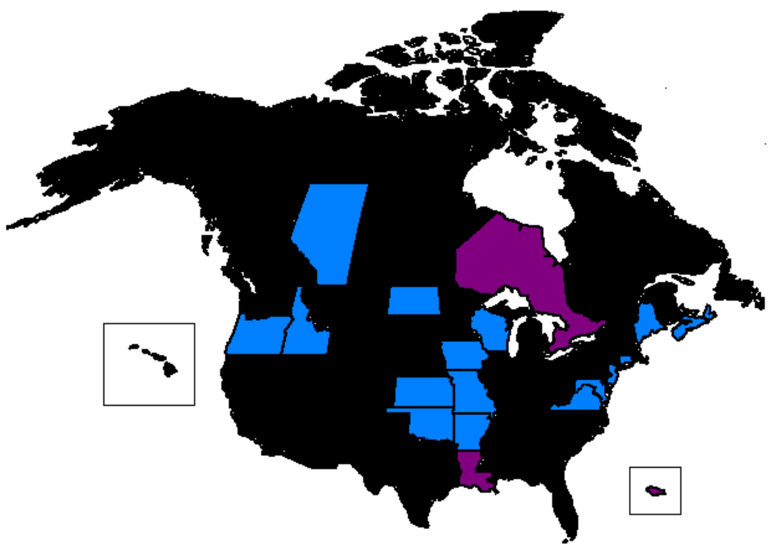
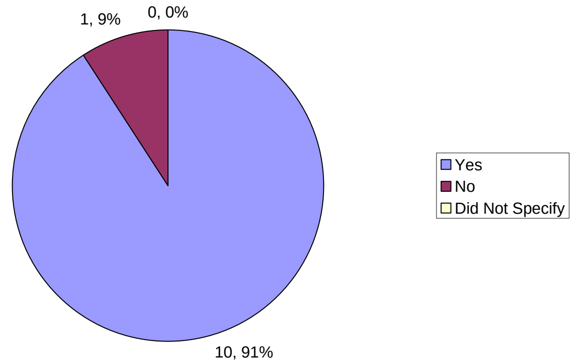


Fig. 4.13 Reported Using De-icing Chemicals on Bridge Deck Surfaces (out of 19)

Figure 4.14 shows the states that reported bridge deck cracking as a problem and use de-icing chemicals. Of the 11 states that reported cracking as a problem, 10 (91%) also use de-icing chemicals and only 1 (9%) does not use de-icing chemicals (Puerto Rico). Similar to Figure 4.14, Figure 4.15 shows the agencies that reported cracking was not a problem and use de-icing chemicals. Of the 7 agencies that reported cracking was not a problem, 5 (71%) use de-icing chemicals and 2 (29%) do not use de-icing chemicals. The two agencies that do not consider cracking a problem and do not use de-icing chemicals are Louisiana and Ontario. Again, Ontario believes that the waterproof membrane prevents the salts from penetrating the deck and reacting with the reinforcement. There are then five agencies that do not consider bridge deck cracking to be a problem, yet do use de-icing chemicals: North Dakota, Iowa, Maine, Connecticut, and Maryland. North Dakota is not sure of the definition of serious and Maine use waterproof membranes.



Yes	No	Did Not Specify
Alberta	Puerto Rico	
Arkansas		
Idaho		
Kansas		
Missouri		
New Jersey		
Oklahoma		
Oregon		
Virginia		
Wisconsin		

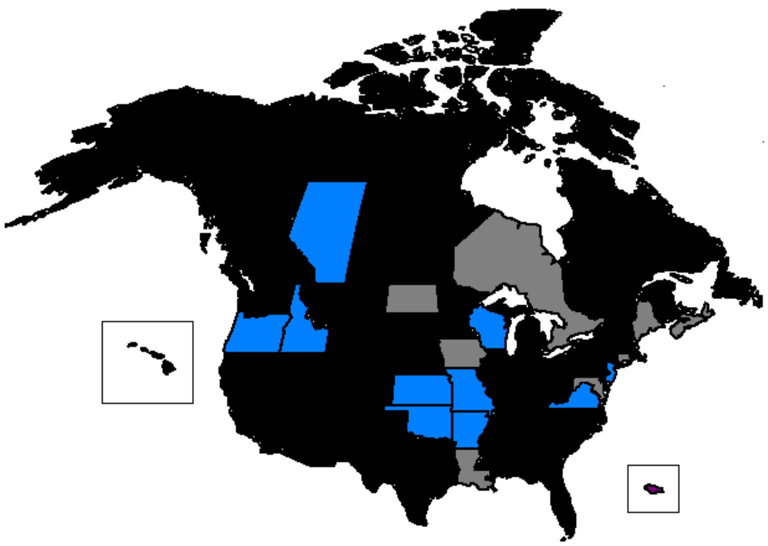
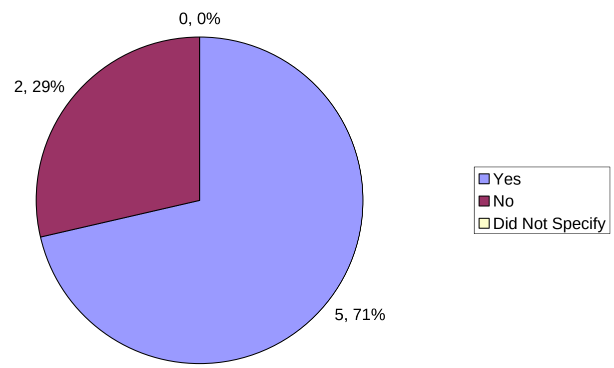


Fig. 4.14 Reported Bridge Deck Cracking as a Problem and Use De-icing Chemicals (out of 11)



Yes	No	Did Not Specify
Connecticut	Louisiana	
Iowa	Ontario	
Maine		
Maryland		
North Dakota		

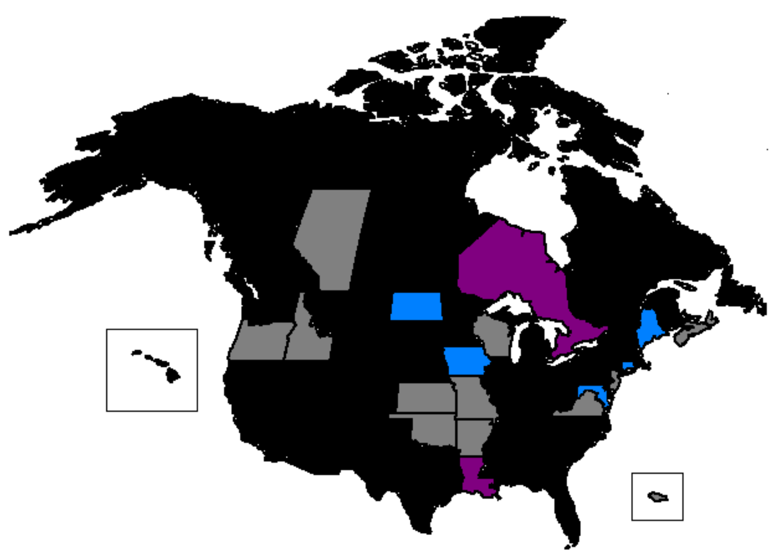
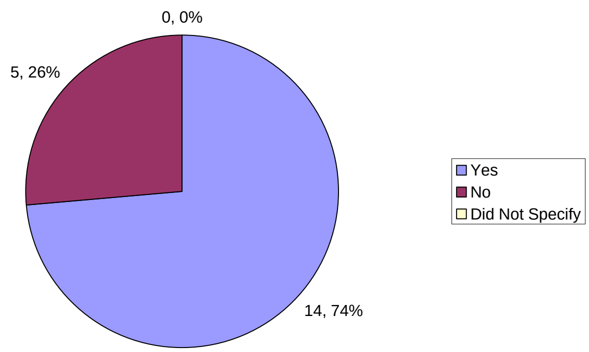


Fig. 4.15 Reported Bridge Deck Cracking was not a Problem and Use De-icing Chemicals (out of 7)

The fifth objective of the questionnaire was to determine how agencies are monitoring cracking and maintaining their bridge decks. Figure 4.16 shows which agencies perform inspections to determine the extent of cracking on the bridge deck surface. Of the 19 respondents, 14 (74%) responded that they do perform deck inspections and the other 5 (26%) do not. Of the agencies that reported not conducting inspections, Oklahoma performs routine NBIS inspections which allows for the identification of bridges with cracking problems, and Missouri only inspects the decks if it is a problem and cracking is not a criteria for accepting decks. Many agencies will inspect the decks after the construction. Alberta inspects the deck after curing and again prior to the warranty expiration. Connecticut conducts semi-final inspection after construction. New Jersey allows looks for cracks as a part of the construction inspection and also inspects for cracks per NBIS requirements. Oregon performs inspections every two years in accordance with FHWA requirements. Kansas as performed many intensive deck inspections in the last 15 years as part of multiple research projects, and some decks have been inspected as many as two to three times.



Yes	Yes (cont.)	No
Alberta	New Jersey	Louisiana
Arkansas	Nova Scotia	Maine
Connecticut	Ontario	Missouri
Idaho	Oregon	North Dakota
Iowa	Puerto Rico	Oklahoma
Kansas	Virginia	
Maryland	Wisconsin	

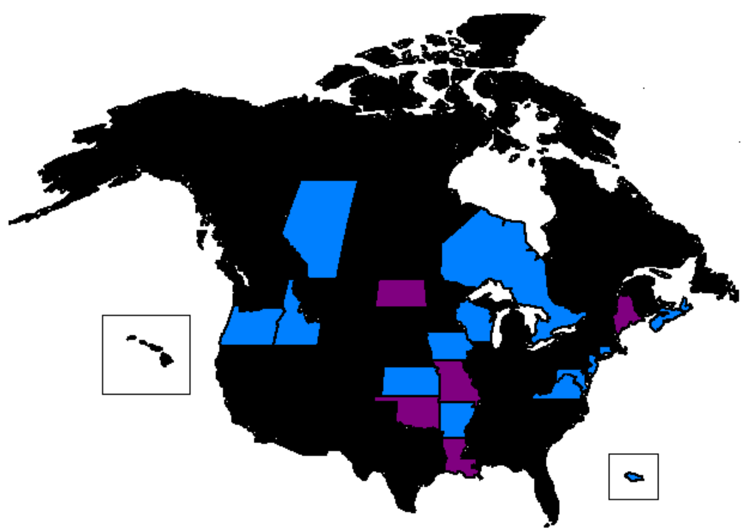
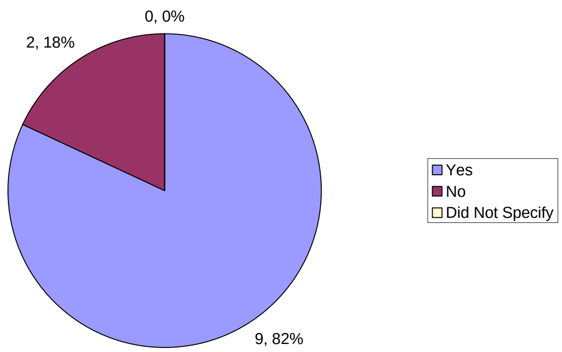


Fig. 4.16 Perform Bridge Deck Inspections to Determine the Extent of Cracking (out of 19)

Figure 4.17 shows which agencies reported cracking as a problem and perform deck inspections. Of the 11 respondents, nine (82%) reported that they perform inspections and only two (18%) responded that they do not (Oklahoma and Missouri). Figure 4.17 shows how many agencies repair cracking on their bridge decks. Of the 19 respondents, 13 (68%) reported that cracks are repaired and six (32%) reported that they do not. Oklahoma seals cracks on new decks and does flood coats on older ones. Alberta uses either gravity flow epoxy or epoxy injection. Idaho reports that due to budget limitations, repairs can be inconsistent. Several agencies look at deck cracking on a case by case basis or only repair major cracks (Wisconsin, Virginia, and Oregon). Kansas reports that cracks are repaired mostly by using rigid concrete overlays. Missouri uses either bituminous or penetrating sealers. New Jersey reported using cracks sealers to repair its decks. Some of the agencies that reported not repairing cracks will occasionally repair more serious cracks. Iowa reported that in some instances penetrating sealers have been used. Louisiana only repairs cracks that are not induced by recurring loads. Maine only repairs major cracks in the concrete wearing surface. Ontario believes that since its decks are waterproofed, crack repair is not necessary. Figure 4.19 shows which agencies responded that cracks are repaired. Of the 13 respondents, 11 (84%) also consider bridge deck cracking to be a problem. Only one (8%) does not consider cracking to be a problem (Maryland) and one (8%) did not respond. Figure 4.20 shows the agencies that reported that they perform deck inspections and perform crack repairs. Of the 14 agencies, 11 (79%) also repair the cracks on the deck surface and only three (21%) do not (Ontario, Connecticut, and Iowa).



Yes	No	Did Not Specify
Alberta	**Missouri**	
Arkansas	**Oklahoma**	
Idaho		
Kansas		
New Jersey		
Oregon		
Puerto Rico		
Virginia		
Wisconsin		

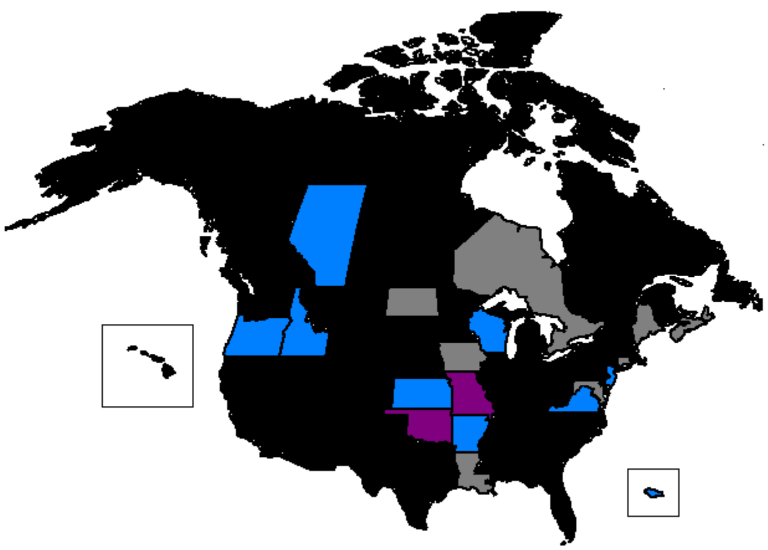
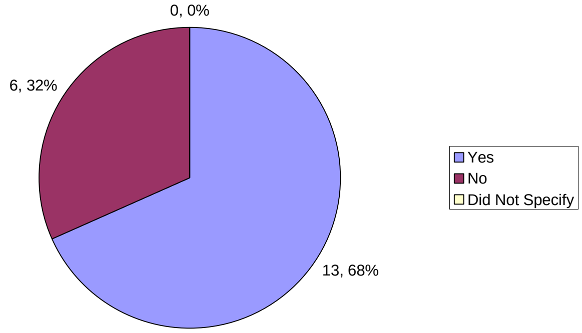


Figure 4.17 Reported Bridge Deck Cracking as a Problem and Perform Crack Inspections (out of 11)



Yes	No	Did Not Specify
Alberta	Connecticut	
Arkansas	Iowa	
Idaho	Louisiana	
Kansas	Maine	
Maryland	North Dakota	
Missouri	Ontario	
New Jersey		
Nova Scotia		
Oklahoma		
Oregon		
Puerto Rico		
Virginia		
Wisconsin		

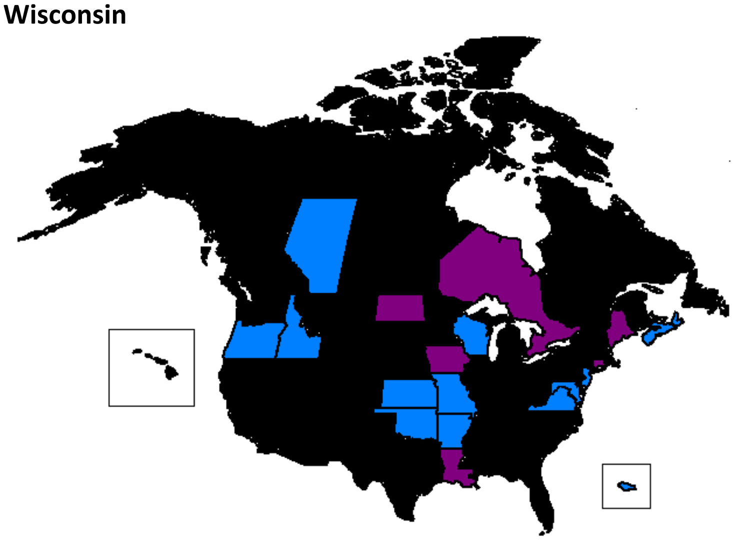
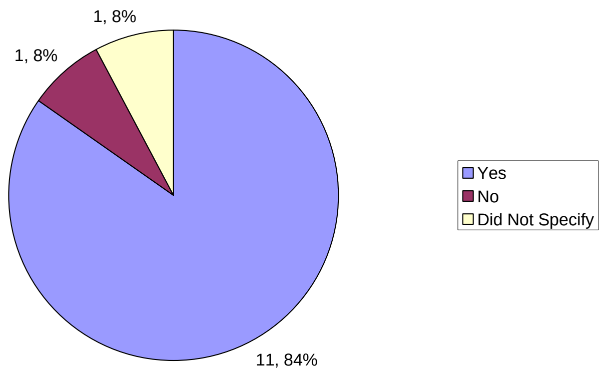


Fig. 4.18 Perform Maintenance to Repair Bridge Deck Cracking (out of 19)



Yes	No	Did Not Specify
Alberta	**Maryland**	**Nova Scotia**
Arkansas		
Idaho		
Kansas		
Missouri		
New Jersey		
Oklahoma		
Oregon		
Puerto Rico		
Virginia		
Wisconsin		

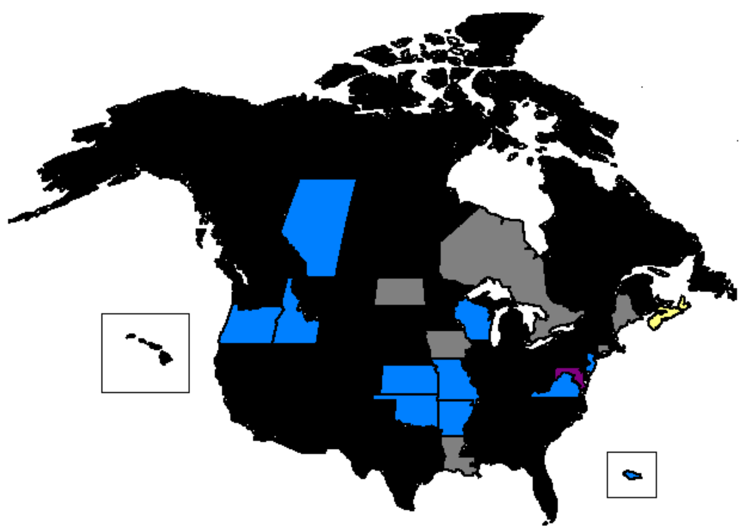
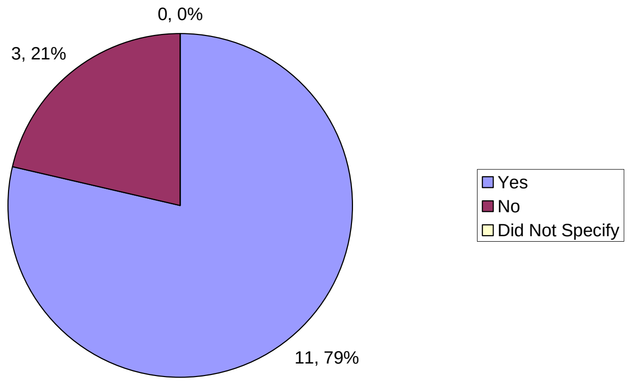


Fig. 4.19 Repair Cracks and Consider Bridge Decks Cracking a Problem (out of 13)



Yes	No	Did Not Specify
Alberta	Connecticut	
Arkansas	Iowa	
Idaho	Ontario	
Kansas		
Maryland		
New Jersey		
Nova Scotia		
Oregon		
Puerto Rico		
Virginia		
Wisconsin		

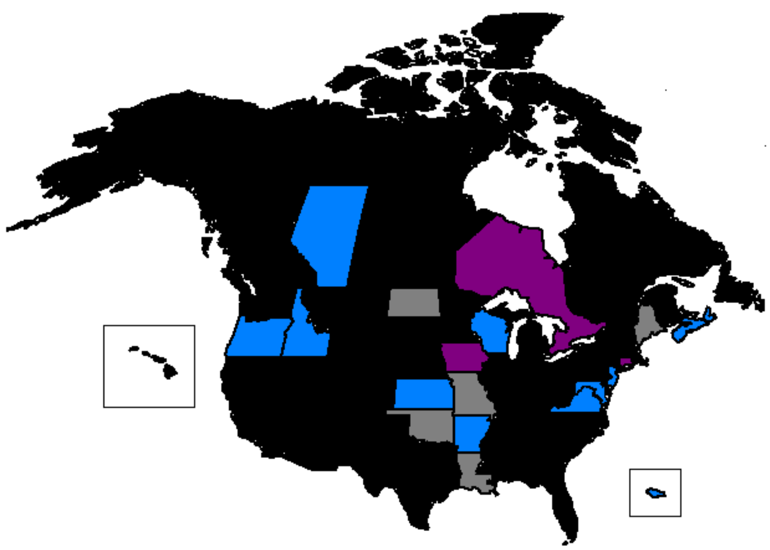
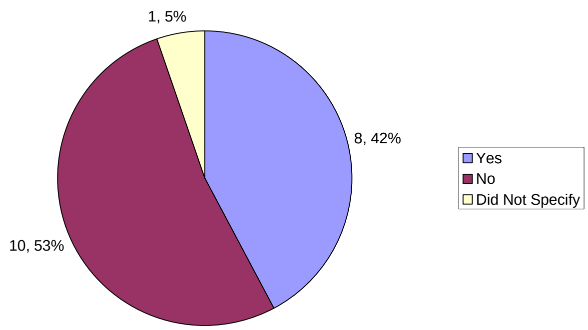


Fig. 4.20 Perform Crack Inspections and Repair Bridge Deck Cracks (out of 14)

The sixth objective was to look for any general structural trends that may influence the amount of cracking that the bridge deck will undergo. Figure 4.21 shows which agencies reported a difference in bridge deck cracking between pre-cast concrete girder and steel girder. Of the 19 respondents, eight (42%) noticed a difference and ten (53%) did not. One (5%) agency did not specify. Alberta reported that cracking is more severe in composite steel girders than in other bridge types. Maine also noticed increased cracking in continuous steel structures. Missouri reported that lighter, more slender structures tend to have more cracks than heavier, more rigid structures. Ontario noticed that decks on steel girders tend to have more cracks than decks on prestressed girders. Virginia believes that the steel girders are more flexible which leads to greater cracks. Oregon reported that the increase in cracking is probably due to the wider spacing of steel girders which would cause construction flaws to be magnified. Arkansas reported no difference in deck cracking between the two, but also reported that that they have much fewer prestressed bridges than steel girder bridges. Figure 4.22 represents which agencies reportedly use different specifications for their bridge decks on precast girders than for their bridge decks on steel girders. Only one (5%) agency, Puerto Rico, reported doing this. The other 18 (95%) agencies do not have separate specifications for the two different bridge systems. Iowa reported no difference in the specifications, but does require sequential concrete placement for two-span continuous steel girder bridges.



Yes	No	Did Not Specify
Alberta	Arkansas	North Dakota
Kansas	Connecticut	
Maine	Idaho	
Maryland	Iowa	
Oklahoma	Louisiana	
Ontario	Missouri	
Oregon	New Jersey	
Virginia	Nova Scotia	
	Puerto Rico	
	Wisconsin	

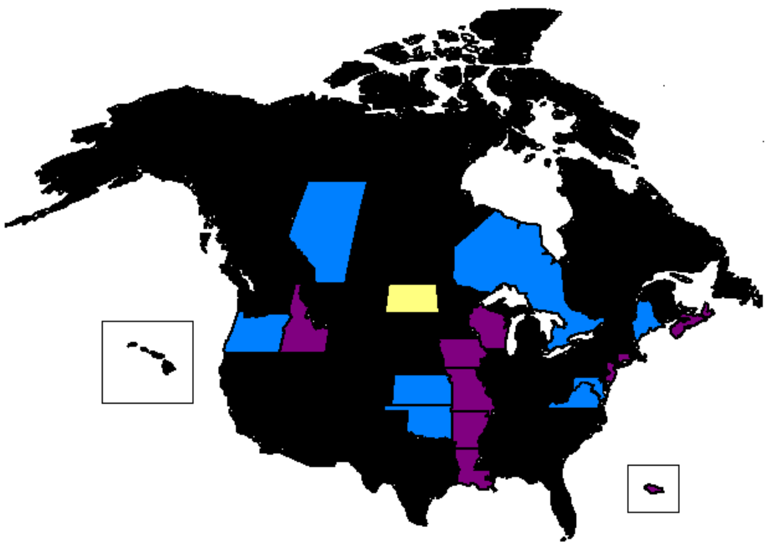
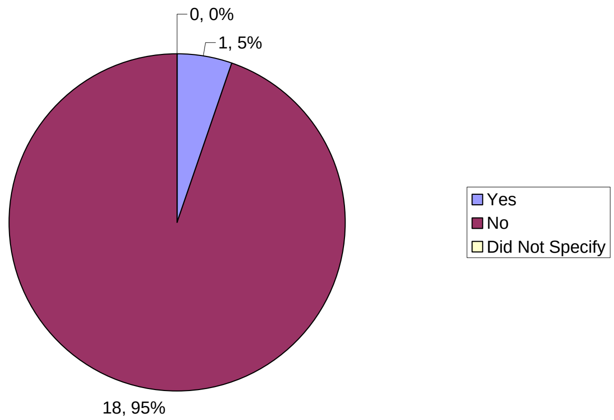


Fig. 4.21 Noticed a Difference in Bridge Deck Cracking between Precast and Steel Girder Bridges (out of 19)



Yes	No	No (cont.)
Puerto Rico	Alberta	Missouri
	Arkansas	New Jersey
	Connecticut	North Dakota
	Idaho	Nova Scotia
	Iowa	Oklahoma
	Kansas	Ontario
	Louisiana	Oregon
	Maine	Virginia
	Maryland	Wisconsin

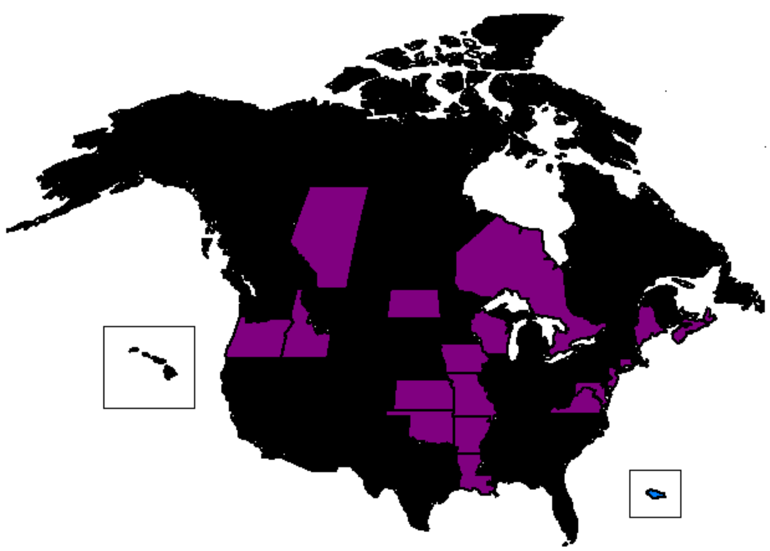
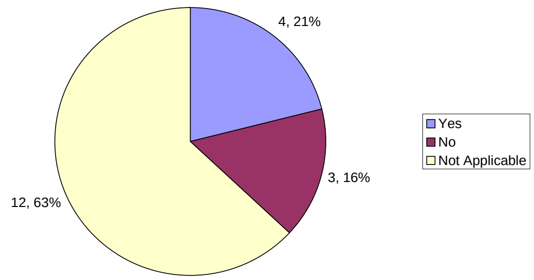


Fig. 4.22 Reported Using Different Specifications for Bridge Decks Supported by Precast Girders versus Steel Girders (out of 19)

The final objective of this questionnaire was to determine the difference in cost for using a low-cracking concrete bridge deck versus a standard deck. Figure 4.23 represents which agencies reported an increased initial cost for using a low-cracking concrete mix compared to a standard concrete mix. Of the 19 respondents, four (21%) reported an increase in the initial costs, three (16%) reported that there is no increase in initial costs, and twelve (63%) replied that this was not applicable. Alberta believes that the increased cost may be from optimizing the concrete mix design. Kansas reported that the increased initial costs are acquired during the research phase. Figure 4.24 represents the difference in recurring costs for a low-cracking concrete deck compared to a standard concrete mix deck, as reported by the agencies. Two (11%) of the agencies, Wisconsin and Alberta, reported an increase in the recurring and maintenance costs for low-cracking concrete. One (5%) agency, Kansas, reported that this information is unknown at this time. Sixteen (84%) reported that this was not applicable. Alberta reported that more cracking results in more maintenance which is not consistent with Figure 4.10, where they reported having success with their implemented measures. Idaho has only tried one low-cracking concrete mix so not enough is known yet. Maryland is still developing a low-cracking concrete mix.



Yes	No	Not Applicable
Alberta	New Jersey	Arkansas
Idaho	Puerto Rico	Connecticut
Kansas	Wisconsin	Iowa
Maryland		Louisiana
		Maine
		Missouri
		North Dakota
		Nova Scotia
		Oklahoma
		Ontario
		Oregon
		Virginia

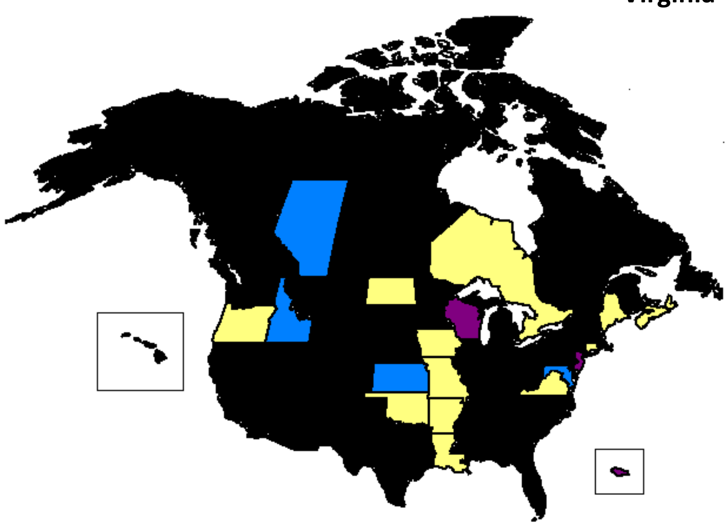
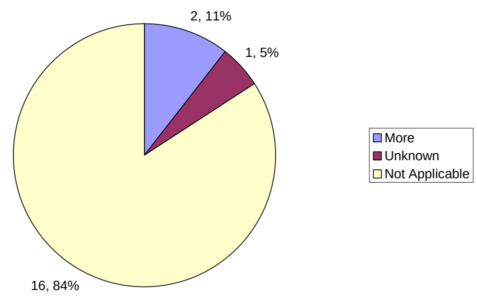


Fig. 4.23 Increased Initial Cost for Low-Cracking Concrete Bridge Decks Compared to Standard Concrete Bridge Decks (out of 19)



More	Unknown	Not Applicable
Alberta	Kansas	Arkansas
Wisconsin		Connecticut
		Idaho
		Iowa
		Louisiana
		Maine
		Maryland
		Missouri
		New Jersey
		North Dakota
		Nova Scotia
		Oklahoma
		Ontario
		Oregon
		Puerto Rico
		Virginia

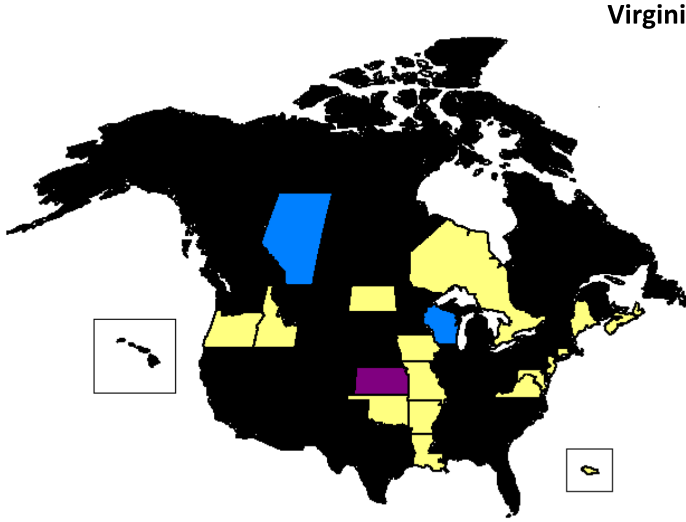


Fig. 4.24 Difference in Recurring and Maintenance Cost for Low-Cracking Concrete Bridge Decks Compared to Standard Bridge Decks (out of 19)

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CHAPTER 5 WORK PLAN

Task 3. Finalize a work plan incorporating previous SDDOT and other research results for monitoring construction and evaluating performance of the experimental bridges.

A work plan was developed for the project to monitor construction and to evaluate the bridge deck performance per SD-DOT's construction schedules and by incorporating previous SD-DOT and other research. The project was successfully completed by following the work plan.

CHAPTER 6 RESULTS OF LABORATORY TESTS

Task 4. Conduct shrinkage tests using SDDOT standard mix designs using South Dakota materials, and using LC-HPC concrete based on the draft specifications, and compare the results.

Laboratory tests were conducted mainly to establish the drying shrinkage performance of LC-HPC and to compare it with that of A45 concrete. The standard fresh concrete properties were also determined for the two concretes in SDSM&T laboratory. The materials for the laboratory mixes were supplied by SD-DOT. The mixture proportions were supplied by Birdsall Sand and Gravel (see Table 6.1). The materials and the mixture proportions supplied were used to mix concrete in SDSMT materials laboratory.

Table 6.1 Mix Design of Standard SDDOT A45 and LC-HPC for Pennington County Bridges

Mix	Standard A45	LC-HPC
w/c	0.42	0.43
Cement (lb/yd^3)	570	563
Fly Ash (lb/yd^3)	125	-
Coarse Aggregate (lb/yd^3)	1710	1375
Medium Aggregate (lb/yd^3)	-	457
Fine Aggregate (lb/yd^3)	1155	1204
Water (lb/yd^3)	283	242
Air Content (%)	6.5	8.0

The fresh concrete properties determined were air content, slump, unit weight and concrete temperature. The tests were conducted per the following ASTM standards:

- Air content – pressure method - ASTM C231
- Slump - ASTM C143
- Concrete temperature - ASTM C1064

The air content determined in the laboratory for A45 concrete was 5.3%, while for LC-HPC the air content was 6.4%. The slump for the mixes was about 3.75 inch and the unit weight of wet concrete was about 148 lb/ft^3. The wet concrete temperature was 72° F (average).

The hardened properties determined in the laboratory tests were the concrete compressive strength at different ages, the static modulus of elasticity of concrete, and drying shrinkage. The tests were conducted per the following ASTM standards:

- Compressive strength - ASTM C39
- Static modulus of elasticity – ASTM C469
- Drying shrinkage - ASTM C157

6.1 Compressive Strength

Three replicates for each of the four sets were tested in the laboratory for the determination of compressive strength of concrete at 7 days, 14 days and 28 days. The test results for A45 concrete and LCHPC are shown in Fig. 6.1. The figure also shows the compressive strength at different ages of both A45 concrete and LC-HPC for the mixes as recorded by SDDOT for a Pennington County bridges (strength of concrete cylinders made from field samples). The test results indicate that compressive strength of LC-HPC is comparable to A45 concrete for the laboratory tests. Similarly, compressive strength of LC-HPC of field concrete is reasonably close to that of A45 field concrete.

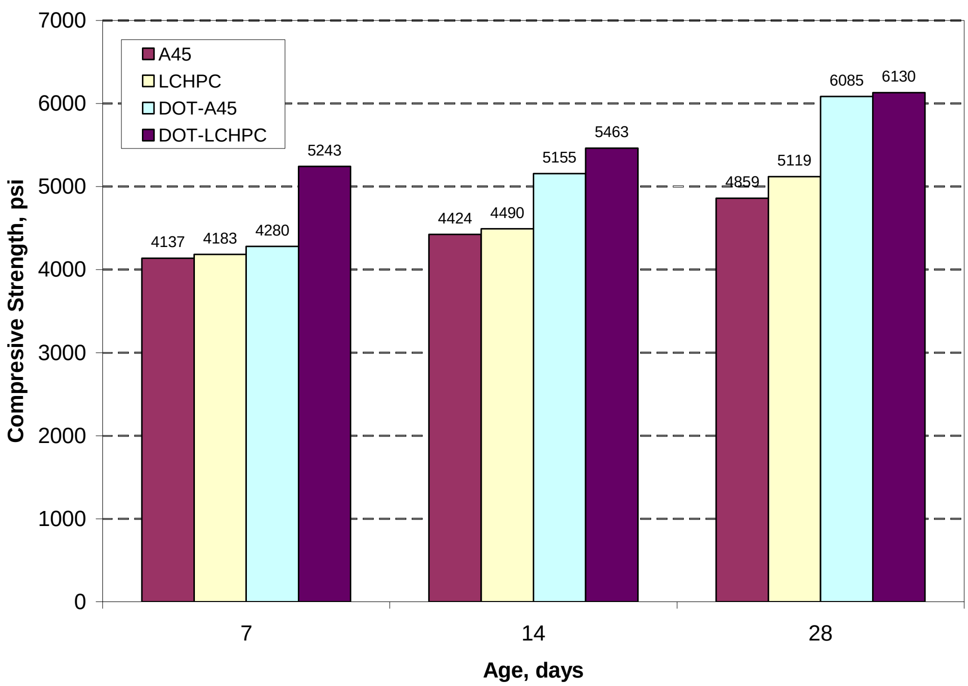


Fig. 6.1 Comparison of Concrete Compressive Strength at Different Ages

The compressive strength of field concrete as determined by SDDOT is about 15 to 20% greater than that of the laboratory concrete. The difference might be because of the different W/C ratio and the amounts of HRWR's (superplastizers) that were used in the contractor's batching plant. The contractor used a W/C ratio of about 0.34 in the actual concrete placement while the laboratory results are based on concrete mixed with a W/C ratio of 0.43 which was from the original mix design (See Appendix B).

6.2 Static Modulus

Elastic static modulus of A45 concrete and LC-HPC was determined at different ages in the SDSMT laboratory. Fig. 6.2 shows the comparison of the static modulus of A45 concrete with that of LC-HPC. At all the three ages (7, 14, and 28 days), the static modulus of LC-HPC was found to be very close to that of A45 concrete.

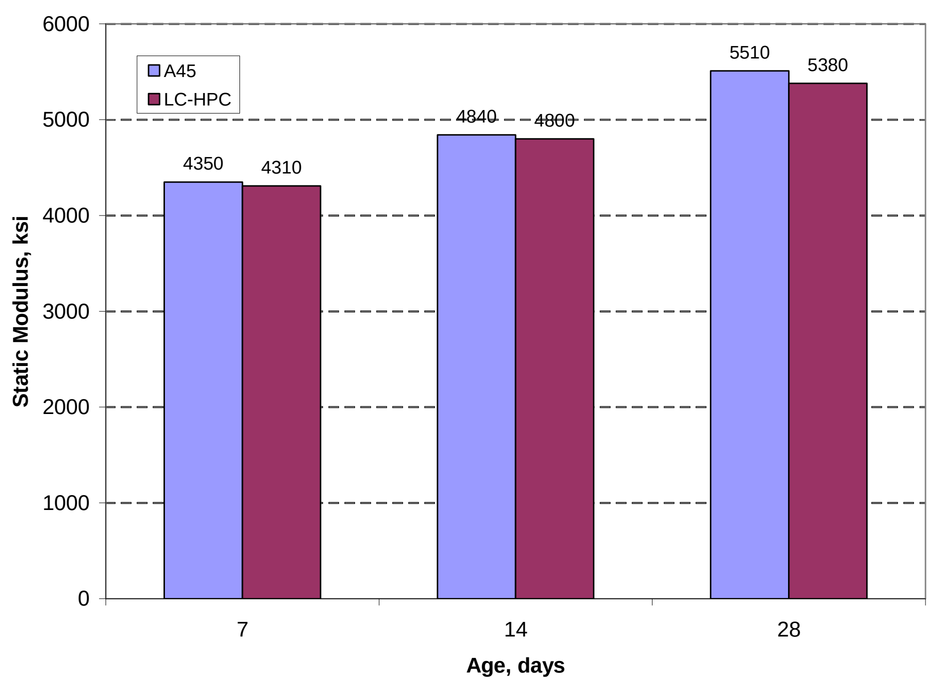


Fig. 6.2 Comparison of Static Modulus of Concrete at Different Ages

6.3 Drying Shrinkage

The drying shrinkage deformations of the concrete specimens made in the laboratory were evaluated for sixty days. Three replicate specimens of size 11.25 inch x 3 inch x 3 inch (286 mm x 75 mm x 75 mm) per mix were used to evaluate the shrinkage deformations. The development of drying shrinkage deformations over 60 days for A45 concrete, and the corresponding values for LC-HPC are shown in Fig. 6.3. At the end of sixty days, A45 concrete had a unit drying shrinkage strain less than 250 x 10^{-6}, while the unit drying shrinkage strain of LC-HPC was found to be less than 150 x 10^{-6}. The drying shrinkage of LC-HPC was less than that of A45 concrete for most of the duration of tests. Shrinkage strains of both A45 concrete and LC-HPC were much lower than expected.

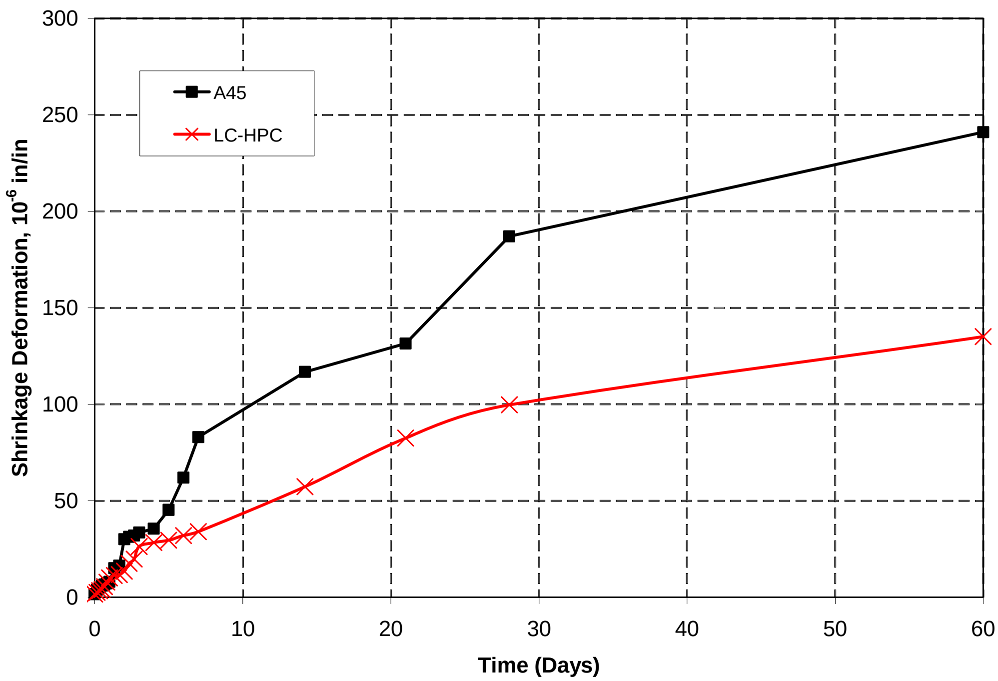


Fig. 6.3 Comparison of Drying Shrinkage Strains of A45 Laboratory Concrete with the Drying Shrinkage Strains of LC-HPC

CHAPTER 7 DOCUMENTED PROPERTIES OF FIELD CONCRETE

Task 5. Document the concrete physical properties (seven, fourteen, and twenty-eight day compressive strengths, air content, slump, and temperature) of SDDOT-furnished mix designs, for the control bridges and the LC-HPC concrete used on this project in the final report.

SDDOT engineers recorded properties of concrete for the two bridge decks. The properties recorded were wet concrete properties measured at the location of the pours and compressive strength results as determined by the DOT personnel at different ages in their central laboratory. DOT engineers also recorded temperature measurements for the concrete decks using I-buttons. Datasheets of the test results and temperature measurements were provided by the DOT engineers to the researchers and the results were summarized by the researchers in Tables 7.1 and 7.2 for the Pennington County bridges and in Tables 7.3, 7.4 and 7.5 for the Brule County bridges. It should be noted that the control (westbound) bridge deck in Brule County was cast in two separate placements (July 26, 2006 and August 14, 2006). The first placement was discontinued by SDDOT engineers due to deficient concrete placement and finishing techniques. Those deficiencies are discussed in Chapter 8.

The wet concrete properties recorded by SD-DOT technicians were:

- Air content
- Slump
- Concrete temperature
- Unit weight

The hardened concrete properties recorded are the concrete compressive strength at different ages.

Table 7.1 Fresh Concrete Properties of A45 Concrete-Pennington County

Pennington County - West Bound Bridge Deck						
Date of Pour - Oct. 18, 2005						
Time	Air Content	Slump	Conc. Temp.	Air temp	Unit Weight	W/C Ratio
	%	Inch	Deg F	Deg F	Lb/Ft^3	
7:30 AM	6.1	2.2	69	59	143.9	0.30
8:30 AM	8.4	3.0	71	70	139.2	0.36
9:00 AM	6.6	2.7	70	70	143.1	0.34
10:00 AM	6.5	2.5	71	72	143.1	0.34
10:45 AM	7.2	3.2	71	65	142.5	0.34
10:45 AM	7.2	3.2	71	65	142.5	0.34
Max	8.4	3.2	71	72	144	0.36
Min	6.5	2.5	70	65	139	0.34
Average	7.0	2.8	70	67	142	0.34

Table 7.2 Fresh Concrete Properties of LC-HPC-Pennington County

Pennington County - East Bound Bridge Deck						
Date of Pour - Oct. 29, 2005						
Time	Air Content	Slump	Conc. Temp.	Air temp	Unit Weight	W/C Ratio
	%	Inch	Deg F	Deg F	Lb/Ft^3	
8:00 AM	6.1	1.8	62	48	147	0.39
8:00 AM	6.1	1.8	62	48	147	0.39
8:30 AM	6.7	3.0	62	49	146	0.48
9:30 AM	7.1	2.5	61	54	145	0.38
9:30 AM	7.1	2.5	61	54	145	0.38
12:00 PM	8.9	2.3	62	54	142	0.37
12:00 PM	7.7	2.3	63	56	144	0.50
12:00 PM	7.7	2.3	63	56	144	0.50
1:15 PM	7.8	2.3	63	65	143	0.36
Max	8.9	3.0	63	56	147	0.50
Min	6.1	1.8	61	48	142	0.37
Average	7.2	2.3	62	52	145	0.42

Table 7.3 Fresh Concrete Properties of A45 Concrete, First Pour-Brule County

Brule County - West Bound Bridge Deck-First Pour						
Date of Pour – July 26, 2006						
Time	Air Content	Slump	Conc. Temp.	Air temp	Unit Weight	W/C Ratio
	%	Inch	Deg F	Deg F	Lb/Ft^3	
12:00 AM	7.7	2.75	77	72	139.2	0.30
1:00 AM	6.5	2.50	75	72	141.9	0.33
1:35 AM	6.0	2.50	75	72	143.0	0.34
1:35 AM	5.7	2.50	75	72	143.3	0.33
2:30 AM	6.8	2.75	78	72	142.0	0.38
2:35 AM	5.2	2.00	77	72	144.8	0.38
Max	7.7	2.75	78	72	139.2	0.38
Min	5.2	2.00	75	72	144.8	0.30

Table 7.4 Fresh Concrete Properties of A45 Concrete, Second Pour-Brule County

Brule County - West Bound Bridge Deck-Second Pour						
Date of Pour – August 14, 2006						
Time	Air Content	Slump	Conc. Temp.	Air temp	Unit Weight	W/C Ratio
	%	Inch	Deg F	Deg F	Lb/Ft^3	
10:25 PM	6.2	2.25	81	76	142.6	0.34
11:15 PM	5.2	1.25	77	74	145.9	0.33
11:38 PM	5.5	1.50	84	72	145.7	0.35
1:00 AM	6.3	3.00	81	67	142.7	0.36
2:00 AM	7.2	3.00	75	61	141.8	0.36
3:30 AM	6.3	2.25	85	60	143.2	0.37
Max	7.2	3.00	85	76	145.9	0.37
Min	5.2	1.25	75	60	141.8	0.33

Table 7.5 Fresh Concrete Properties of LC-HPC-Brule County

Brule County - East Bound Bridge Deck						
Date of Pour – June 27, 2007						
Time	Air Content	Slump	Conc. Temp.	Air temp	Unit Weight	W/C Ratio
	%	Inch	Deg F	Deg F	Lb/Ft^3	
4:10 AM	6.0	1.25	60	58	145.1	0.38
4:25 AM	6.9	2.75	62	59	142.3	0.38
4:35 AM	6.6	3.25	62	59	142.5	0.38
5:10 AM	6.5	2.00	62	59	143.9	0.40
5:35 AM	6.5	2.50	65	59	143.6	0.39
6:10 AM	7.2	2.75	63	58	140.9	0.37
6:55 AM	7.4	2.50	60	57	141.7	0.37
7:55 AM	6.6	1.25	67	61	143.6	0.36
8:20 AM	7.1	2.75	68	63	142.4	0.36
9:00 AM	7.6	2.75	68	67	141.6	0.37
Max	7.6	3.25	68	67	145.1	0.40
Min	6.0	1.25	60	57	140.9	0.36

7.1 A45 Concrete Used in West Bound Bridge Deck in Pennington County

The fresh concrete test results for A45 concrete as the contractor unloaded concrete from trucks at site for the west bound bridge deck as performed by SDDOT are summarized in Table 7.1. Slump of the concrete varied between 2.5 to 3.2 inches, and air content varied from 6.5 to 8.4% with an average air of 7%. The concrete temperature was between 70 and 71° F. The average unit weight of concrete was 142 lb/ft^3.

The development of compressive strength of A45 field concrete over 28 days is shown in Fig. 7.1 and Fig. 7.2. Two sets of concrete specimens were tested for A45 concrete.

7.2 LC-HPC used in East Bound Bridge Deck in Pennington County

Slump of the concrete varied between 1.8 to 3 inches with an average slump of 2.3 inch, and air content varied from 6.1 to 8.9% with an average air of 7.2%. The concrete temperature was 61 to 63° F. The unit weight of fresh concrete was between 142 and 147 lb/ft^3 with an average of 145 lb/ft^3.

The development of compressive strength of LC-HPC field concrete over 28 days is shown in Fig. 7.1. Three sets of concrete specimens were tested for LC-HPC concrete. All the three corresponding curves are shown in the figure along with those for the two sets of A45 concrete specimens. All the cylinders were made to 4 inch diameter and 8 inch length.

The temperature within the deck concrete was monitored at four stations. The four stations were located as follows:

- Temp#1: NE corner approx. 10' from the east end 10' in from the north edge on the top mat
- Temp#2: NE corner approx. 12' from the east end 3' in from the north edge this is a HIGH/ LOW placed on the surface under the deck coverings
- Temp#3: south edge approx 100' from the east end 6' in from the south edge top mat
- Temp#4: same location as Temp #3 but for the bottom mat

SD-DOT recorded the temperatures and provided the data to the researchers. The temperature plots for the four stations are shown in Fig. 7.3. The four temperature readings were averaged and the average temperature is plotted with time in Fig. 7.4. In general, the concrete temperature was in the high 70’s soon after the placement of concrete, possibly due to cement hydration. The temperature decreased to below 60° F within the first two days of curing and gradually reduced to 40's in the following days. The ambient temperature during the curing period of the deck was very low (in 30's and 40's) and the concrete temperature also followed the general trend of the ambient temperature.

7.3 Comparison of A45 Field Concrete with LC-HPC in Pennington County Bridges

From Fig. 7.1, the strength development of the two concretes is reasonably close to each other. A45 concrete seemed to develop strength at a slower rate, but the strengths are within 200 psi at 28 days. The concrete compressive strengths over 28 days are also shown in Fig. 7.2 at different ages. The figure shows the averaged test results of the two sets of specimens for A45 concrete and the average test results of the three sets of specimens for LC-HPC. These two figures demonstrate that there is no discernable difference between the compressive strength of A45 concrete and that of LC-HPC concrete at 28 days. Fig. 7.2 indicates that the LC-HPC concrete developed better strength during the initial few days of curing.

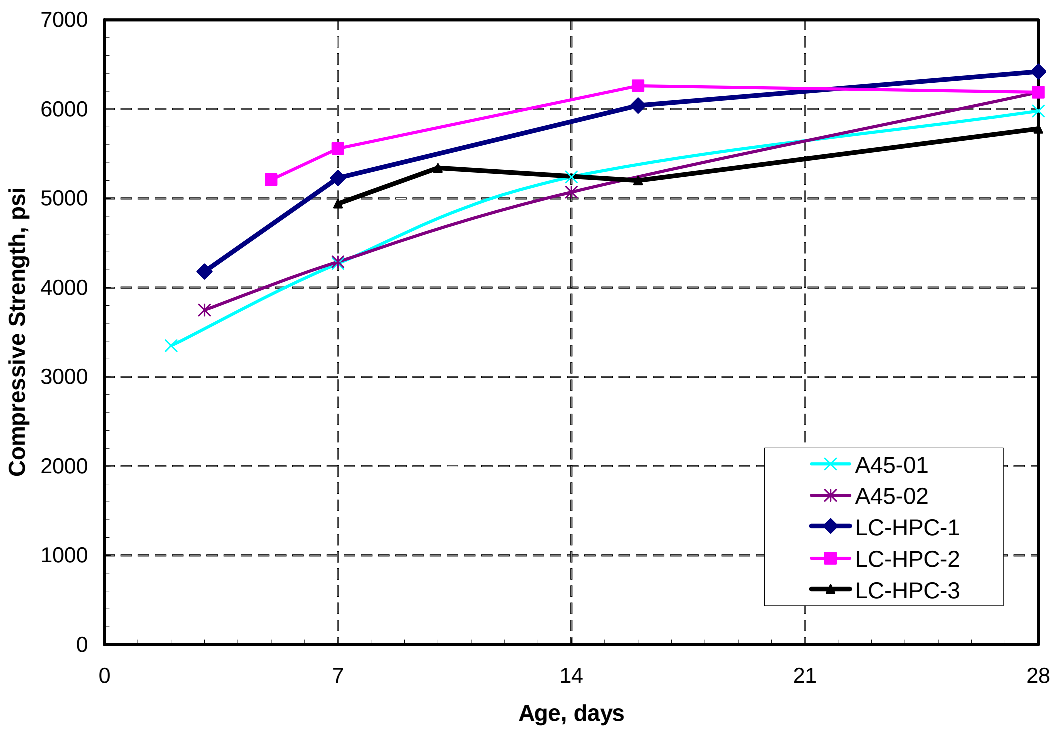


Fig. 7.1 Development of Concrete Compressive Strength Over 28 Days

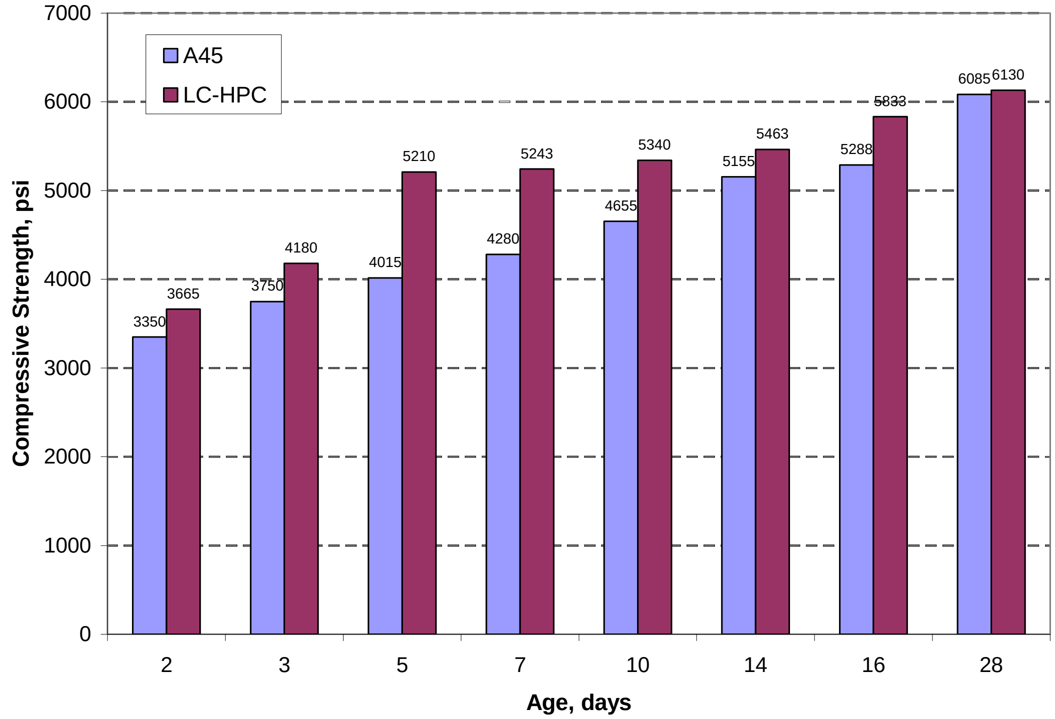


Fig. 7.2 Comparison of Compressive Strength of A45 Concrete with that of LC-HPC over 28 Days

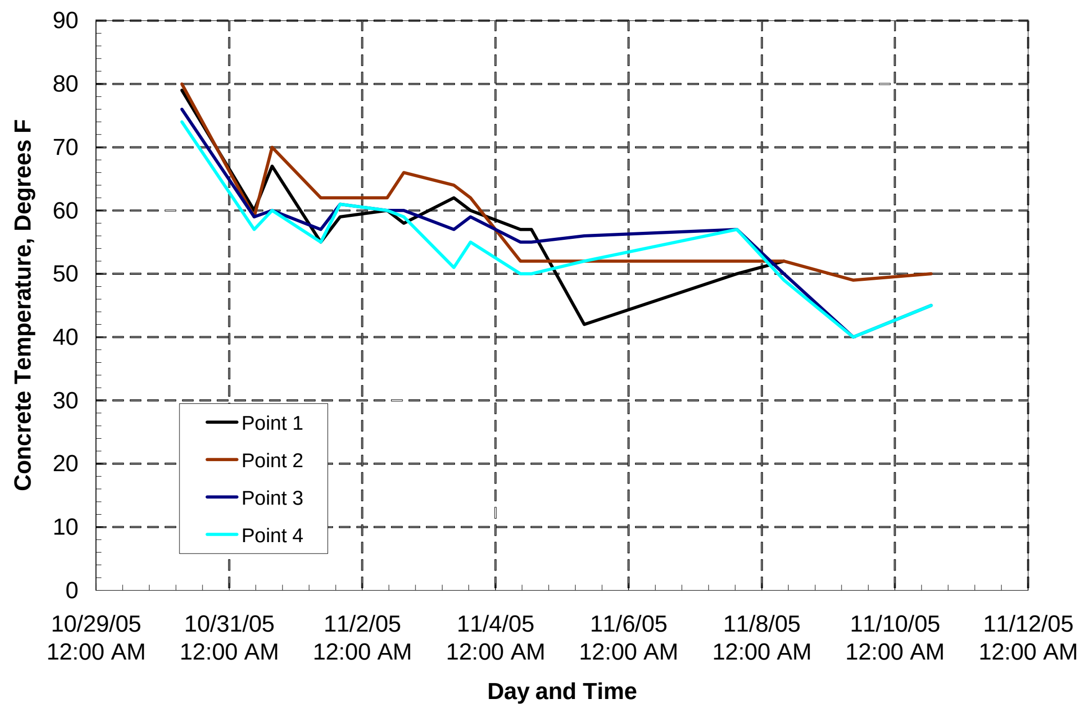


Fig. 7.3 Temperature Recorded in the Concrete of LC-HPC Bridge Deck at Four Points

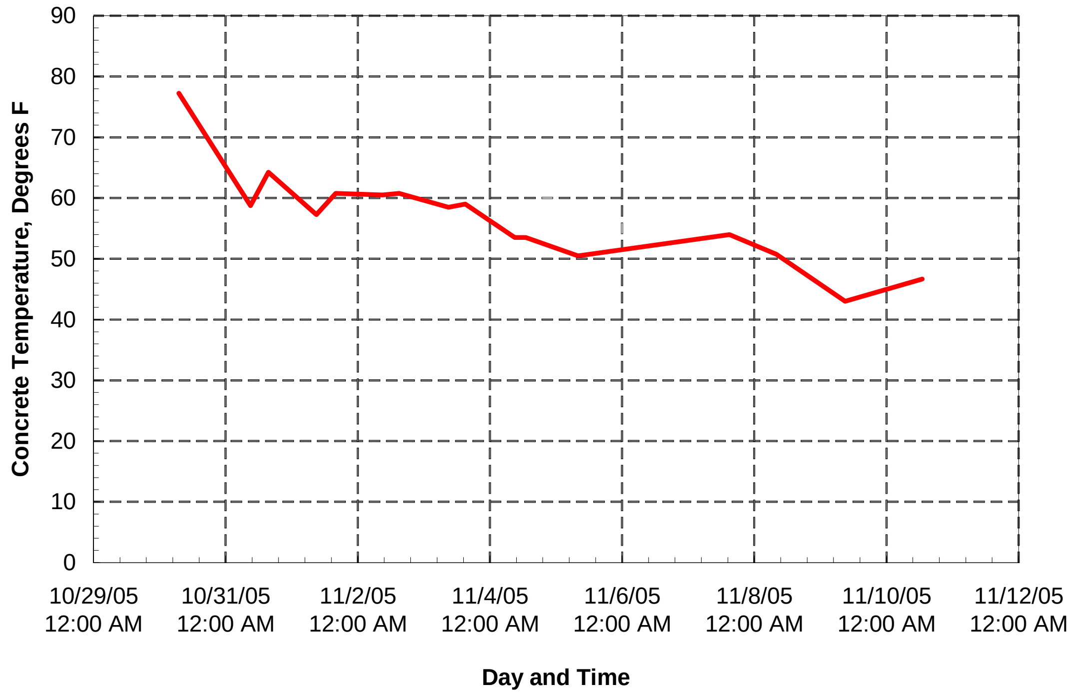


Fig. 7.4 Average Temperature Recorded in the Concrete of LC-HPC Bridge Deck

7.4 A45 Concrete Used in West Bound Bridge Deck in Brule County

The fresh concrete test results for A45 concrete are summarized in Table 7.3 for the first concrete placement and in 7.4 for the second concrete placement.

The fresh concrete properties were measured and recorded by SDDOT personnel as the contractor unloaded concrete from trucks on the construction site. For the first placement, slump of the concrete varied between 2.00 to 2.75 inches, air content varied from 5.2 to 7.7%, concrete temperature was between 75 and 78°F, the unit weight of concrete varied between 139.2 and 144.8 lb/ft^3, and the w/c ratio as reflected on the delivery tickets ranged between 0.30 and 0.38. For the second placement, slump of the concrete varied between 1.25 to 3.00 inches, air content varied from 5.2 to 7.2%, concrete temperature was between 75 and 85°F, the unit weight of concrete varied between 141.8 and 145.9 lb/ft^3, and the w/c ratio as reflected on the delivery tickets ranged between 0.33 and 0.37.

The concrete strength data for the A45 deck concrete were not furnished to the research team and are not presented in this report.

7.5 LC-HPC Used in East Bound Bridge Deck in Brule County

The fresh concrete test results for LC-HPC concrete are summarized in Table 7.5.

The fresh concrete properties were measured and recorded by SDDOT personnel as the contractor unloaded concrete from trucks on the construction site. Slump of the concrete varied between 1.25 to 3.25 inches, air content varied from 6.0 to 7.6%, concrete temperature was between 60 and 68°F, the unit weight of concrete varied between 140.9 and 145.1 lb/ft^3, and the w/c ratio as reflected on the delivery tickets ranged between 0.36 and 0.40.

The results of only three 28-day concrete compressive strength tests were furnished by SDDOT to the research team. The reported compressive strength results were 5,890 psi, 4,880 psi, and 4,580 psi with an average 28-day strength of 5,117 psi.

CHAPTER 8 CONSTRUCTION MONITORING AND CONSTRUCTIBILITY ISSUES

Task 6. Monitor construction of the control bridge decks and the LC-HPC bridge decks, including test slabs, and document the results. Submit an interim report, after construction of each of the four bridges, detailing constructability issues and suggestions of best practices for use on future bridges.

PART A: PENNINGTON COUNTY BRIDGES

8.1 Pennington County Test Slab

A test slab was constructed using the special provision of LC-HPC to test the constructability of the new concrete. This was done in the yard of Heavy Construction (the contractor for the project) in Rapid City. The pour was monitored by several engineers, researchers and graduate students from the two universities. The observations of each monitor were documented on a feedback form that was developed to capture all the special features of LC-HPC. This section outlines the summary of the comments of the respondents.

8.1.1 General Details

Location: Heavy Construction's Yard at 4101 Deadwood Avenue, Rapid City, SD57702

Test Slab:	LC-HPC Oct. 11, 2005 - Pour Started 12:55 PM and Ended at 3:00 PM.
Comments from:	SDDOT: 2 (construction), 1 (research), Heavy: 1 (construction), SDSU: 2, SDSMT: 2

8.1.2 Concrete Quality and Proportion

It was generally unknown if the mix design for concrete was developed for pumping. While the concrete was meant to be for pumping per contractor, difficulties arose from pumping.

8.1.3 Concrete Placement

Preparations were generally adequate. Free fall of concrete was less than 5 ft. Concrete temperature was between 50° F and 75° F. Placement rate was too slow.

Two respondents (one from Heavy) thought the additional arrangements cost more for LC-HPC than for A45 concrete.

8.1.4 Consolidation

Vibrator setup (see Fig. 8.1) worked well. Vibration time needed to be increased for proper consolidation. A backup for possible breakdown of the system needs to be in place. Delays in pumping concrete interfered with consolidation.

Vibrator mounting and related arrangements cost more than for A45 concrete.

8.1.5 Finishing

All eight observers were dissatisfied with finishing (Figs. 8.2 and 8.3). While hand finish was kept to a minimum, the placement rate and concrete (was stiff initially) affected the finish. In the later half of pour, the finish got better than the first half. However, fines were removed excessively from the concrete and pushed aside by the finishing machine rollers (Fig. 8.2).

8.1.6 Prevention of Rapid Evaporation

The respondents were unsure of the evaporation rate. However, the evaporation rate was checked per ACI 305R by the researchers based on weather data and concrete temperature, and was found to be within 0.2 lb/sq. ft/hr. A good record of the weather data was maintained throughout the pour. The location of the weather meter needed to be changed so as to meet specifications.

There were problems noted by observers that include uncontrolled fogging that caused puddles of water on the concrete surface (Fig. 8.4). This led to washing out of cement paste in small patches close to the nozzles where the fogging was excessive. Some of the nozzles did not function well. The end result was either excessive fogging or too little of it. This needed to be addressed. The cost increase due to revised specifications was minor and was unknown.

8.1.7 Curing

Six out of eight observers felt that curing did not commence within 10 minutes of concrete strike off. The curing material (blanket) was not in place within 10 minutes of strike-off. It took as much as 2 hours to cover concrete after strike-off. This process needed to be expedited for the bridge deck pour. The soaker pipes were flooding the concrete surface at the edges of the sheets and at the location of the overlap between sheets (Fig. 8.5). The concrete finish was severely affected at these locations due to the puddles caused by soaker pipes that washed out the cement from the surface. The blanket widths needed to be adjusted to increase the overlap and to cover the slab edges so that water would not reach under the sheets or collect on the concrete surface.

8.1.8 Compliance of Frequency of Field Tests

Slump of the concrete was within the specified limits. Air content was acceptable for load 1 and marginal for load 4 (9.5%). For all other loads (2, 3, and 5), air was in the range that required rejection (<6.5% or >9.5%). Unit weight fluctuated between 138 and 146 lb/ft^3. The concrete temperature was acceptable (values varying from 64° F to 74° F) with the last two loads almost at the limit (75° F).

From the field tests, the quality of concrete was described as ranging from “inconsistent” to “needs to be rejected”. For test slabs, this description was inconsequential and no loads were actually rejected because it was a test slab. If it were in the actual decks, these loads would merit rejection.

8.1.9 Overall Scores

From the scores given by the observers, the test slab pour performed “Fairly” with an average score for the special provisions of 3.33 and average score for constructability of 2.8. The rating was based on a score of 5 maximum for best performance, and 1 minimum for unacceptable performance.

8.2 Pennington County A45 Concrete Deck (West Bound)

A summary of observation on the constructability of the west bound A45 concrete bridge deck is given in this section.

8.2.1 General Details

Location: A45 Bridge Deck – I-90 West Bound over East North Street, Pennington County.

A45 Concrete: 10/18/2005 - Pour Started 7:45 AM and Ended at 1:00 PM.

8.2.2 Weather Data

The atmospheric temperature was about 40° F at the start of the pour and gradually increased to about 70° F close to the finish of the pour. The wind velocity was very mild for the first half of the pour, and varied between 0 mph to about 4 mph. However, in the second half, particularly towards end of the pour and during finishing operations, wind was blowing with speeds of above 30 to 40 mph. The day also felt warm and had low humidity. A general view of the deck pour is shown in Fig. 8.6.

8.2.3 Concrete Quality and Proportion

The mix design for concrete was developed for pumping and is the same as the A45 mix design that is normally used by SD-DOT for bridge decks. The concrete delivery was uninterrupted. The concrete was easily pumpable generally with good consistency.

8.2.4 Concrete Placement

Placement rate was considered slow with an average 30 to 40 feet of bridge deck length poured per hour.

8.2.5 Consolidation

Consolidation was done with hand-held needle vibrators as is the normal practice for construction of DOT bridge decks. Two to four vibrator operators were vibrating concrete at any time (Fig. 8.7). The consolidation seemed good. The vibration duration was about right.

8.2.6 Finishing

The finishing of the bridge deck was found to be mostly satisfactory at the time of placement. Hand finish was kept to a minimum at the edges. The slow placement rate and workability of concrete allowed, what seemed at the time of placement, good finishing (Fig. 8.8). The only problem observed at the time of placement was at the Southwest corner of the slab. Heavy wind was blowing when placing concrete at that corner (last 30 ft. length of the deck). Therefore there was some difficulty in finishing the deck. Foggers were mounted on the cross beam of the finishing machine (Fig. 8.9). There was some difficulty in controlling the foggers to regulate the amount of fogging.

After the setting of concrete, DOT engineers observed open graded texture on the top surface of the bridge deck in the Northeast corner. The bridge top surface was devoid of fines and showed a rough surface with exposed aggregate. While the reason for poor finish of the surface is not conclusively known, it is believed that lack of retarder in the concrete mix was unhelpful in finishing the deck surface under highly windy conditions at the time of concrete placement. The deck surface roughness was repaired with high strength polymer grout prior to grooving.

8.2.7 Prevention of Rapid Evaporation and Curing

Continuous fogging was done during and after placement of concrete. Nonwoven polypropylene curing blankets (Fig. 8.9) were placed in three widths to cover the entire width of the deck and were saturated with water.

8.2.8 Compliance of Frequency of Field Tests

The fresh concrete properties such as slump, air content, unit weight and concrete temperature were tested at site per specifications.

8.2.9 Concrete Quality as Observed at Site

Slump of the concrete varied between 2.5 to 3.25 inches, and air content varied from 6.5 to 8.4% with an average air of 7%. The concrete temperature was between 70 and 71° F.

8.2.10 Overall Observation

The construction of the A45 bridge deck was generally no different from the several other A45 bridge decks that are routinely constructed by SDDOT other than for the surface texture that needed some repair. This bridge deck serves as good basis for comparison between the A45 bridge deck and the companion LC-HPC bridge deck.

8.3 Pennington County LC-HPC Concrete Deck (East Bound)

A summary of observation on the constructability of the east bound LC-HPC bridge deck is given in this section.

8.3.1 General Details

Location:	LC-HPC Bridge Deck – I-90 East Bound over East North Street, Pennington County
LC-HPC	10/29/2005 - Pour Started 8:00 AM and Ended at 1:30 PM.
Comments from:	SDDOT: 2 (construction) & 1 (research), SDSU: 1, SDSMT: 1

8.3.2 Weather Data

The weather at the time of the pour was ideal for placing concrete. The temperature was 50° F at the start of the pour and gradually increased to 60° F close to the finish of the pour. The relative humidity was 80% at the start and 60% close to the finish of the pour. The wind velocity was mild and varied between 0 mph to about 5 mph. There were light showers (drizzle) for about one hour at around 11:25 AM for about 20 to 30 minutes increasing the humidity to approximately 80%. The weather meter was installed one foot over the surface of the concrete.

8.3.3 Falsework

The falsework might have cost the contractor more than that of the falsework for A45 concrete. The additional cost is due to the need to support additional weight of the wet curing fabric during the curing period, and the girder supporting the frame supporting the vibrators.

8.3.4. Concrete Quality and Proportion

It was generally unknown if the mix design for concrete was developed for pumping or for conveyor placement. While the concrete was meant to be for pumping per contractor, difficulties arose from pumping during the test slab pour. Therefore, the contractor used a conveyor (Fig. 8.10). However, there seemed to be no problem with the switch.

The concrete delivery was uninterrupted and smoother than that of the test slab. The concrete was generally with good consistency and not too stiff nor too flowy (Fig. 8.11). The workability was adjusted with the addition of superplasticizer.

There are no special requirements in DOT specifications for mix design for conveyor placement of concrete.

8.3.5 Concrete Placement

Concrete was placed with a conveyor. Preparations were generally adequate. Free fall was less than 5 ft. Concrete temperature was between 50° F and 75° F. Ambient conditions were very favorable during the pour.

Placement rate could have been slightly better. Early on things were a little slow. After the first 20 or 30', the placement became more systematic. The contractor was frequently having trouble getting the air content where they wanted it.

It is generally thought that the use of conveyor costs “significantly” more (five times as much) than pumping.

Cold weather protection was required to be maintained after the pour for the duration of the curing period.

8.3.6 Consolidation

The vibrator setup is sown in Fig. 8.12 and was observed to have worked well. Vibration time was increased as required for proper consolidation compared to test slab placement.

Due to the number of vibrators mounted on the mechanism and the width of the deck, the spacing was not on even multiples of the width of the machine. For this reason, two locations received double vibration. There was no walking back through the vibrated concrete. The bridge holding the vibrating equipment prevented that. Vibrator mounting and related arrangements were noted by some of the observers to cost more than for A45 concrete.

8.3.7 Finishing

The drag pan behind the finish machine was not attached correctly at the beginning of the pour, and the finish was poor as a result. Contractor made adjustments and finishing was much better after making the adjustment (Figs. 8.13 and 8.14).

The finishing of the bridge deck was mostly satisfactory and much better than what was achieved for the test slab. Hand finish was kept to a minimum. The placement rate and workability of concrete allowed good finishing. Fines were not removed excessively from the concrete by the finishing machine rollers. Finish machine ran out of gas at approximately 11:10 PM which was approximately 100 ft from the west end of the bridge. There was some interruption in finishing due to this.

No excessive fog spray was required prior to the application of curing materials. No fogging was required close to concrete surface to finish concrete (Fig. 8.15). There were some minor problems with water from curing blankets and fogging running into the curb lines in the areas (Fig. 8.16) where some hand finishing was required. Some of this water may have been worked into the surface and may result in some scaling.

Some deviations from the special provisions were allowed:

- Two rollers were used to finish in lieu of one roller as specified.
- Use of the rota-vibe on the finish machine was allowed.

It is not documented if grinding of the finished concrete surface was required after concrete curing period.

8.3.8 Prevention of Rapid Evaporation

Air temperature, humidity, and wind speed were measured 12 inches above the deck. The location of the weather meter met the specifications. A good record of the weather data was maintained throughout the pour. The evaporation rate was checked per ACI 305R based on the weather data and concrete temperature, and was found to be within 0.2 lb/sq. ft/hr.

Continuous fogging was done during and after placement as needed per specifications. Weather conditions were ideal for placing concrete and therefore, the importance of fogging is somewhat diminished. The atmospheric conditions during the pour were ideal with overcast skies, cool temperature and some fog to mist type precipitation. Fogging will be very critical if concrete is placed in hot weather.

There was some perceived misconception as to what extent of fogging was to be required. Some attending the pour were of the assumption that "Continuous Fogging" meant that the foggers had to be left on continuously. The real intent was to be such that fogging was used (continuously or intermittently) to maintain a continuous sheen on the concrete surface.

8.3.9 Curing

Three out of the five observers felt that curing did not commence within 10 minutes of concrete strike off. Others felt that curing commenced within ten minutes of the strike-off with strike-off defined as the burlap drag. The exception was at the beginning and ending of the pour at the headers where additional time was required for hand finishing. Excessive fog spray was not required prior to the application of curing materials. The concrete was not allowed to dry after strike-off.

Adequate fogging was done to maintain a continuous sheen on the concrete surface. Nonwoven polypropylene curing blankets were placed on the deck in a timely manner (Fig. 8.17). The contractor had some minor problems with water from curing blankets and fogging running into the curb lines in the areas where the hand finishing was required. Some of this water may have been worked into the surface and may result in some scaling.

Conditions were nearly perfect for placement of concrete. Sheen of water developing under the fogging nozzles was maintained by regulating fogging and the concrete surface was kept moist. The humidity over the wet concrete surface remained around 90 to 100% during the placement. Due to ideal weather conditions for concrete placement at the time of the pour, commencement of curing within 10 minutes of strike-off was not very important for this pour.

The wet cure was in place for 21 days with the exception of being removed for grooving at 20 days and then replaced. Cold weather protection was required to be maintained after the pour for the duration of the curing period.
The definition of "strike-off" needs clarification.

8.3.10 Compliance of the Requirements for Frequency of Field Tests

The fresh concrete properties such as slump, air content, unit weight and concrete temperature were tested at site per specifications by four or five testers continuously.

The special provisions for LC-HPC stated that out-of-spec material (as a result of testing at the truck) was not to be allowed to be placed on the deck. During the above testing, it became apparent that the air content tests from the very back of each truck were not representative of the entire load. The concrete had high air content at the back of the truck, but in the middle of the load and out on the deck, the air contents were within specifications. Therefore, concrete with high initial air content was allowed to be placed on the deck with the expectation that the air would be within the specification once placed.

8.3.11 Concrete Quality as Observed at Site

Slump of the concrete varied between 1.75 to 3 inches with an average slump of 2.3 inch, and air content varied from 6.1 to 8.9% with an average air of 7.8%. The concrete temperature was 61 to 62° F.

8.3.12 Overall Performance

From the scores given by the observers, the LC-HPC deck slab pour performed “good” to “fair” with an average score for the special provisions of 3.7 and average score for constructability of 3.3. The rating was based on a score of 5 maximum for best performance, and 1 minimum for unacceptable performance.

8.3.13 Additional Comments

Weather conditions were excellent for this pour, or at least as good as can be expected. How this mix behaves under hot dry summer conditions is still an open question. Conditions did not test constructability related to prevention of rapid evaporation during the placement. LC-HPC concrete seems to be a good mix design but needs continuous fogging and/or high humidity to get a good surface. The concrete supplier had some difficulty maintaining the air content near the upper end since tests off the end of the belt placer were much lower than at the back of truck.

The contractor’s personnel and SDDOT personnel worked extremely hard to make the LC-HPC pour very successful. Overall, it is believed that an excellent bridge deck was constructed.

Fig. 8.1 Vibrator Setup

Fig. 8.2 Fines from Concrete Being Excessively Removed by the Roller

Fig. 8.3 Poor Finish of Concrete

Fig. 8.4 Excessive Fogging Caused Puddles of Water over Wet Concrete

Fig. 8.5 Water Soaked Curing Blankets in Place

Fig. 8.6 General View of A45 Concrete (West Bound) Bridge Deck

Fig. 8.7 Concrete Placement and Consolidation

Fig 8.8 Finishing of Concrete

Fig. 8.9 Arrangement for Placing the Curing Blanket in Three Widths

Fig. 8.10 Placement of Concrete with a Conveyor

Fig. 8.11 Consistency of Concrete at the Back of the Truck

Fig. 8.12 Concrete Consolidation with Vibrators Mounted on a Cross Beam

Fig. 8.13 Concrete Finishing with Two Rollers

Fig. 8.14 Finish Achieved

Fig. 8.15 Fogging Arrangement

Fig. 8.16 Puddles of Water from Excessive Fogging in Some Patches

Fig. 8.17 Curing Blankets in Place and Being Water Soaked

PART B: BRULE COUNTY BRIDGES

8.4 Construction Monitoring and Constructability Issues of Brule County Bridges

A pair of newly constructed prestressed girder bridges, near Kimball, SD was selected for the performance evaluation of LC-HPC bridge decks. The SDDOT LC-HPC deck was placed on the East bound bridge, Structure Number 08-230-131. The control deck of SDDOT A45 with conventional concrete was placed on the West bound bridge, Structure Number 08-230-130.

The concrete mixtures for SDDOT A45 and LC-HPC bridge decks in Brule County are listed in Table 8.1. Both concrete mixes had the same w/c ratio but the paste content for SDDOT A45 was 25.4% while that for the LC-HPC was 21% by volume. The SDDOT A45 mix utilized fly ash, and LC-HPC utilized optimized aggregate. The A45 LC-HPC also had higher design air content.

Table 8.1 Mix Design of Standard SDDOT A45 and LC-HPC

Mix	Standard A45	LC-HPC
w/c	0.43	0.43
Cement (lb/yd^3)	570	563
Fly Ash (lb/yd^3)	125	-
Coarse Aggregate (lb/yd^3)	1710	1375
Medium Aggregate (lb/yd^3)	-	457
Fine Aggregate (lb/yd^3)	1155	1204
Water (lb/yd^3)	283	242
Paste Volume (%)	25.4	21
Air Content (%)	6.5	8.0

Construction observations were made by the research team and SDDOT personnel during the placement of A45 and LC-HPC bridge decks and two LC-HPC test slabs. The observations are summarized in this section.

8.5 Brule County A45 Bridge Deck Pour (West Bound)

The A45 bridge deck in Brule County was cast in two separate pours (July 26, 2006 and August 14, 2006). The first pour was discontinued by SDDOT engineers due to deficient concrete placement and finishing techniques. In both pours, a concrete pump was used to place the concrete and two individual electric-powered vibrators were used to consolidate the concrete. Fogging was performed manually by one of the construction workers and wet curing blankets were placed on top of the freshly finished concrete.

<u>First Concrete Pour</u>

The first concrete pour was performed on July 26, 2006. The concrete pour started at 1:05am and was stopped at 3:20am. The ambient temperature during the entire concrete placement was approximately 72°F. The relative humidity record was not provided to the research team.

The concrete finishing machine bridge was placed along the bridge deck skew. The axis of the rotating finishing drums was set parallel to the longitudinal axis of the bridge deck, but the augur was skewed relative to the drums. Although the finishing machine had two rotating drums, only one drum was used to finish the concrete surface. Figure 8.18 shows the rotating drums with the skewed augur.

Fig. 8.18 Rotating Drums and Skewed Augur on the Finishing Machine

During the placement of the deck concrete, the following deficiencies were identified:

- The progress was slow. The rate of placement was approximately 20 feet of deck length per hour.
- The finishing plate behind the rotating drums was creating a ridge of paste (fines) as it moved across the bridge deck. This required hand finishing in many locations.
- The rotating drums were slightly above the concrete at the deck crown. This also required hand finishing at some locations.
- With the skewed augur set up, a strip of approximately 8' wide next to the deck edge could not be finished with the rotating drums. The entire width of that strip had to be finished manually.

As a result of those deficiencies, the SDDOT engineer requested that the concrete placement be stopped and be resumed at a later date when the deficiencies were addressed and corrected. The pour was terminated at the face of the first diaphragm location. This location corresponded to approximately 50' of deck length. The finished segment of the deck was then covered with a wet curing blanket. The cold joint is shown in Figure 8.19.

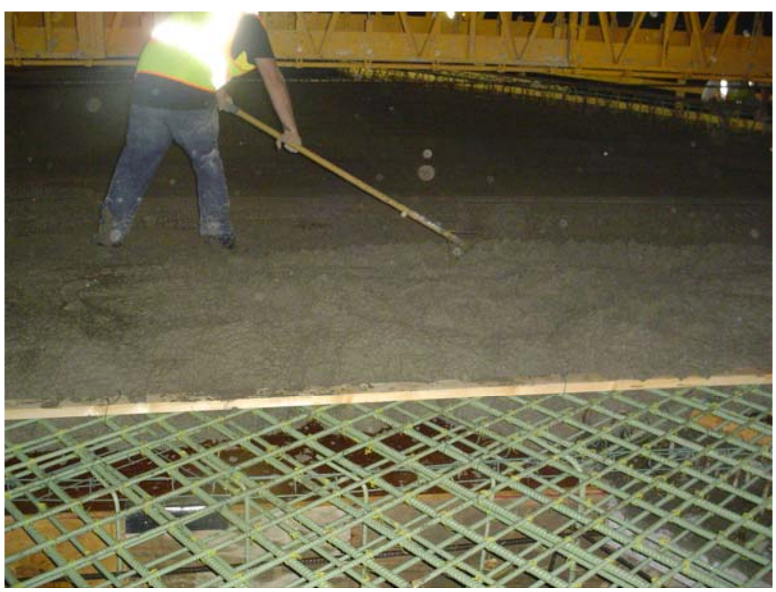


Fig. 8.19 Cold Joint in the A45 Concrete Deck in Brule County

Second Concrete Pour

The second concrete pour was performed on August 8, 2006. Two major modifications were made to the finishing machine. The augur was set in line with the rotating drums and both drums, instead of one, were used to finish the concrete surface. Figure 8.20 shows the augur arrangement for the second pour.

Fig. 8.20 Augur Set Up for the Second Concrete Pour

The second pour commenced at 10:30pm and was competed by 4:35am of the next day. During concrete placement, the air temperature varied between 76°F and 60°F. No record of the relative humidity was furnished to the researchers. The start of the second pour at the cold joint is shown in Figure 8.21.

Fig. 8.21 Start of the second Pour

The second pour went relatively smoothly and was free of the deficiencies experienced in the first pour. However, the finishing machine was advancing at a relatively low speed. The slow finishing process required the concrete placement at one point to be stopped in order for the finishing machine to catch up with the concrete placement. Wet curing blankets were placed on top of the finished concrete surface.

8.6 Test Slabs

The placement of one test slab was required to assess the ability and preparedness of the construction crew to place the LC-HPC bridge deck concrete in accordance to the special provisions. However, construction observations were made on two test slabs because it was determined after the placement of the first test slab that significant modifications to the construction procedures were needed to assure proper placement and finishing of LC-HPC.

A feedback response form was created to evaluate the construction process. The intent of the feedback response form was to provide a tool for the SDDOT and contractor to improve the process of placing LC-HPC. Representatives from SDSU, SDSM&T, and the SDDOT provided feedback. Formal questions were answered and general comments were made on the feedback response form. Once a satisfactory test slab had been performed, the contractor was able to pour the bridge deck which was also observed in the same manner as the test slabs. The construction observations for each event have been summarized below.

8.6.1 Brule County Test Slab I

The first test slab using the special provisions of the LC-HPC for the I-90 East bound bridge near Kimball, SD was placed on May 17, 2007. The 30 foot long test slab was constructed on site, in the right of way of the road passing underneath the bridge. The LC-HPC mix was to be discharged directly from the ready-mix truck. The vibrator set up, finishing roller, and curing materials were mounted on finishing bridges. The finishing bridges were steel trusses that spanned across the bridge. The finishing bridges allowed the workers to finish the concrete from above the concrete deck. The vibrator set up and finish roller were on the first finishing bridge followed by a second finishing bridge with the curing materials and foggers. The test slab was constructed with a skew angle to represent the skew in the actual bridge. However, the finishing bridges were placed normal to the longitudinal axis of the test slab. Thus, the finishing bridges were not aligned along the skew.

SDSU and SDSM&T representatives observed the construction process and filled out a feedback response form to evaluate the feasibility of the construction of the LC-HPC test slab.

Weather Conditions:

The air temperature was in the 58°F and 66°F under clear skies, the wind was blowing at approximately 25 miles per hour, and the relative humidity was about 40%. The weather conditions and evaporation rates are given in Table 8.2.

Table 8.2 Weather Conditions for Test Slab I

Project:	LC – HPC Test Slab I Kimball
Date:	May 17th, 2007

Time	**Concrete Temp. Tc (°F)**	**Air Temp Ta (°F)**	**Relative Humidity, r**	**Wind velocity V (mph)**	**Rate of Evaporation E (lb/ft2/hr)**
10 a.m.	73.0	57.9	0.47	19.6	0.297
11 a.m.	73.0	61.0	0.41	20.7	0.312
12 p.m.	73.0	63.0	0.41	25.3	0.363
1 p.m.	73.0	66.0	0.42	23.0	0.313

Weather Source: http://www.wunderground.com

Concrete Quality: As expected, the LC-HPC mix was stiff due low w/cm and low paste amount. The measured air content ranged from 5 to 7% and was either on the low end of the acceptable range or below the acceptable limit. It was noted that the air content needed to be increased to allow for loss of air when the concrete is pumped or conveyed on a belt.

Slump was sometimes below the acceptable limit of 1.5” and ranged from 1.25 to 2.875”.

The deposited concrete was drying up fast due to the unfavorable weather conditions that led to high evaporation rate.

Concrete Placement:

The concrete was deposited directly from the concrete trucks. The concrete placing crew had difficulty placing the correct depth of unconsolidated concrete. As a result there was a lot of manual movement of the concrete material that had already been placed. The rate of placement was too slow and the unconsolidated concrete was exposed to the wind for an extended time. This increased the evaporation of water from the mix. The concrete placement started at 10:07 a.m. but the finishing machine did not start until 10:55 a.m, some 38 minutes past the maximum delay time allowed by the LC-HPC specifications. The slow rate of placement and finishing caused extended delays before the loaded concrete trucks were able to discharge their concrete loads.

Concrete Consolidation:

The vibrators set up seemed to be inadequate. There was a significant delay of approximately 10 minutes between placement and vibration of concrete. In some cases, the deposited concrete seemed to have started to set before it was vibrated. The vibrators set up also did not allow for vibration of the concrete placed next to the slab edge. This necessitated the use of single vibrators that were not attached to the vibrators set up.

Concrete Finish:

There was a concern about the speed at which the finishing machine was operated. In some cases, the drums were rotating at a very low speed. The transverse speed of the drums were low initially, but was later increased. The finishing improved with the faster drum speed. The drum head was fitted with three fogging nozzles to allow for finishing. Use of hand trowels was excessive at the edges of the slab. Excessive cracking appeared at the surface due to high evaporation rate. The finish surface had many areas of exposed aggregates, either due to lack of paste, improper drum speed, or excessively dry mix. The finish obtained on the test slab can be seen in Figure 8.22.

Fig. 8.22 Test Slab I Finished Surface

Concrete Protection and Curing:

Since the finishing bridges were not skewed, it took too long to start the fogging equipment after the start of concrete placement in order to avoid soaking the concrete surface on one side of the slab. The time delay between finishing and fogging was approximately 90 minutes. The fogging nozzles dripped water onto the concrete surface and the water was puddling on the surface of the slab.

- Placement of curing blanket was about 2 hours after strike-off and excessive water was being placed on the blanket.

The curing blankets set up and fogging/finishing can be seen in Figure 8.23. The excessive water can be seen on the concrete surface. It can also be seen that the blankets did not reach the edge of the test slab.

Fig. 8.23 Curing Blankets and Concrete Finish for Test Slab I

Conclusions:

It was evident the contractor was not capable of placing the LC-HPC bridge deck in accordance with the special provisions for LC-HPC. A second test slab with skewed finishing bridges arrangement was recommended for construction process qualification.

8.6.2 Brule County Test Slab II

The second test slab for the I-90 East bound bridge near Kimball, SD was placed on June 15, 2007. The slab was constructed on site, in the right of way of the road passing underneath the bridge. SDSU and SDSM&T representatives observed construction and filled out a feedback response form to evaluate the feasibility of the construction of low-cracking bridge decks. The weather conditions and evaporation rates are given in Table 8.3. The LC-HPC was discharged using a conveyor belt. Ice was added to trucks after the first two truckloads had excessive concrete temperatures. Both finishing bridges were skewed at the angle of the bridge deck for test slab II.

Weather Conditions:

During concrete placement and finishing the air temperature varied between 66°F and 75°F, the wind varied between approximately 5 and 9 miles per hour, and the relative humidity varied between 73% and 90%. The weather conditions and evaporation rates are given in Table 8.3.

Table 8.3 Weather Conditions for Test Slab II

Project:	LC – HPC Test Slab II Kimball
Date:	June 15th, 2007

Time	**Concrete Temp, Tc (°F)**	**Air Temp, Ta (°F)**	**Relative Humidity, r**	**Wind velocity, V (mph)**	**Rate of Evaporation, E (lb/ft2/hr)**
7 a.m.	75.0	66.0	0.9	4.6	0.048
8	75.0	69.1	0.87	9.2	0.066
9	75.0	72.0	0.78	9.2	0.067
10	75.0	75.0	0.73	8.1	0.056

Weather Source: http://www.wunderground.com/

Concrete Quality:

The LC-HPC mix used for the second test slab was easier to place and seemed to possess better workability properties than the LC-HPC mix used for the first test slab. The favorable weather conditions during the placement of the second test slab may have helped in attaining prolonged workability.

The researchers were unable to obtain the measured fresh concrete properties from SDDOT. However, a spot check on one of the delivered concrete batches showed a slump of 1.5", an air content of 6.0%, a concrete unit weight of 144.5 lb/ft^3, and a mix temperature of 79°F.

Concrete Placement:

The concrete was placed using a conveyor belt. The conveyor allowed for the placement of concrete where needed with relative ease. However, the concrete material was allowed to leave the conveyor belt based on its own momentum, resulting in some segregation as the concrete discharged from the conveyor. The heavier particles traveled the farthest and the finer materials tended to fall at the conveyer end.

The concrete temperature in the first two batches was approximately 78°F. Ice was added to the concrete in the subsequent loads to reduce the concrete temperature to below 75°F.

The concrete placement progress was slow due to the long time gap between the arrivals of successive concrete trucks. The rate of placement was about 20 ft of slab length per hour.

Concrete Consolidation:

Although the vibrator set up was mechanized, the operator moved slowly across the finishing bridge. This slowed down of the entire consolidation and finishing process. It was also observed that the vibrator insertion depth was inconsistent. With proper training and instruction, the operator should have been able to run the consolidation process more smoothly.

Concrete Finish:

The spray nozzles were emitting water spray rather than mist or fog. This caused some water accumulation at the concrete surface. The problem could have been resolved by controlling the pressure of the fogging system.

The drum speed on the finishing machine was slow moving across the slab and the speed was somewhat inconsistent. This led to inconsistency in the finished surface.

Concrete Protection and Curing:

Fogging started approximately 40 minutes after strike-off whereas the specifications call for 10 minutes. The curing blankets were not pulled all the way to the slab sides leaving some of the concrete surface exposed. The nozzles used to moisten the blankets were bigger than needed and were left on while the bridge was not moving causing water to puddle.

After observing the construction of the second test slab the following recommendations were made.

- The consolidation process could be expedited by increasing the pace of the vibrators.
- The finish drum lateral speed should be increased.
- The curing materials need to be placed within 10 minutes of final strike-off.
- A worker needs to be in charge of the foggers to prevent problems due to leaks, spills, or excess fogging; manual moistening of the curing materials may yield better results.
- The concrete needs to be delivered at a rate that would not hinder placement.
- The second test slab went very well due to the favorable weather conditions but the contractor should not be overly confident; there was still room for improvement.

8.7 Brule County LC-HPC Bridge Deck Pour (East Bound)

The LC-HPC bridge deck was poured on June 27, 2007. The LC-HPC mix was placed using a conveyor belt system. As the LC-HPC was placed in the deck, workers moved the fresh concrete into a consistent depth for the finishing machine. The finish roller was mounted on the first finish bridge. The deck surface was being "fogged" with water as it was being finished. A row of foggers were mounted on the back of the first bridge. These foggers kept the finished concrete moist until the second finish bridge could place the curing materials. Presoaked curing blankets were unrolled on the bridge deck. The curing blankets were kept moist by foggers on the back of the second finish bridge. The curing blankets were pulled tight to the formwork so the curing process could begin.

SDSU and SDSM&T representatives along with SD-DOT engineers observed the construction process and filled out a feedback response form to evaluate the feasibility of the construction of low-cracking bridge decks. The bridge deck placement started at 4 a.m. The air temperature was in the 50's, the wind was 5 miles per hour, and the relative humidity was nearly 80%. The weather and evaporation rates are given in Table 8.4. A summary of the bridge deck pour feedback response form is presented below.

Table 8.4 Weather Conditions for Bridge Deck

Project:	LC – HPC Bridge Deck Pour – Kimball
Date:	June 27th, 2007

Time	Concrete Temp, Tc (°F)	Air Temp, Ta (°F)	Relative Humidity, r	Wind velocity, V (mph)	Rate of Evaporation, E (lb/ft2/hr)
4 a.m.	60.0	59.0	0.78	6.0	0.024
5 a.m.	62.0	54.0	0.83	6.0	0.042
6 a.m.	63.0	54.0	0.81	4.0	0.037
7 a.m.	64.0	58.0	0.72	8.0	0.060
8 a.m.	67.0	64.0	0.68	0.0	0.014
9 a.m.	68.0	69.0	0.57	9.0	0.072
10 a.m.	72.0	72.0	0.49	9.0	0.103

Weather Source: http://www.wunderground.com/

Weather Conditions:

The weather conditions were optimum for placing concrete. The temperature varied between 54°F and 72°F. The wind speed recorded on site was between 4 to 9 mph. The relative humidity ranged between 49 and 83%.

Concrete Quality:

The concrete mix appeared to possess uniform quality throughout the pour and was workable due to the ideal weather conditions. Slump ranged from 1.25 in. to 3 in., and air content varied from 6.6% to 7.6%.

Concrete Placement:

The concrete was deposited by means of a conveyor belt. Figure 8.24 shows the conveyor belt set up on the bridge deck. At the beginning of the concrete pour, the concrete quantity that was deposited on the deck was in excess of what is required to provide the required deck thickness. Figure 8.25 shows the excess concrete. This caused some progress delay as the construction crew had to move the excess concrete. As the work progressed, the rate of concrete placement improved and reached an average of 35 ft of deck length per hour.

Fig. 8.24 Conveyor Belt Set Up for the LC-HPC Deck in Brule County

Fig. 8.25 Excess Concrete Deposit at the LC-HPC Deck in Brule County

Concrete Consolidation:

Six vibrators mounted on a steel bridge allowed for transverse and longitudinal movement and vertical insertion of the vibrator set all at once. Set up allowed for the vibrator gang to be moved parallel to the skew and advanced 1’ along the bridge length. Due to operator error, the vibrator set up was not always moved 6’ across the bridge and several spots were vibrated twice. It was also observed that the vibrators should have been moved across the deck a little quicker. The concrete gang vibrator set up is shown in Figure 8.26.

Fig. 8.26 Gang Vibrator Set Up for the LC-HPC Deck in Brule County

Concrete Finish:

The favorable weather conditions allowed for reduced effort to finish the concrete. The roller on the finishing machine provided a smooth finished surface with minimal hand finishing. Upon the request of SDDOT engineers, no drag carpet was used behind the roller.

Concrete Protection:

Figure 8.27 shows the fogging system that was used to provide moisture to the finished surface. It was observed that the fogging nozzles provided a fine water spray rather than mist. This resulted in some pooling water on the surface of the deck. The contractor tried to minimize the water pooling by controlling the on/off operation of the fogging system.

Wetting of the formwork was performed promptly and near-ideal weather conditions did not require specific measure for concrete protection during placement.

Fig. 8.27 Concrete Fogger Set Up for the LC-HPC Deck in Brule County

Concrete Curing:

Initially, the curing blanket was placed approximately 35 minutes after strike off. Later, the blanket placement was done within 10 minutes of concrete strike off.

A soaker hose was used to wet the curing blanket after it was placed on the finished concrete surface. However, it was observed that excess water was applied to the curing blanket causing the water to run and collect on the sides of the slab. Figure 8.28 shows the soaked curing blankets being placed on the deck.

Fig. 8.28 Curing Blankets on the LC-HPC Deck in Brule County

The following general conclusions and comments were made after the construction observation of the LC-HPC bridge deck.

- The overall pour went relatively smoothly. The weather conditions were ideal and at times fogging was not necessary.

- Excessive fogging did occur and there were some leaks in the lines.

- The entire process could have been sped up if the depth of concrete placement was more consistent. .

- Curing materials need to be applied sooner and not be soaked to prevent puddling of water.

- LC-HPC placement and finishing is manageable when the weather conditions are favorable and sufficient advance preparations are made. For successful placement, the contractor and concrete supplier must be familiar with the behavior of the fresh mix and the weather must be favorable.

CHAPTER 9 CRACK SURVEYS OF NEW BRIDGES

Task 7. Conduct an extensive crack survey of each of the four bridges prior to its being opened to traffic and annually for three years after opening to traffic. The crack surveys will be conducted according to the protocol developed as part of Pooled Fund TPF-5(051) Construction of Crack-Free Concrete Bridge Decks.

PART A: CRACK SURVEYS OF PENNINGTON COUNTY BRIDGES

Crack surveys were conducted on the top surface of the A45 concrete bridge (west bound) and LC-HPC bridge (east bound) in Pennington County. The Pooled Fund crack survey protocol was used to map the cracks. A copy of the Kansas University specification for crack survey as supplied by SD-DOT is attached in Appendix C.

The following four sets of crack surveys were conducted for the control bridge decks and the LC-HPC bridge decks:

(v) prior to the bridges being open to traffic
(vi) one year after being open to traffic
(vii) two years after being open to traffic and
(viii) three years after being open to traffic

9.1 Crack Densities

The details of the crack survey conducted for the A45 control bridge deck along with the age of the deck at which the crack surveys were performed are shown in the following table (Table 9.1). The table also shows the crack densities of A45 control deck. After about one year of service, the A45 control deck showed a crack density of 0.573 m/m^2 which means the average length of visible cracks from a waist height of person of normal height is 0.573 m of crack length over an area of one square meter of the deck top surface. The crack density was almost zero at about 28 days and increased to a density of 0.573 one year later and finally to 0.73 m/m^2 at the end of three years of service.

Table 9.1 Crack Densities of A45 Control Bridge Deck in Pennington County

A45 Control Deck				
Date of Pour	10/18/2005			
Survey #	CS1	CS3	CS5	CS7
Survey Date	11/14/2005	5/16/2007 and 5/23/2007	11/26/2007	11/13/2008
Age (Months)	0.89	18.90	25.28	36.89
Crack Density (m/m^2)	0.006	0.573	0.643	0.73

Similarly, the details of the crack survey conducted for the LC-HPC bridge deck along with the age of the deck at which the crack surveys were performed are shown in Table 9.2. The table also shows the crack densities for LC-HPC deck. The crack density was zero at about 28 days. Therefore, an additional survey was requested by SD-DOT to be conducted about six months after the casting of the deck. The additional crack survey was performed and the crack density was found to be 0.066 m/m^2 which was reasonably low after close to six months of service. The crack density reached 0.71 m/m^2 one year after the additional crack survey and gradually increased to 0.85 m/m^2 at the end of three years of service.

Table 9.2 Crack Densities of LC-HPC Bridge Deck in Pennington County

LCHPC Deck					
Date of Pour	10/29/2005				
Survey #	CS2	CS22	CS4	CS6	CS8
Survey Date	11/18/2005	6/6/2006	5/16/2007 and 5/23/2007	11/26/2007	11/14/2008
Age (Months)	0.66	7.23	18.54	24.92	36.56
Crack Density (m/m^2)	0	0.066	0.71	0.747	0.85

9.2 Comparison of Crack Densities of A45 and LC-HPC Bridge Decks

The crack densities determined for the two bridge decks (A45 and LC-HPC) that are located in Pennington County are compared in Fig. 9.1. The two curves shown the figure demonstrate that the control deck using A45 concrete has a smaller crack density than that of LC-HPC bridge deck.

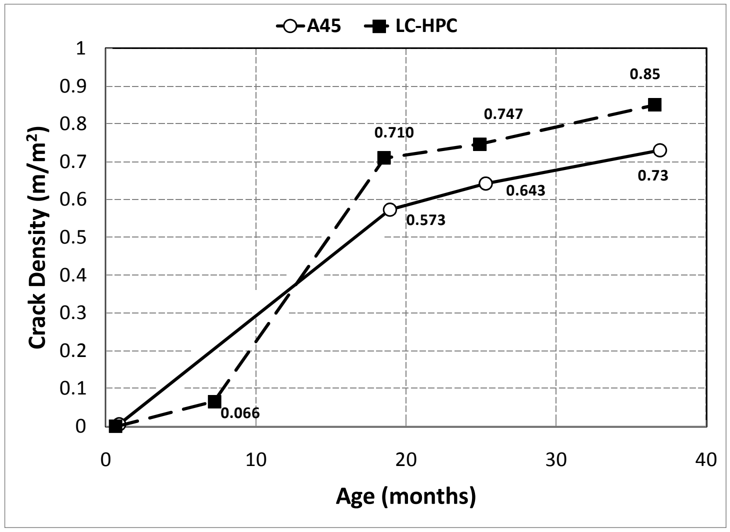


Fig. 9.1 Comparison of Crack Densities of A45 Bridge Deck and LC-HPC Bridge Deck

9.3 Crack Maps

The general layout of the two bridge decks is shown in Fig. 9.6. The first set of crack surveys was conducted in mostly very cold weather conditions (Figs. 9.2 and 9.3). The average atmospheric temperature and average wind speeds leading up to the day of crack survey in the Rapid City region for the first set of crack surveys are shown Figs. 9.4 and 9.5 in Appendix C. The weather data curves show that the average temperatures in the region were very low. It was generally observed that the temperatures were below freezing point most nights and part of the days during the period leading to crack surveys.

The first two crack survey maps are shown in Figs. 9.7 and 9.8. The crack lengths detected on the top surface of A45 bridge deck were minimal. The crack survey protocol does not specifically require crack widths to be recorded. However, crack width measurements revealed that the crack widths were generally less than 0.007 inch for first two crack surveys. No cracks were detected at all on the LC-HPC bridge deck.

Subsequent seven crack maps are plotted and shown in Figs. 9.9 to 9.15.

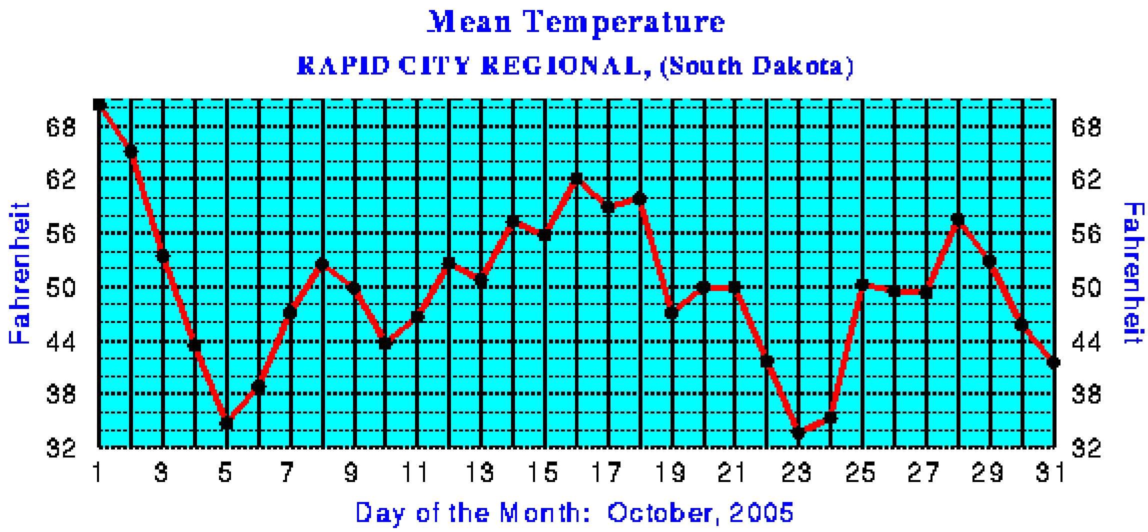


Fig. 9.2 Mean Temperature in October 2005

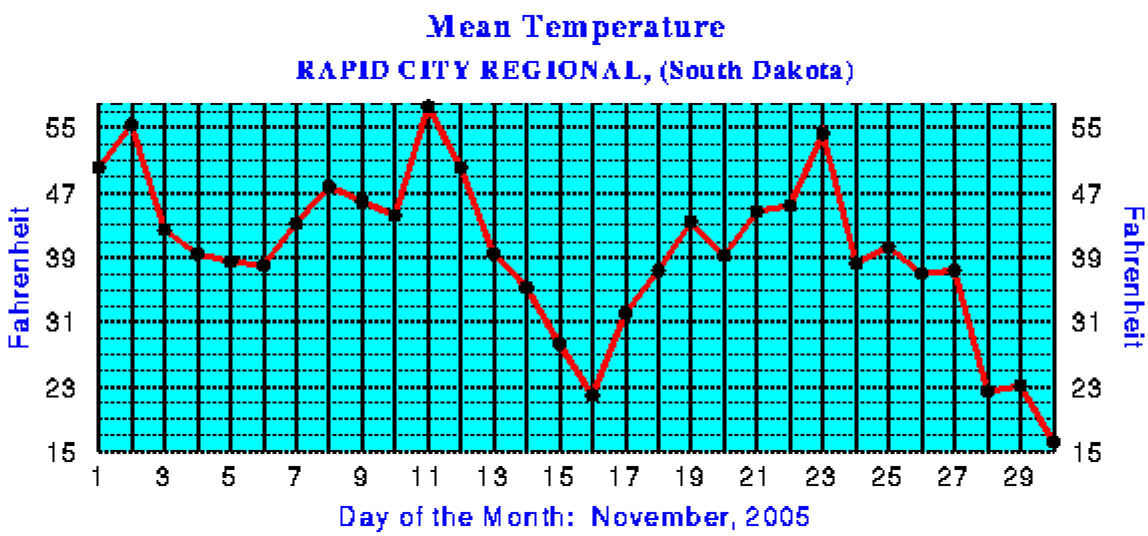


Fig. 9.3 Mean Temperature in November 2005

Fig. 9.4 Crack Survey of A45 Concrete Bridge Deck (West Bound)

Fig. 9.5 Crack Survey of LC-HPC Bridge Deck (East Bound)

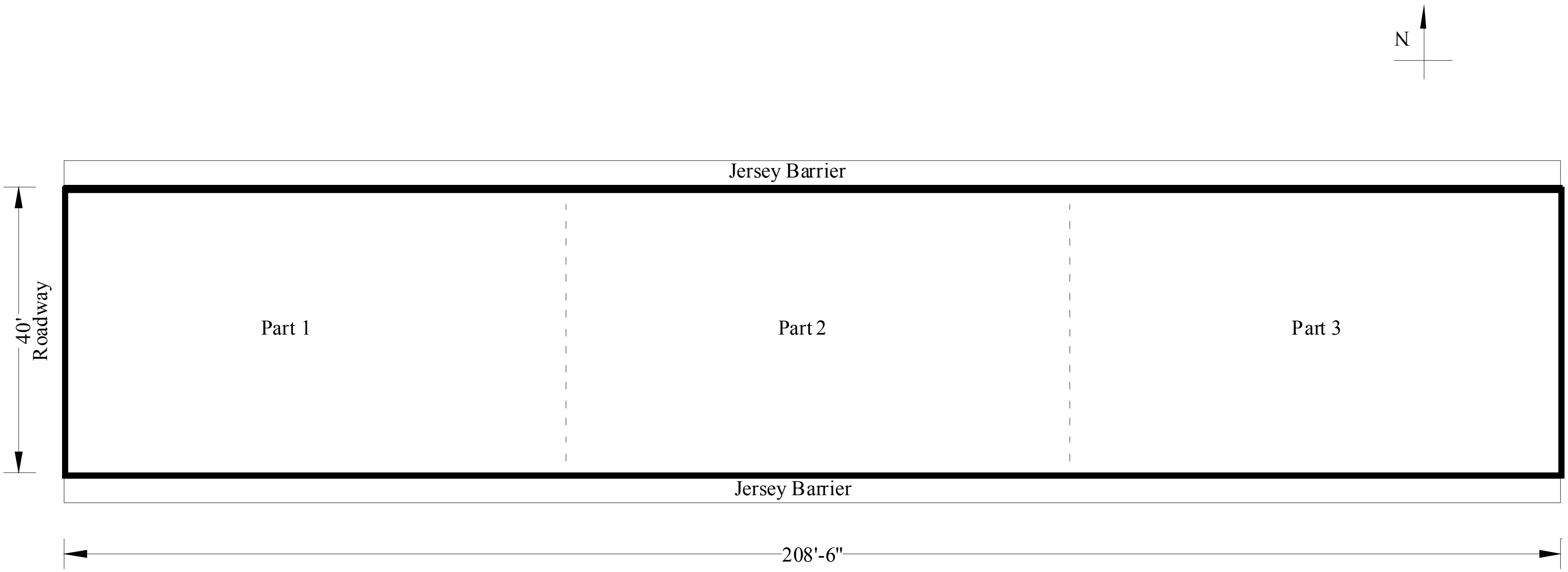
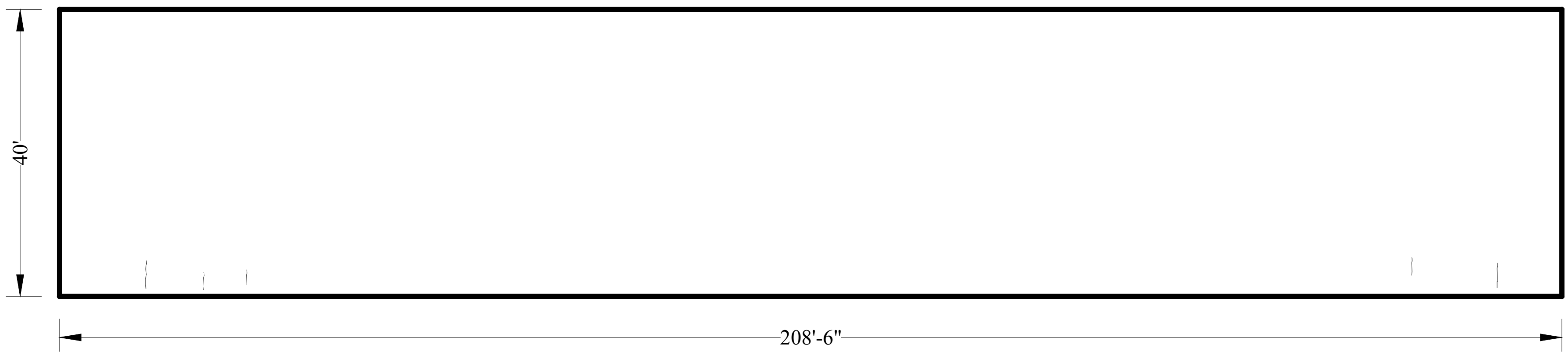


Fig. 9.6 Bridge Deck Dimensions

(Both A45 and LC-HPC Bridges)



Crack Survey # CS1: A45 Concrete Deck
(Prior to Opening to Traffic)

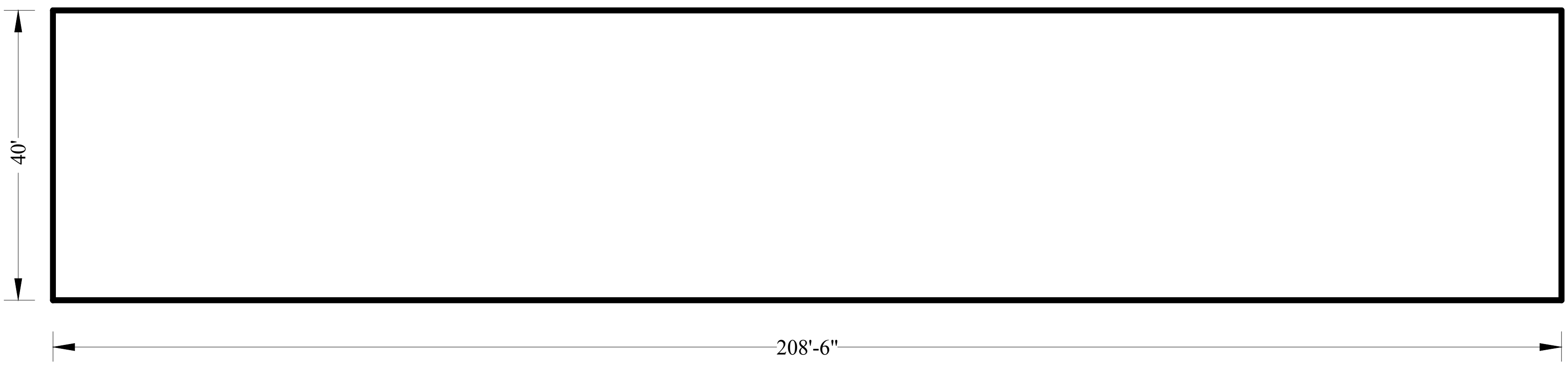
Crack Density = 0.006 m/m^2

Date of Pour: 10/18/2005
Date of Crack Survey: 11/14/2005
Age: 0.9 Months
Time: 15:30 to 16:15
Temperature: 17° F

′-6″ ′-0″ Roadway Composite Steel Girder Bridge with 0° Skew
West Bound Over 16B (A45 Concrete)
Str. No.: 52-435-288 Pennington County

Research Project # SD 2005-11
Evaluation of Crack-Free Bridge Decks

Fig. 9.7 Crack Map CS1 (A45)



Crack Survey # CS2: LC-HPC Concrete Deck
(Prior to Opening to Traffic)

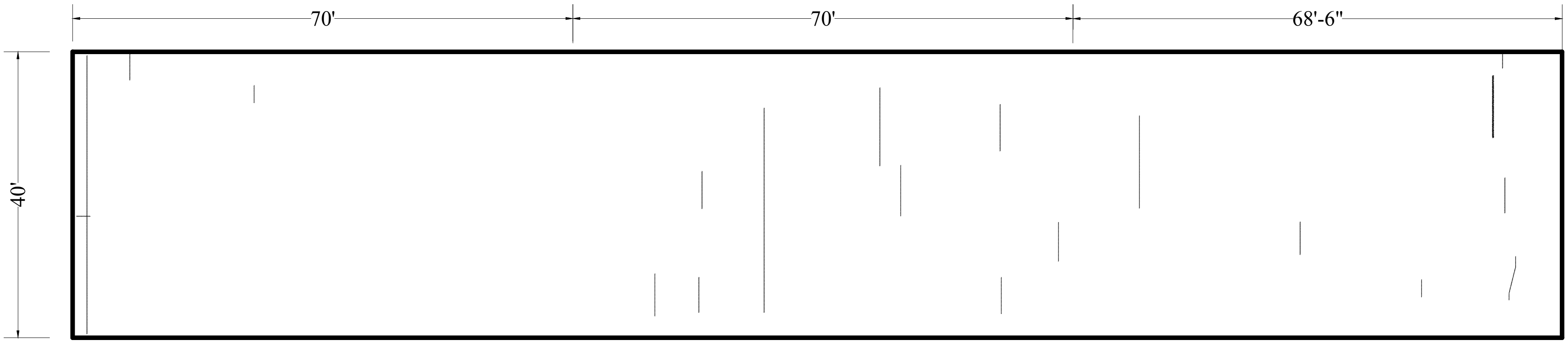
Crack Density = 0.00 m/m^2
(No Cracks were Found)

Date of Pour: 10/29/2005
Date of Crack Survey: 11/18/2005
Age: 0.66 Months
Time: 14:30 to 15:30
Temperature: 58° F

′-6″ ′-0″ Roadway
Composite Steel Girder Bridge with 0° Skew
East Bound Over 16B (LC-HPC Concrete)
Str. No.: 52-435-289 Pennington County

Research Project # SD 2005-11
Evaluation of Crack-Free Bridge Decks

Fig. 9.8 Crack Map CS2 (LC-HPC)



Crack Survey # CS22: LC-HPC Concrete Deck
(Prior to Opening to Traffic)

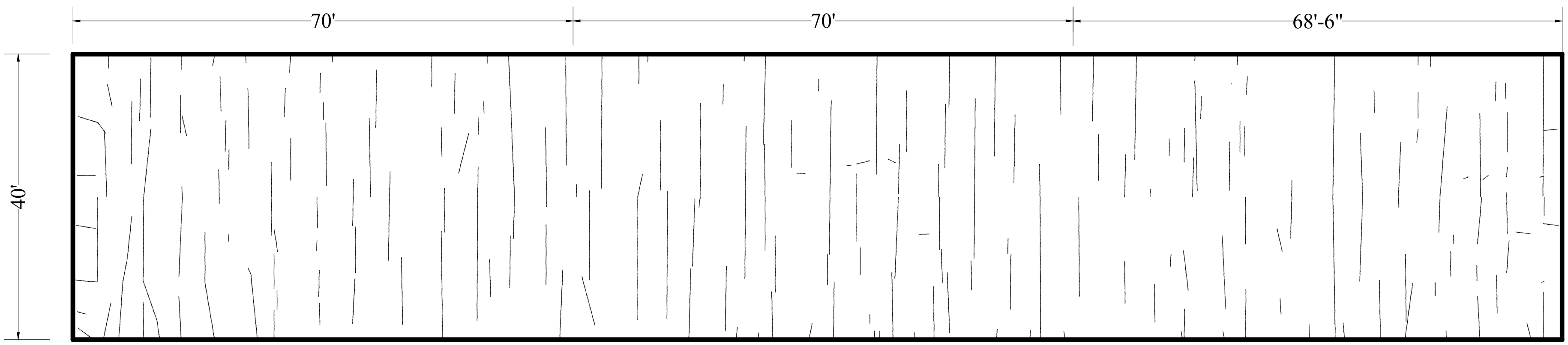
Crack Density = 0.066 m/m^2

Date of Pour: 10/29/2005
Date of Crack Survey: 06/06/2006
Age: 7.23 Months
Time: 6:30 to 8:30 AM
Temperature: 70° F

′-6″
′-0″ Composite Steel Girder Bridge
Roadway with 0° Skew
East Bound Over 16B (LC-HPC Concrete)
Str. No.: 52-435-289 Pennington County

Research Project # SD 2005-11
Evaluation of Crack-Free Bridge Decks

Fig. 9.9 Crack Map CS22 (LC-HPC)



Crack Survey # CS3: A45 Concrete Deck

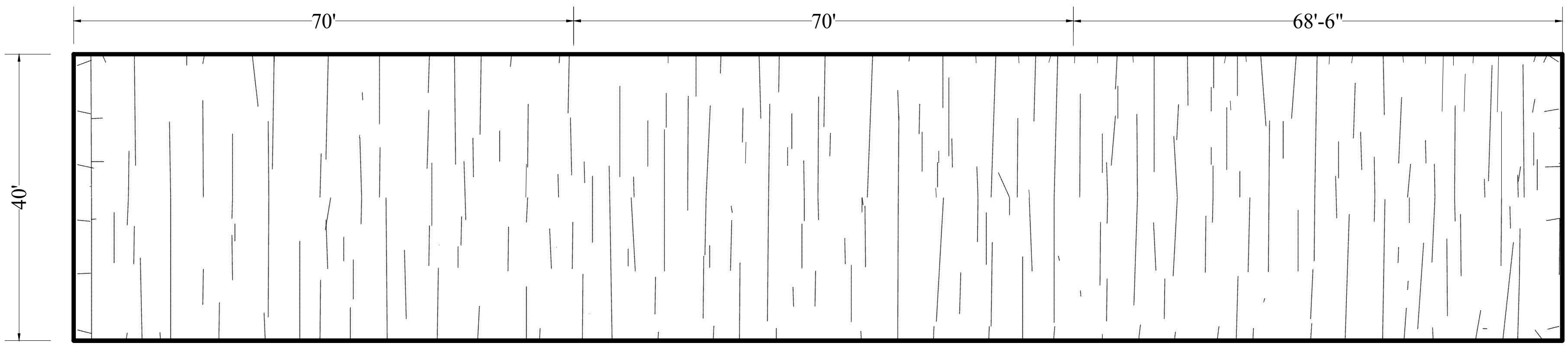
Crack Density = 0.573 m/m^2

Date of Pour: 10/18/2005
Date of Crack Survey: 05/16/2007 and 05/23/2007
Age: 18.9 Months
Time: 8:00 to 12:30 PM
Temperature: 48° F

′-6″ Composite Steel Girder Bridge
′-0″ Roadway with 0° Skew
West Bound Over I6B (A45 Concrete)
Str. No.: 52-435-288 Pennington County

Research Project # SD 2005-11
Evaluation of Crack-Free Bridge Decks

Fig. 9.10 Crack Map CS3 (A45)



Crack Survey # CS4: LC-HPC Concrete Deck

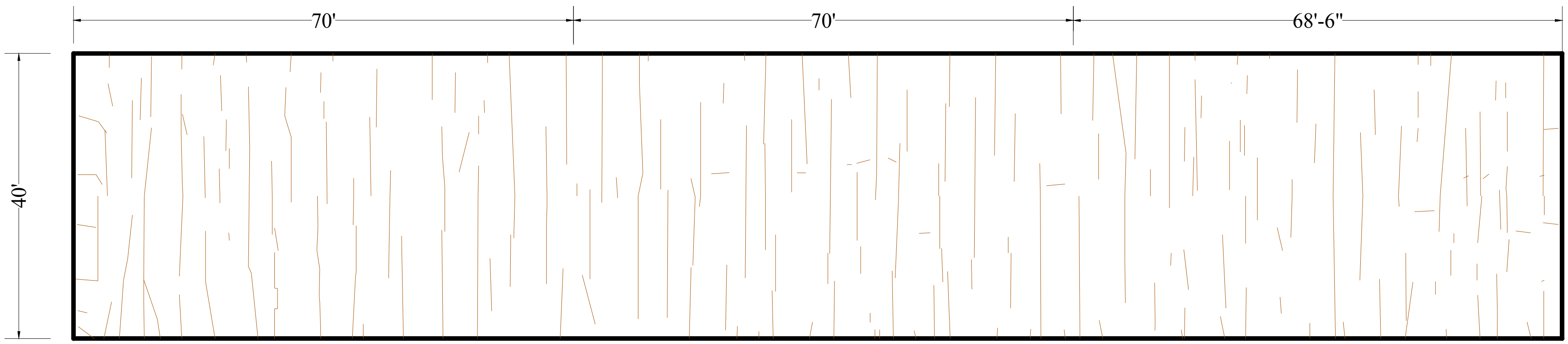
Crack Density = 0.710 m/m^2

Date of Pour: 10/29/2005
Date of Crack Survey: 05/16/2007 and 05/23/2007
Age: 18.54 Months
Time: 13:00 to 16:00
Temperature: 52° F

′-6″ Composite Steel Girder Bridge
′-0″ Roadway with 0° Skew
East Bound Over 16B (LC-HPC Concrete)
Str. No.: 52-435-289 Pennington County

Research Project # SD 2005-11
Evaluation of Crack-Free Bridge Decks

Fig. 9.11 Crack Map CS4 (LC-HPC)



Crack Survey # CS5: A45 Concrete Deck

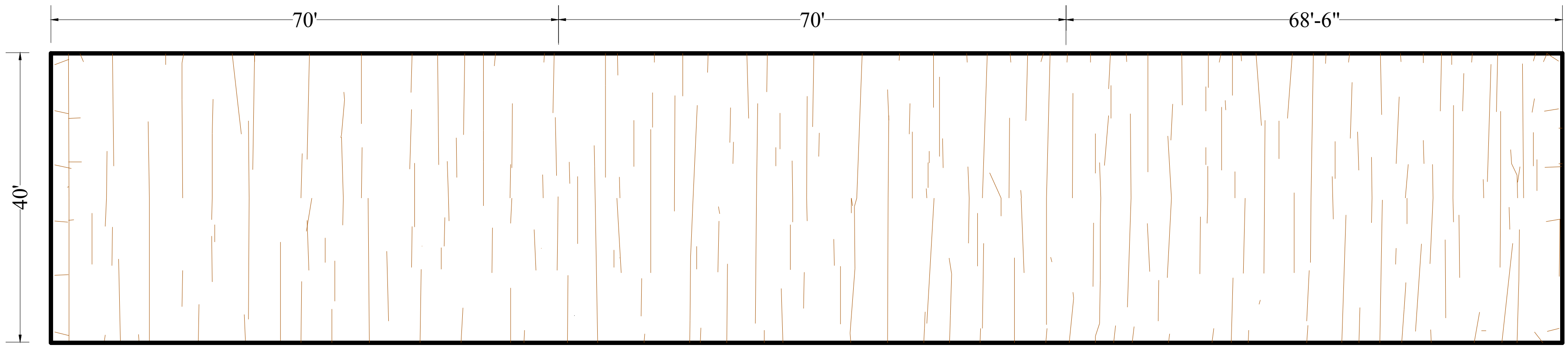
Crack Density = 0.643 m/m^2

Date of Pour: 10/18/2005
Date of Crack Survey: 11/26/2007
Age: 25.3 Months
Time: 8:00 AM to 12:30 PM
Temperature: 38° F

′-6″ Composite Steel Girder Bridge
′-0″ Roadway with 0° Skew
West Bound Over 16B (A45 Concrete)
Str. No.: 52-435-288 Pennington County

Research Project # SD 2005-11
Evaluation of Crack-Free Bridge Decks

Fig. 9.12 Crack Map CS5 (A45)



Crack Survey # CS6: LC-HPC Concrete Deck

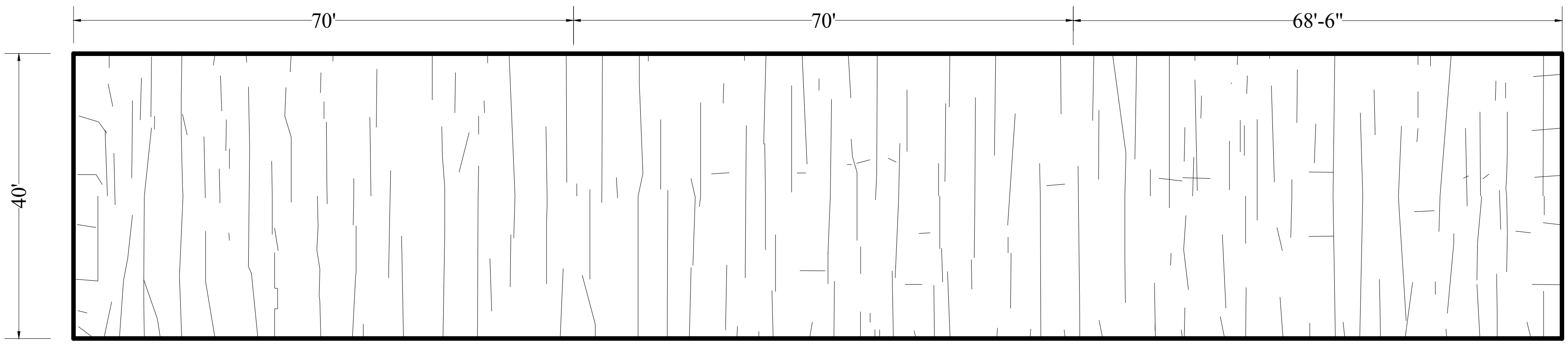
Crack Density = 0.747 m/m^2

Date of Pour: 10/29/2005
Date of Crack Survey: 11/26/2007
Age: 24.9 Months
Time: 13:00 to 16:00
Temperature: 40° F

′-6″ Composite Steel Girder Bridge
′-0″ Roadway with 0° Skew
East Bound Over 16B (LC-HPC Concrete)
Str. No.: 52-435-289 Pennington County

Research Project # SD 2005-11
Evaluation of Crack-Free Bridge Decks

Fig. 9.13 Crack Map CS6 (LC-HPC)



Crack Survey # CS7: A45 Concrete Deck

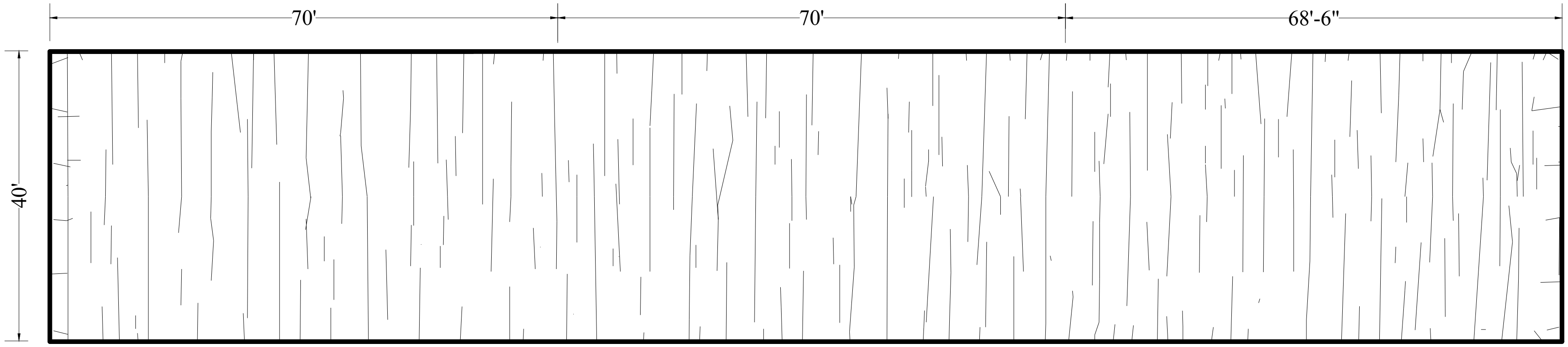
Crack Density = 0.73 m/m^2

Date of Pour: 10/18/2005
Date of Crack Survey: 11/13/2008
Age: 36.9 Months
Time: 12:15 PM to 4:00 PM
Temperature: 32° F
Very windy, with a few snowflakes

′-6″ Composite Steel Girder Bridge
′-0″ with 0° Skew
Roadway
West Bound Over 16B (A45 Concrete)
Str. No.: 52-435-288 Pennington County

Research Project # SD 2005-11
Evaluation of Crack-Free Bridge Decks

Fig. 9.14 Crack Map CS7 (A45)



Crack Survey # CS8: LC-HPC Concrete Deck

Crack Density = 0.85 m/m^2

Date of Pour: 10/29/2005
Date of Crack Survey: 11/14/2008
Age: 24.9 Months
Time: 8:00 Am to 12:00 PM
Temperature: 29° F
Very windy, with a few snowflakes

′-6″ Composite Steel Girder Bridge
′-0″ Roadway with 0° Skew
East Bound Over 16B (LC-HPC Concrete)
Str. No.: 52-435-289 Pennington County

Research Project # SD 2005-11
Evaluation of Crack-Free Bridge Decks

Fig. 9.15 Crack Map CS8 (LC-HPC)

208
40

PART B: CRACK SURVEYS OF BRULE COUNTY BRIDGES

To monitor the development of deck cracks with time, crack surveys of the control deck and LC-HPC deck in Brule County were performed at the end of the curing period and annually thereafter for three years. The crack surveys were used as a tool to directly evaluate the performance of the bridge decks.

Once the field surveys of the bridge decks were complete and transferred to electronic files, they were sent to The University of Kansas for analysis. Using a computer program, the crack densities were determined for each bridge. The crack densities are expressed as length of cracks per area (m/m^2). The crack densities, crack survey date, and bridge deck age are displayed in the figures.

9.4 West Bound Control Deck

The bridge for the West bound lane of I-90 was the control bridge with a deck of SDDOT standard A45 concrete. Problems arose during placement and a cold joint was used to continue placement at a later date. The West 1/3 of the bridge was placed first, and the East 2/3 was placed later. These are referred to as Placement 1 and Placement 2 in the figures. After placement and curing, the deck was surveyed at 30 days. The results of the crack survey are shown in Figure 9.16.

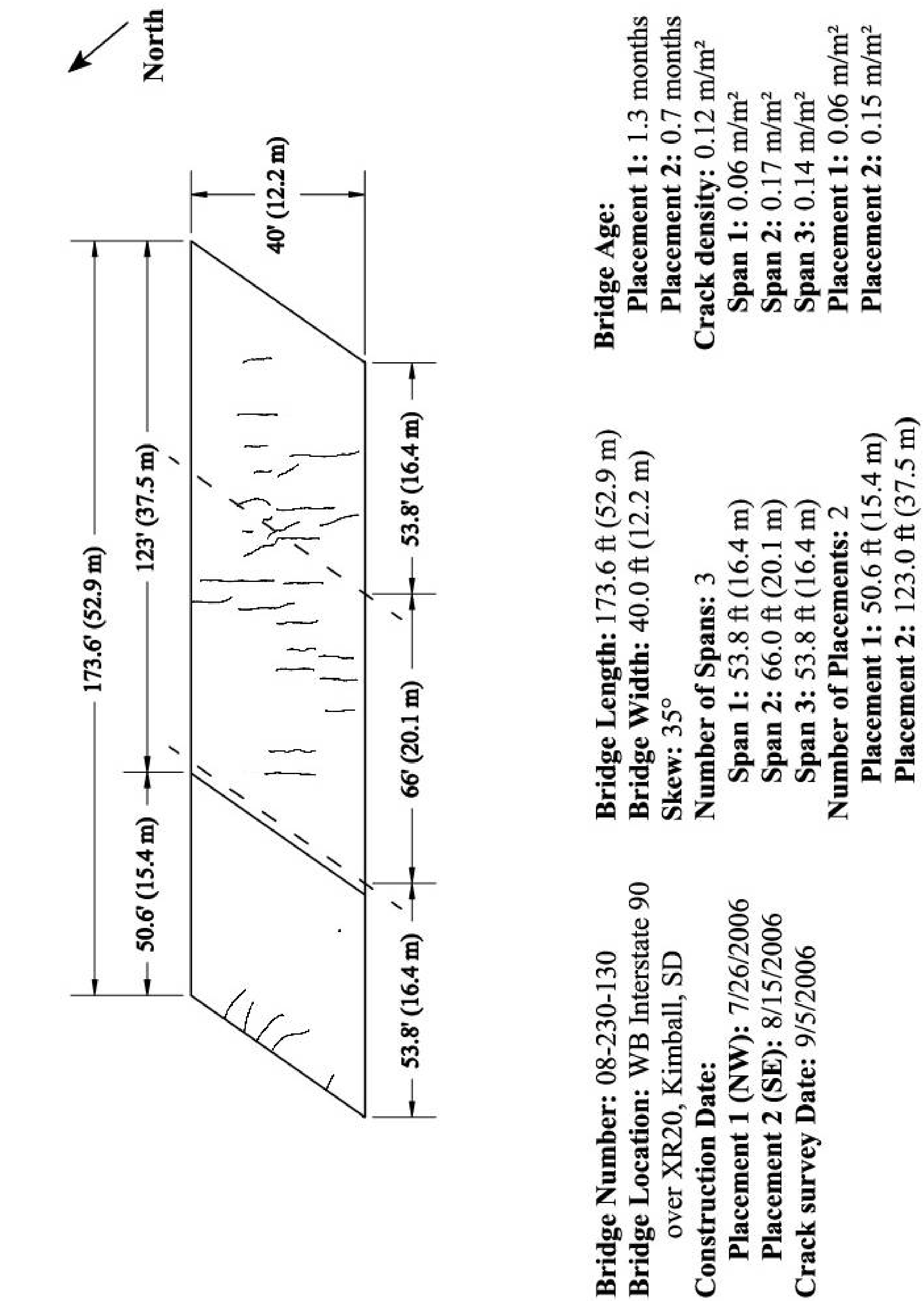


Fig. 9.16 Control Deck-30 Day Crack Survey

The first placement of concrete experienced less cracking (0.06 m/m^2) than the second placement (0.15 m/m^2) after 30 days. No cracks occurred around the cold joint and cracks propagated approximately perpendicular to the bridge deck edge on span 1. The cracks that occurred in spans 2 and 3 propagated across the width of the bridge.

The one year survey was performed on 9/27/2007. The results of the one year survey can be seen in Fig. 9.17.

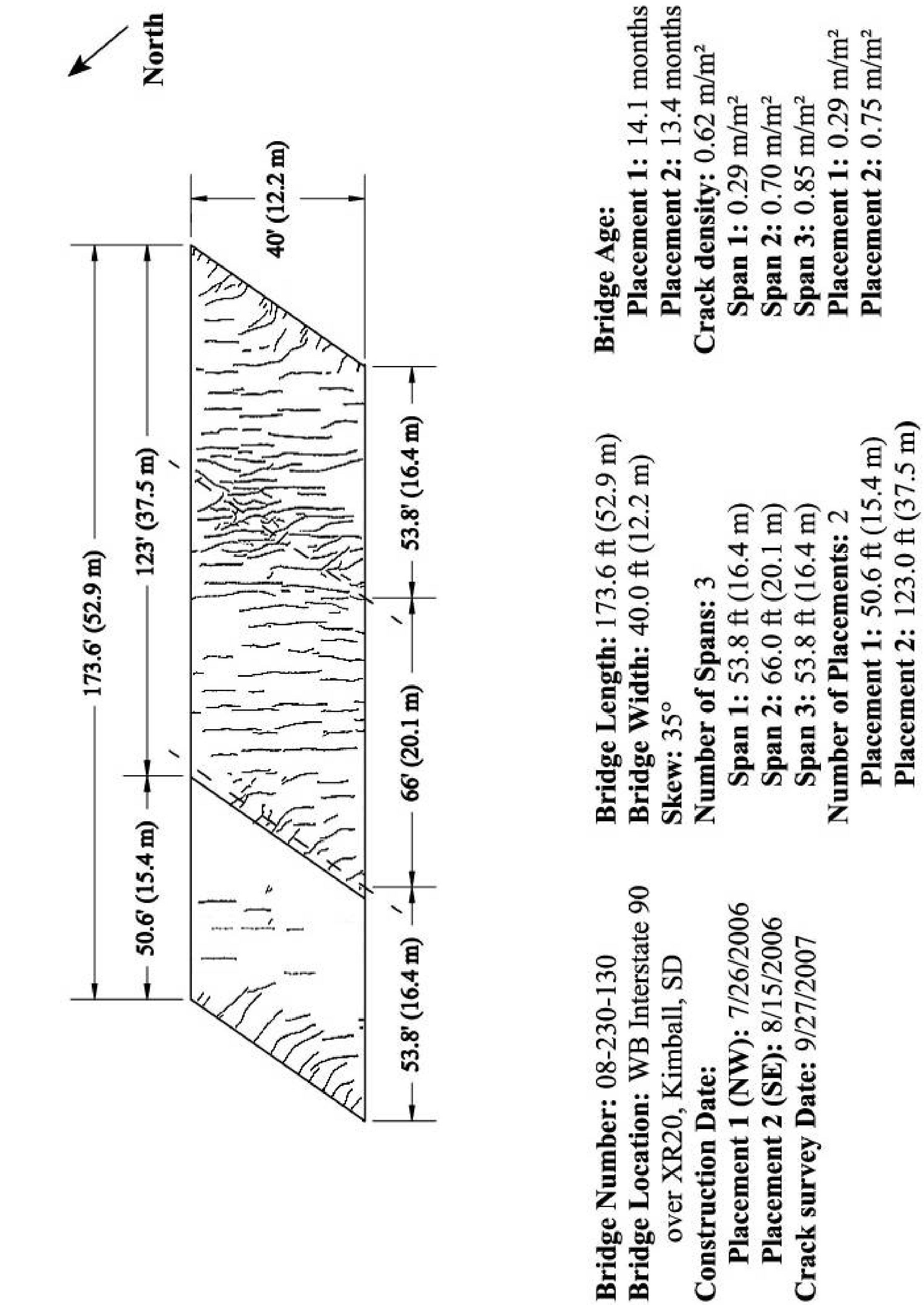
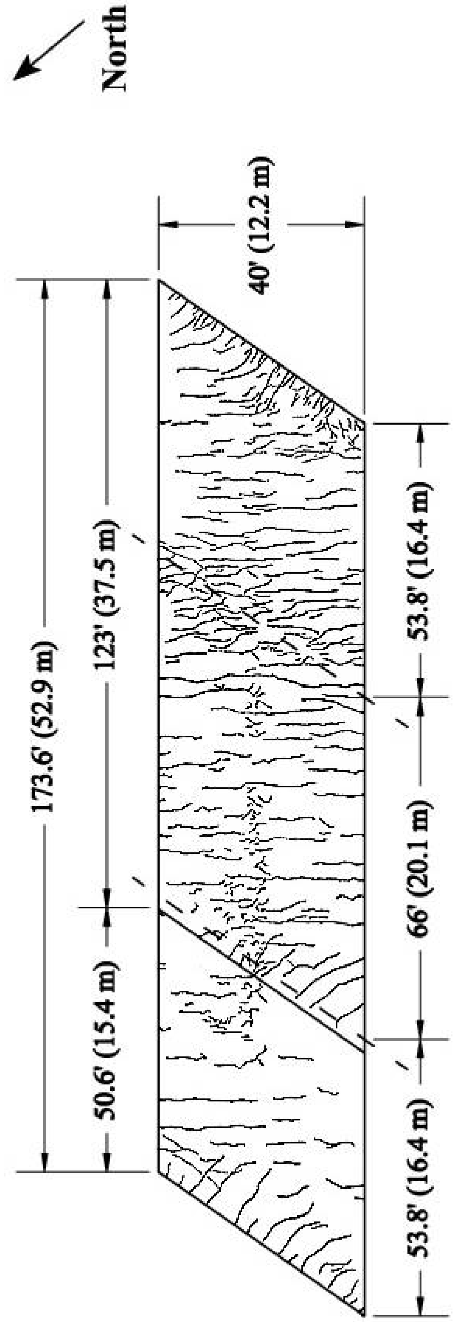


Fig. 9.17 Control Deck-1 Year Crack Survey

The amount of cracking in the control deck increased with time for both placements 1 and 2. The same cracks observed in the 30 day survey have propagated with time. The amount of cracking in placement 2 (0.75 m/m^2) was still higher than placement 1 (0.29 m/m^2). However, the crack densities at one year, for both placements, were approximately five times the crack densities at 30 days. Reduced cracking was seen around the cold joint on the first placement, but on the second placement, perpendicular cracks formed on the cold joint. The unintended cold joint created an unrestrained length of deck in the first placement. While one end of this short length of placement 1 was part of the integral abutment, the other end of the placement was unrestrained allowing the deck to shrink freely before placement 2 was made. Therefore, it is understandable that placement 1 had smaller crack density than that of placement 2.

The two year survey was performed on 8/20/2008. The results can be shown in Figure 9.18.



Bridge Number: 08-230-130
Bridge Location: WB Interstate 90 over XR20, Kimball, SD
Construction Date:
Placement 1 (NW): 7/26/2006
Placement 2 (SE): 8/15/2006
Crack survey Date: 8/20/2008

Bridge Length: 173.6 ft (52.9 m)
Bridge Width: 40.0 ft (12.2 m)
Skew: 35°
Number of Spans: 3
Span 1: 53.8 ft (16.4 m)
Span 2: 66.0 ft (20.1 m)
Span 3: 53.8 ft (16.4 m)
Number of Placements: 2
Placement 1: 50.6 ft (15.4 m)
Placement 2: 123.0 ft (37.5 m)

Bridge Age:
Placement 1: 24.8 months
Placement 2: 24.2 months
Crack density: 0.912 m/m²
Span 1: 0.563 m/m²
Span 2: 0.980 m/m²
Span 3: 1.181 m/m²
Placement 1: 0.566 m/m²
Placement 2: 1.055 m/m²

Fig. 9.18 Control Deck-2 Year Crack Survey

After 2 years, placement 2 (1.055 m/m^2) experienced more cracking than placement 1 (0.566 m/m^2). The incremental increase in cracking between year one and year two was higher for placement 1. In the second year, the cracking in placement 1 nearly doubled, going from 0.29 to 0.566 m/m^2. Placement 2 experienced 40.6% increase in cracking during the second year going from 0.75 to 1.055 m/m^2.

The three year survey was performed on 7/14/2009. The results can be seen in Fig. 9.19.

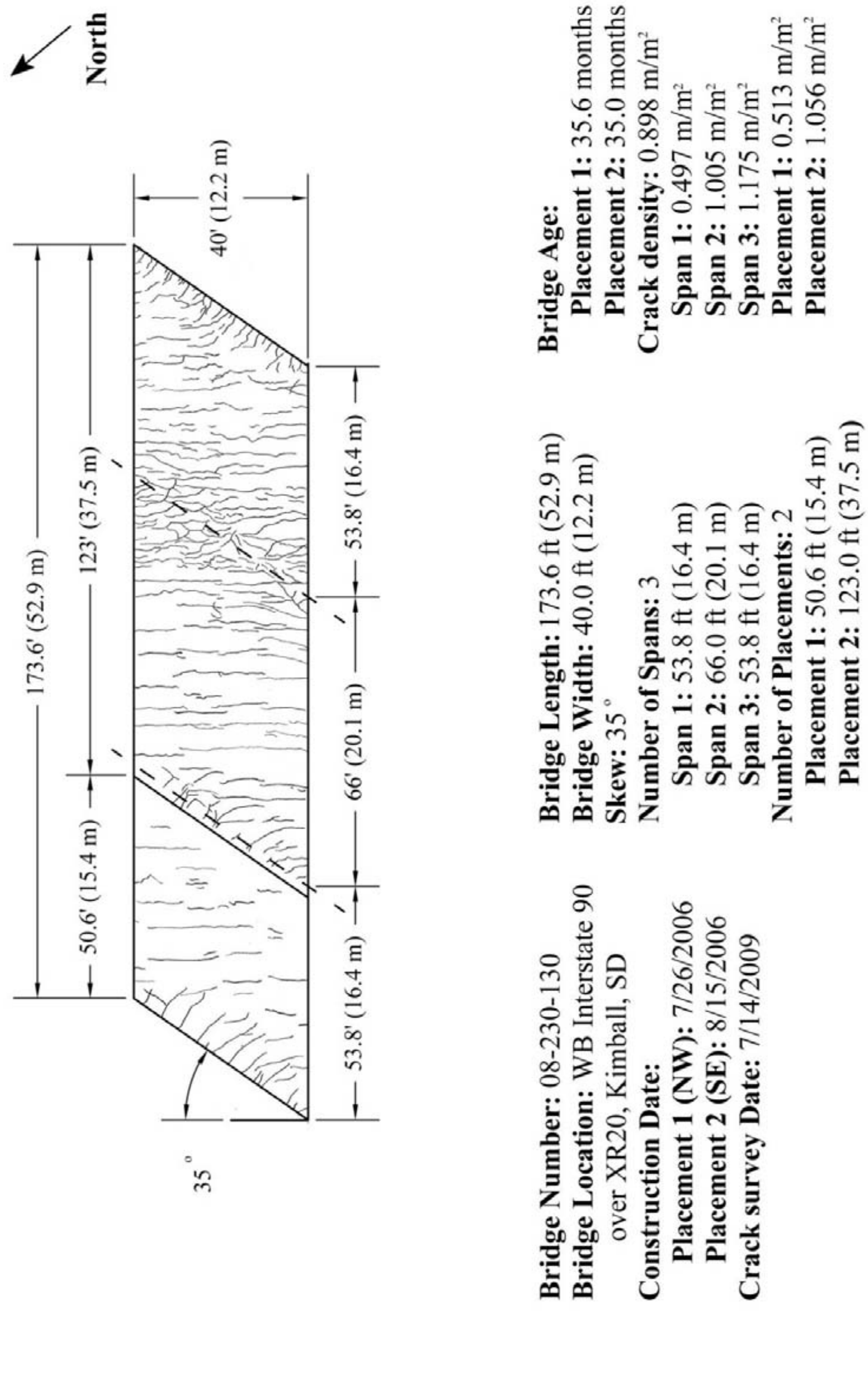


Fig. 9.19 Control Deck-3 Year Crack Survey

The 3 year crack densities for placements 1 and 2 were approximately the same as the 2 year crack densities. The trends in cracking versus time can be seen in Fig. 9.20.

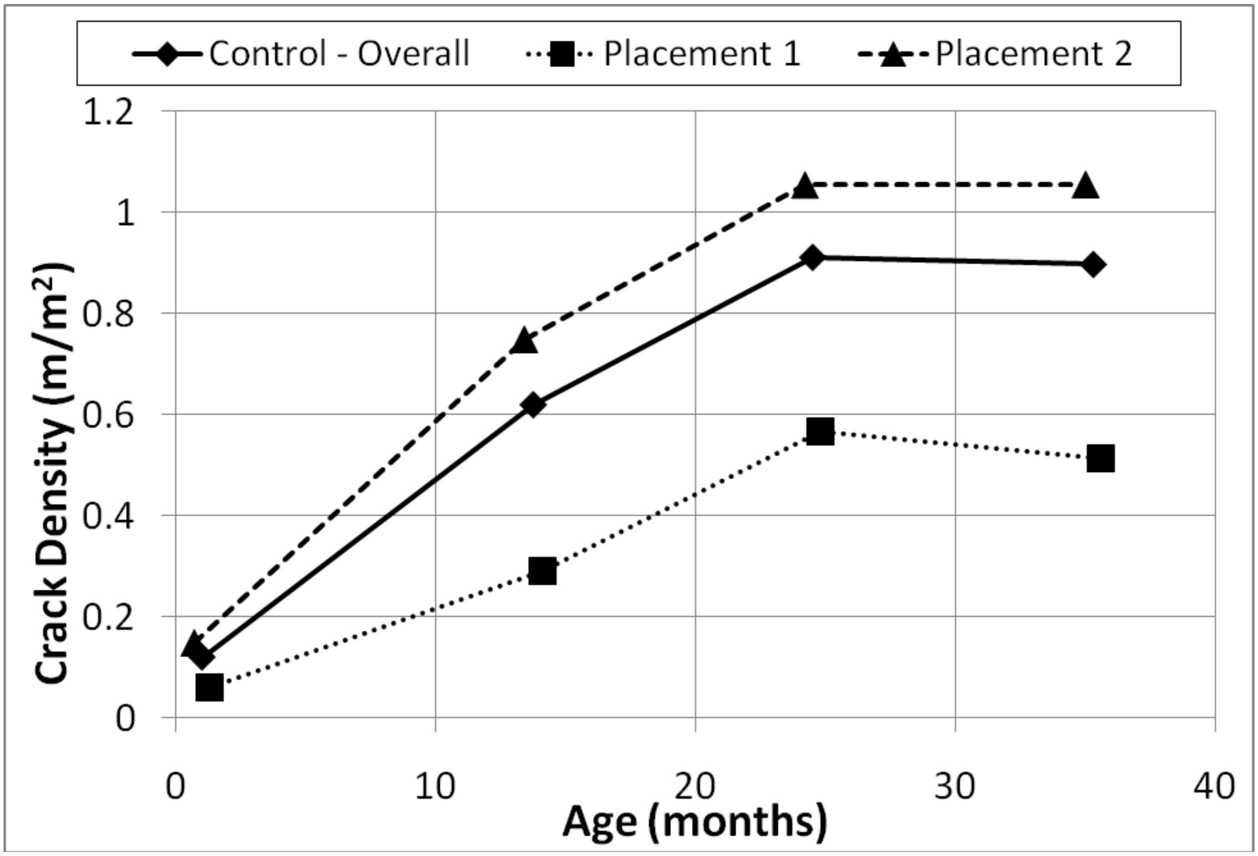


Fig. 9.20 Control Deck Crack Density vs. Time

Both placements 1 and 2 experienced increased cracking with age in the first 2 years. It should be noted that the DOT applied crack sealant to existing cracks between years 2 and 3. While the crack sealant may have prevented the cracks from extending, it also made crack surveying difficult and cracks were not captured in the surveys. This could also explain the small drop in crack density for overall and placement 1 from year 2 to 3.

9.5 East Bound LC-HPC Deck

The bridge for the East bound lane of I-90 was the LC-HPC bridge deck. SDDOT specifications call for 21 days of curing but the contractor elected to extend curing by seven days in lieu of placing a curing compound. The deck was surveys after thirty days of placement. No cracks were observed during the initial survey immediately following the end of the curing period. The moisture provided by curing prevented any early cracks from forming.

The one, two, and three year surveys can be found in Fig. 9.21 through Fig. 9.23. The trends in crack density for the LC-HPC bridge deck can be seen in Fig. 9.24.

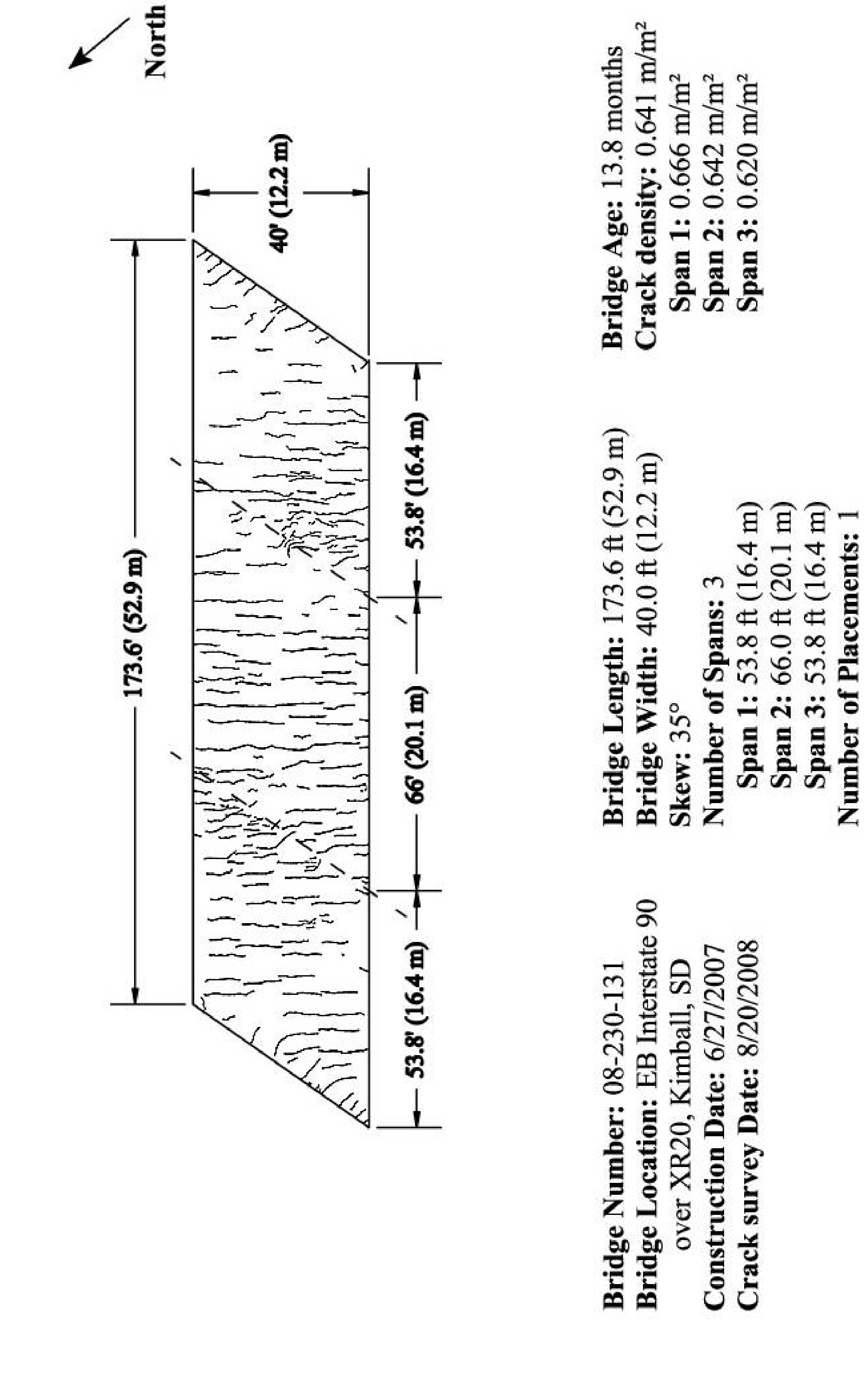


Fig. 9.21 LC-HPC Deck-1 Year Crack Survey

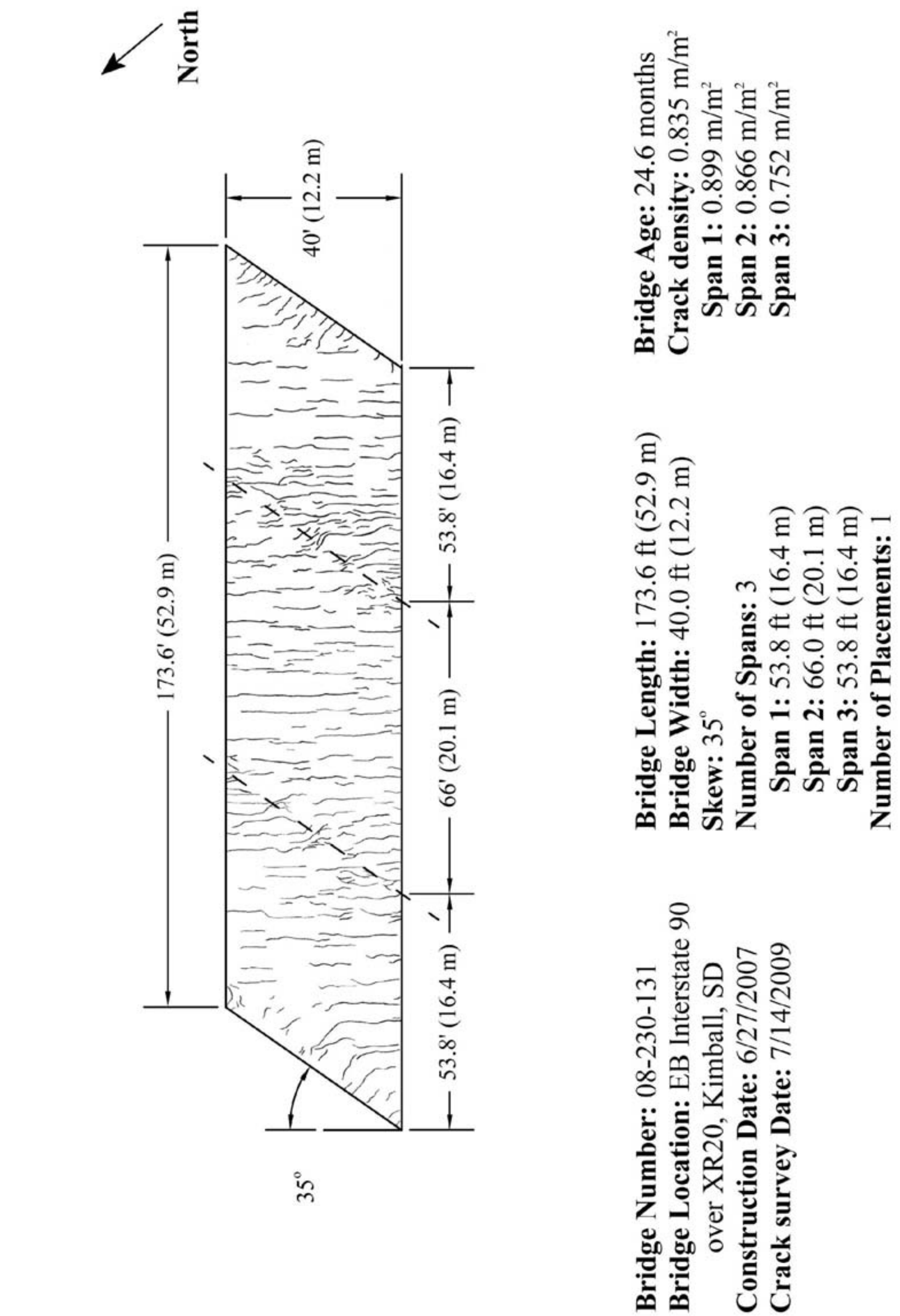


Fig. 9.22 LC-HPC Deck-2 Year Crack Survey

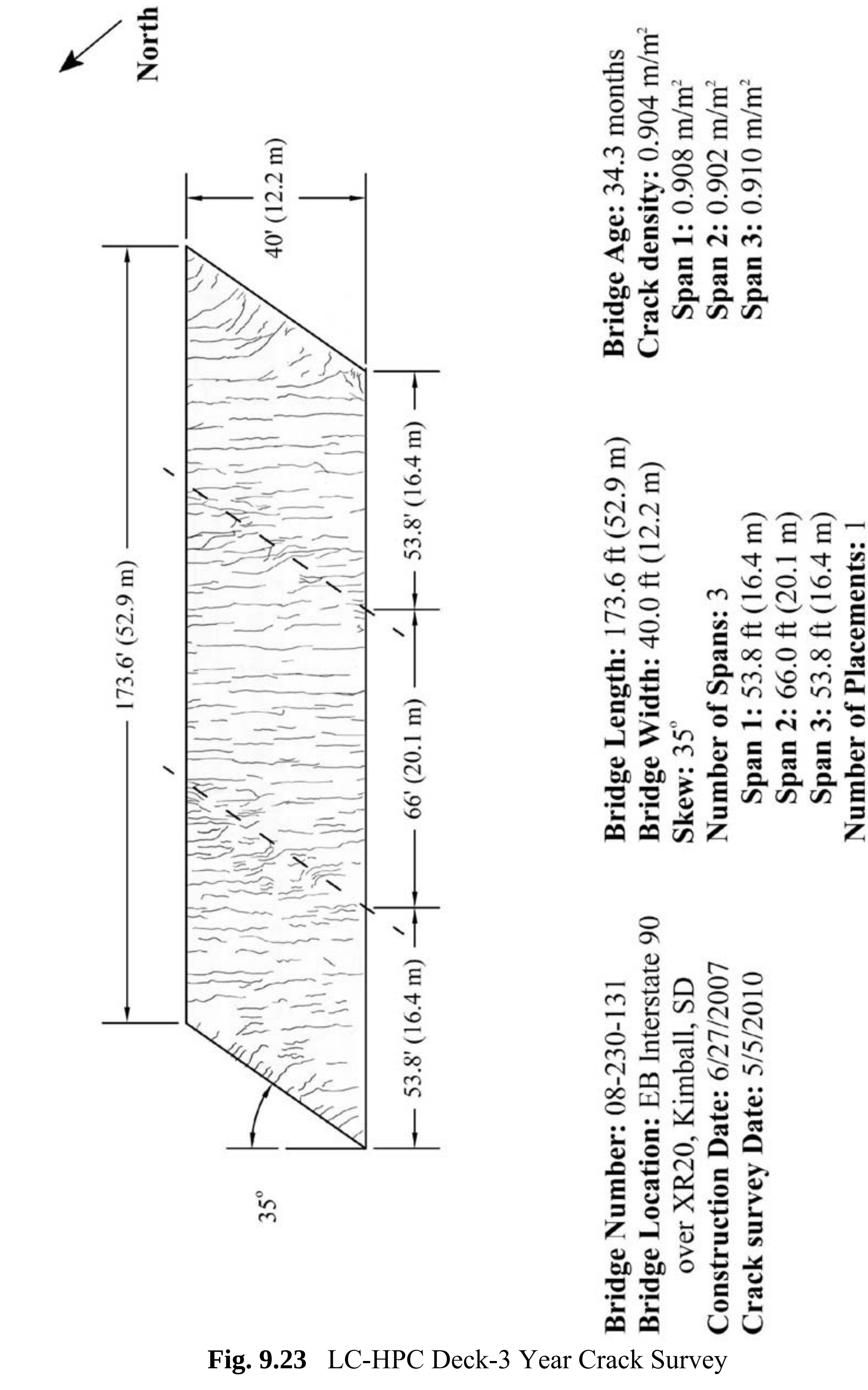


Fig. 9.23 LC-HPC Deck-3 Year Crack Survey

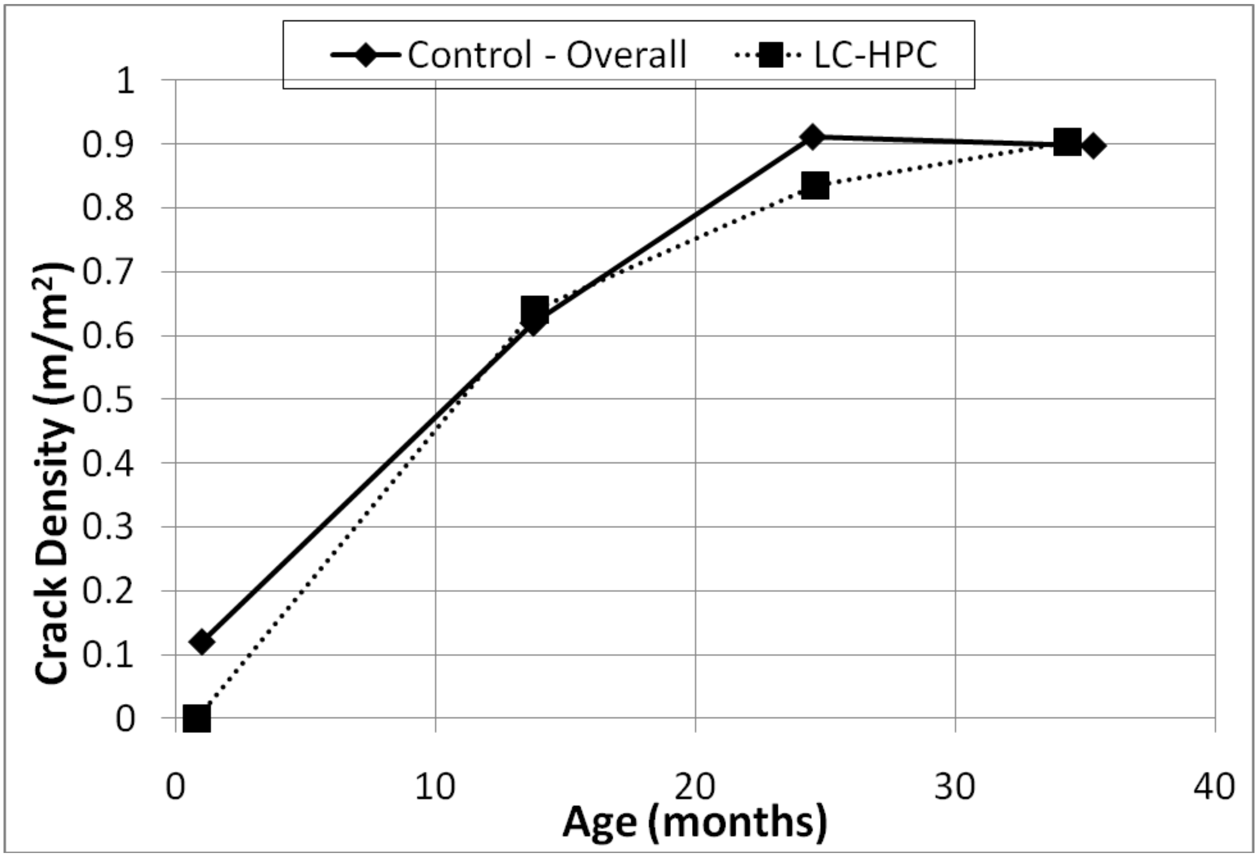


Fig. 9.24 LC-HPC Deck Crack Density vs. Time

The LC-HPC bridge deck cracking performance was similar to the cracking performance of the control deck. The LC-HPC deck should have experienced less cracking but after 3 years, the bridge decks experienced the same amount of cracking. The University of Kansas suggests the fogging system used during placement had an effect on the bridge deck performance. During placement, the deck surface was moistened using a fogger at the finish roller and a fogging bridge between the finish and curing bridge. The foggers at both locations provided a fine water spray rather than a mist. This may have increased the w/c ratio at the top of the deck, raising it past specifications for LC-HPC. The increased w/c ratio led to more cracking and decreased the performance of the LC-HPC bridge deck.

CHAPTER 10 CRACK SURVEYS OF EXISTING BRIDGE DECKS

Task 8. Conduct crack surveys on a minimum of two of each type of South Dakota bridges constructed prior to this research using: old mix design without addition of fly ash; more recent mix design with fly ash but without optimized aggregate; and current mix design with optimized aggregate and fly ash.

10.1 Crack Survey of Existing Bridges

In addition to the two new prestressed girder bridges involved in the LC-HPC study, two more existing prestressed girder bridges and four steel girder bridges constructed three to eight years back were surveyed. Table 10.1 shows all of the bridge decks that were surveyed in this study. All of the bridges had monolithic bridge decks made with or without fly ash, or fly ash and optimized aggregate. The surveys were analyzed for crack trends in girder type, support type, age, and cementitious material. The crack maps for the bridges are shown in Figs. 10.3 to 10.8. Figure 10.1 shows the crack densities versus age of the bridge for the different deck types.

Table 10.1 Summary of Crack Densities of Existing South Dakota Bridge Decks

Structure No.	Date Built	Date of Survey	Girder Type	Deck Type	Deck Length (ft)	Deck width (ft)	Deck Thickness (in)	Skew Angle	Crack Density m/m^2
56-118-127	9/2003	6/2006	Pre-stressed	Current SDDOT mix with flyash	332	36	8.00	15° RHF	0.72
56-118-128	9/2004	6/2006	Pre-stressed	Current SDDOT mix with flyash	332	36	8.00	15° RHF	0.92
18-180-100	9/1988	6/2006	Steel	Old SDDOT mix without flyash	307	40	8.25	0°	0.34
09-126-149	9/1999	6/2006	Steel	Old SDDOT mix without flyash	268	36	8.75	0°	1.16
50-177-199	9/2004	6/2007	Steel	Current SDDOT mix with flyash and optimized aggregate	374	56	8.00	0	0.40
50-178-199	9/2004	6/2007	Steel	Current SDDOT mix with flyash and optimized aggregate	374	56	8.00	0	0.66

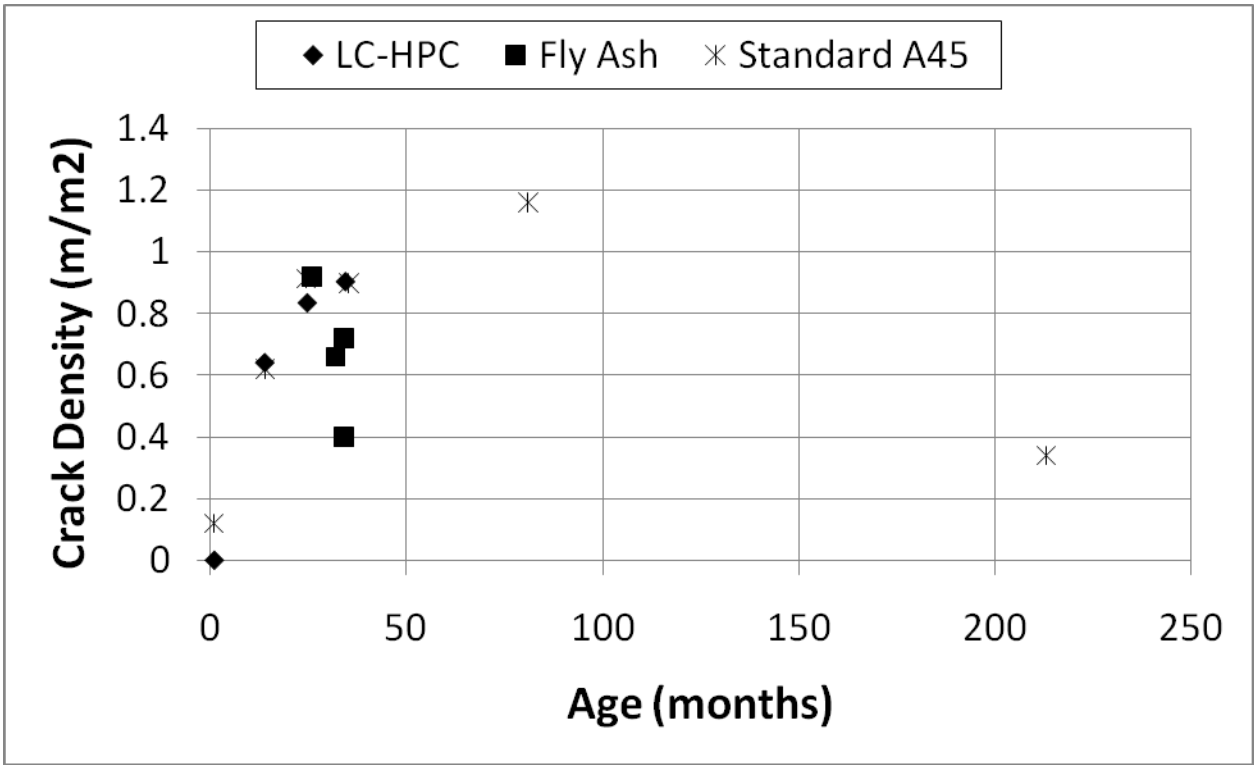


Fig. 10.1 Bridge Deck Crack Densities vs. Time

10.2 Crack Survey Results and Discussion

The results of the crack survey are summarized in Table 10.1 and Fig. 10.1. From the limited data, it was hard to draw conclusions regarding the effects of time on deck cracking. Only the control bridge decks (standard A45 mix) and LC-HPC bridge decks from this project were surveyed more than once. Therefore, there is no direct comparison available for the remaining bridges in this research. Literature shows cracking increases with time and that was seen in the control deck and LC-HPC deck (Schmitt and Darwin, 1995). The other bridges in this study should be surveyed at a later date to determine any age related trends. Of the pairs of bridges with similar decks, it was observed that the older bridge had significantly less cracking. Since deck materials and bridge support types were nearly the same, the difference in crack densities may be the result of different placement conditions and other factors that are unknown and unavailable at this time. Literature states that the temperature high to low variation has a significant impact on early bridge deck cracking (French et al, 1999). Bridges of the same age with different deck types should be surveyed to determine any other trends.

Fig. 10.2 Prestressed and Steel Girder Bridge Deck Crack Densities vs. Time

Figure 10.2 compares the crack densities between prestressed girder and steel girder bridges. Typically the prestressed girder bridges had more cracking than steel girder bridges. The data is limited, but in this study there were two prestressed bridges and two steel girder bridges around the 30 month age. The prestressed bridge had a deck with flyash and the steel girder bridge had a flyash and optimized aggregate deck. The prestressed bridges had significantly more cracking. French et al. (1999) suggested the opposite of this trend. The study concluded that the main cause of bridge deck cracking was the differential shrinkage between the concrete deck and steel girder. The optimized aggregate of the steel girder bridge may have reduced the cracking as the literature suggests (Schmitt and Darwin 1999).

For prestressed girder bridges, the highest crack densities were seen in the interior span. Cracking was concentrated over the supports in the negative moment region. Cracking in these regions was multidirectional, not just transverse. Cracks propagated approximately perpendicular to the supports. This trend is best explained by the integral diaphragm system used; the diaphragm and deck are poured together. The diaphragm may shrink in a transverse direction while the bridge deck shrinks in a longitudinal direction. This same cracking trend was observed at the boundary of the bridge deck. While the prestressed girder is considered simply supported, the abutment beneath the girder may cause the bridge deck to expand transversely.

Steel girder bridges had several trends that agreed with the literature (Schmitt and Darwin 1995). The highest crack densities were seen in the middle span and cracking was concentrated in the negative moment regions. The trend of perpendicular cracks at the bridge ends was also seen. Steel girder bridges have integral abutments where the ends of the girders are cast into the abutments providing a fixed connection. French et al. (1999) suggests that a fixed connection leads to more cracking but transverse expansion of the integral abutment may cause the perpendicular cracks seen at the end of the bridges. With steel girders, more longitudinal cracking was seen which suggest there is a significant amount of flexure occurring over the flange of the girders.

For further research, data should be collected for these existing bridges, for bridges with beam/supports that are not integral with the decks, and more. The bridges should be selected carefully so that they can be compared to other bridges with similar characteristics so more substantial conclusions can be made regarding the trends in bridge deck cracking in South Dakota.

10.3 References

French, C., Eppers, L., Le, Q., Haijar, J. F. (1999). “Transverse Cracking in Concrete Bridge Decks,” *Transportation Research Record 1688*, Transportation Research Board, National Research Council, Washington, D.C., pp. 21-29.

Schmitt, T. R., and Darwin, D. (1995). “Cracking in Concrete Bridge Decks,” *SM Report* No. 39, The University of Kansas Center for Research, Inc., Lawrence, Kansas, 151 pp.

Schmitt, T. R., and Darwin, D. (1999). “Effect of Material Properties on Cracking in Bridge Decks,” *Journal of Bridge Engineering*, ASCE, Feb., Vol. 4, No. 1, pp. 8-13.

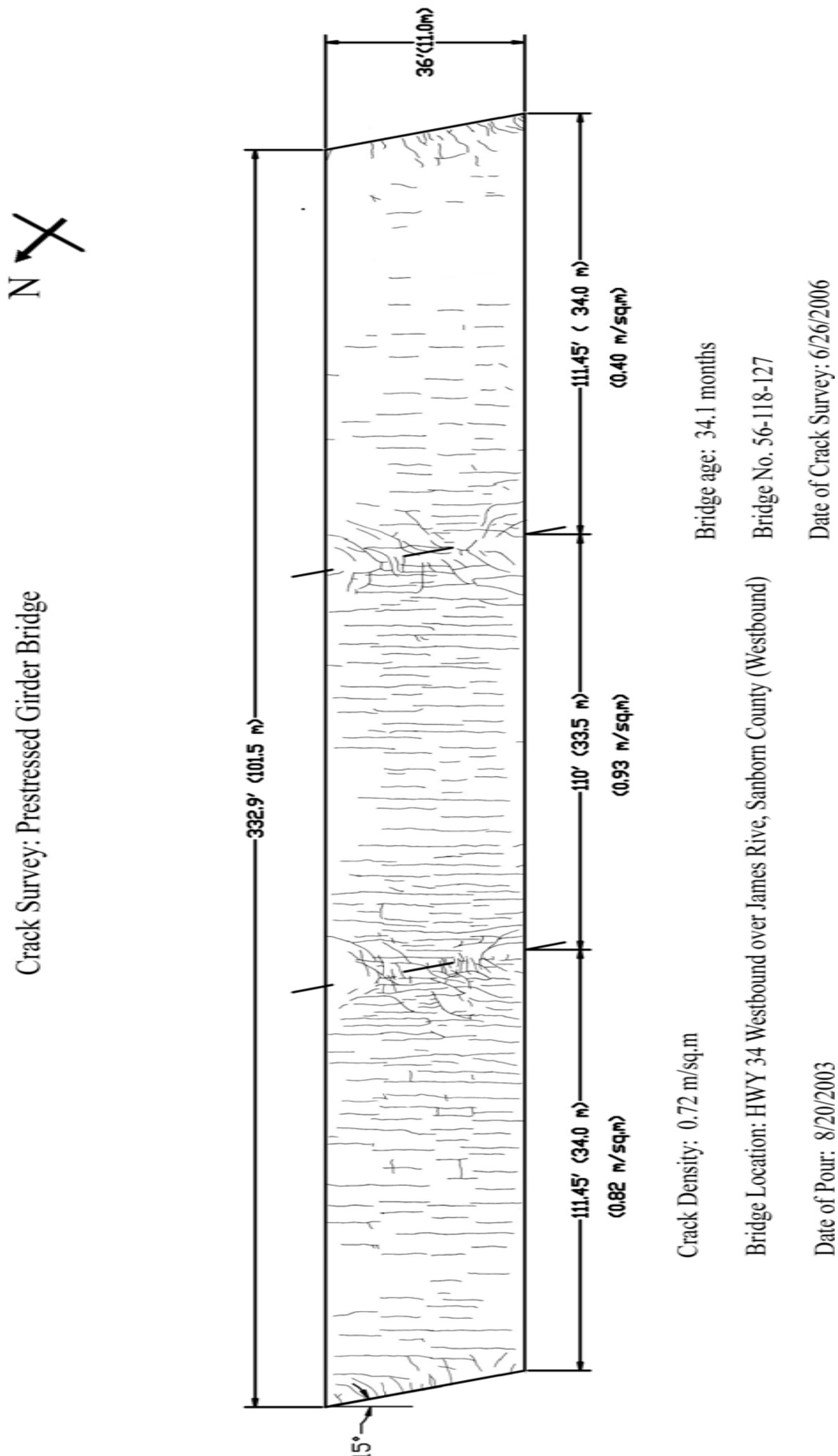


Fig. 10.3 Prestressed Girder Bridge #56-118-127 Crack Survey

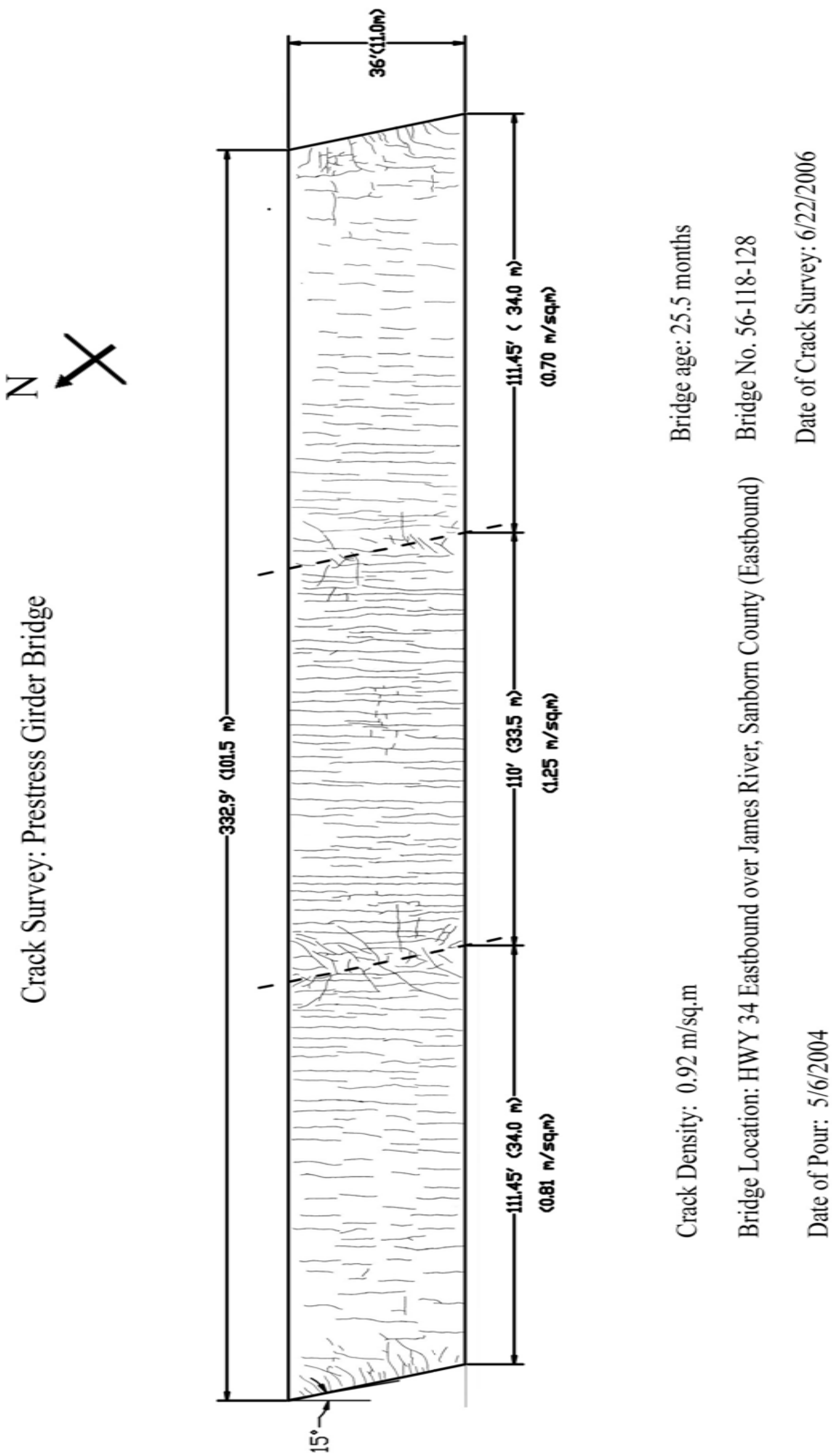


Fig. 10.4 Prestressed Girder Bridge #56-118-128 Crack Survey

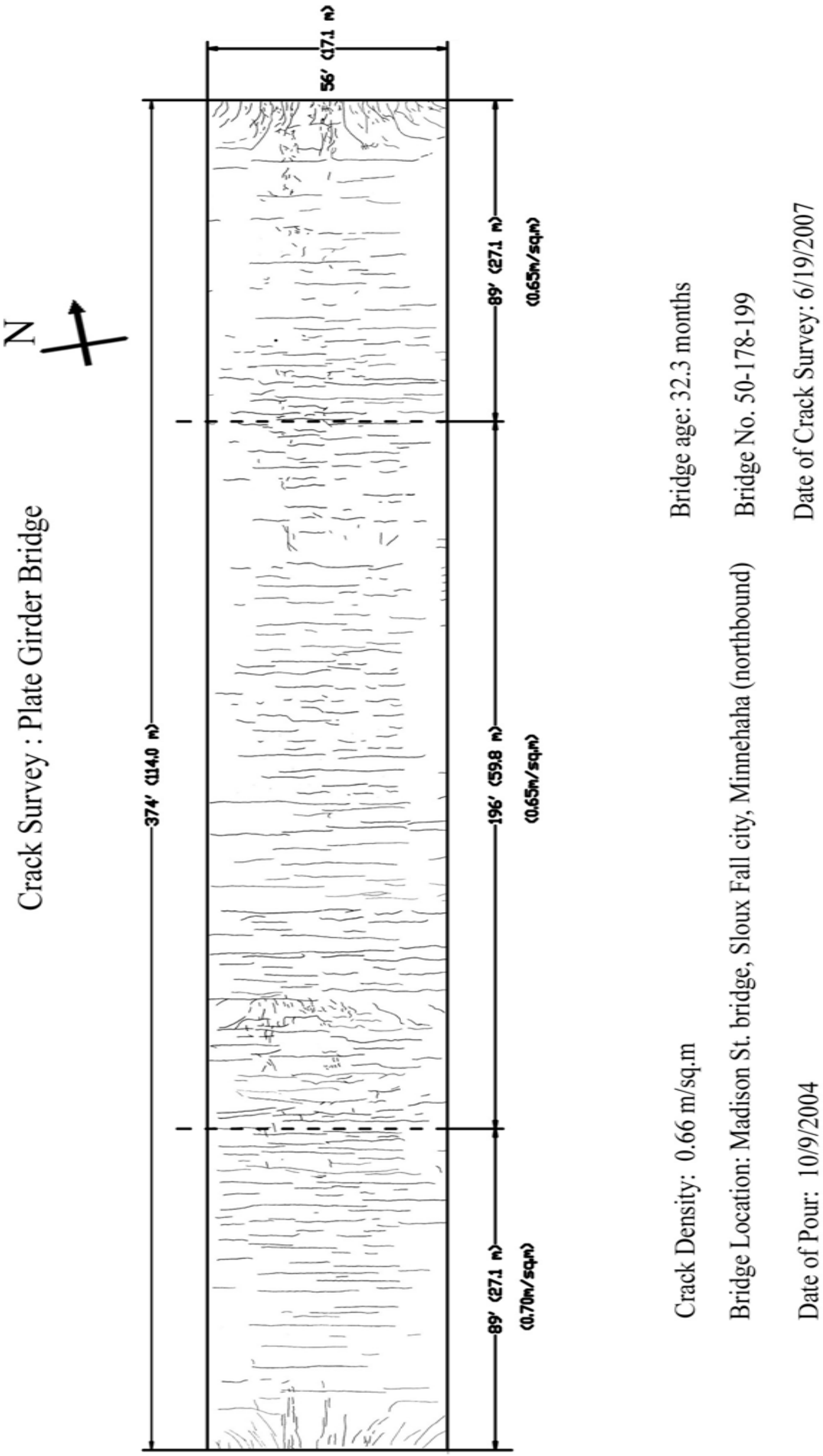


Fig. 10.5 Steel Girder Bridge #50-178-199 Crack Survey

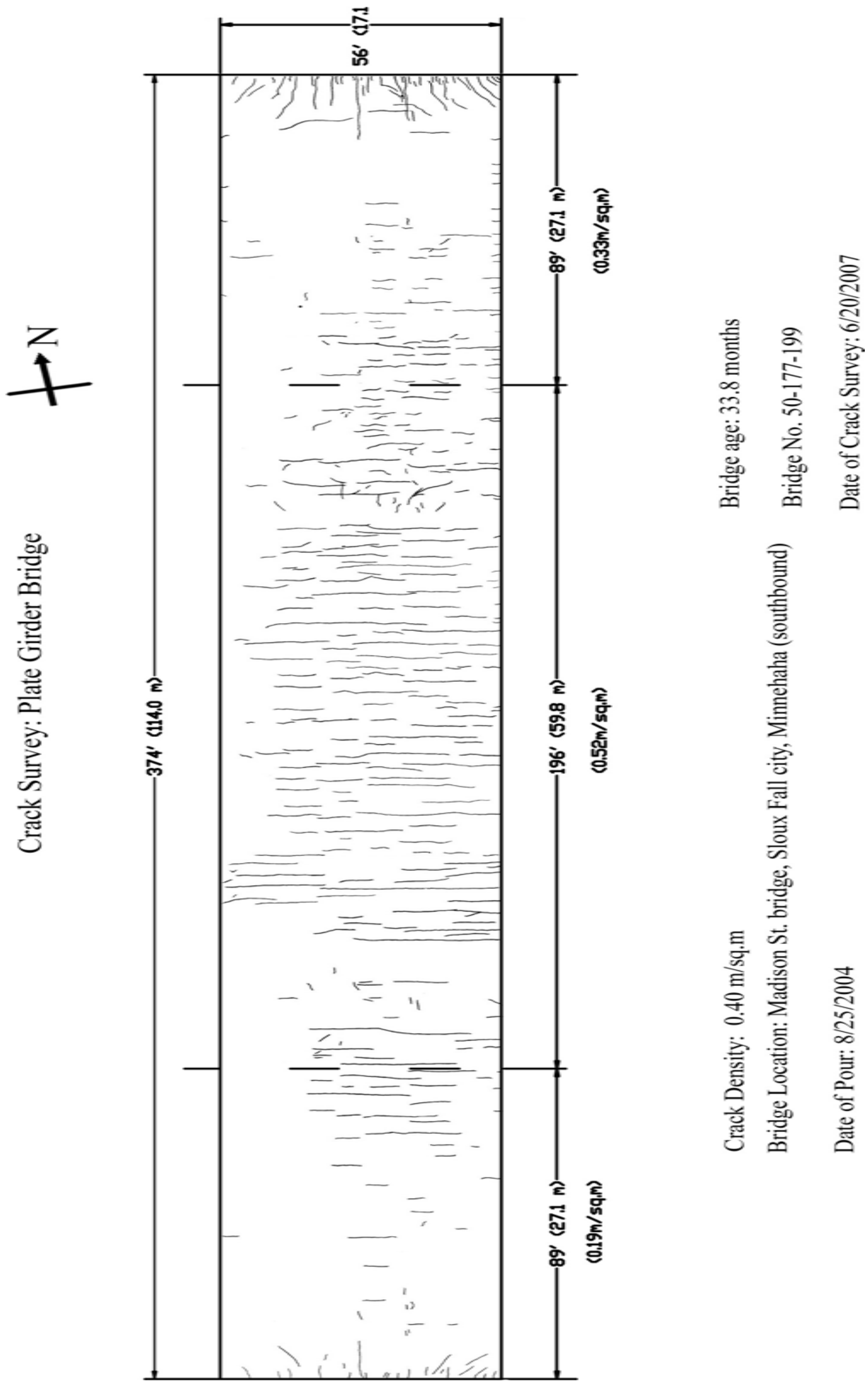


Fig. 10.6 Steel Girder Bridge #50-177-199 Crack Survey

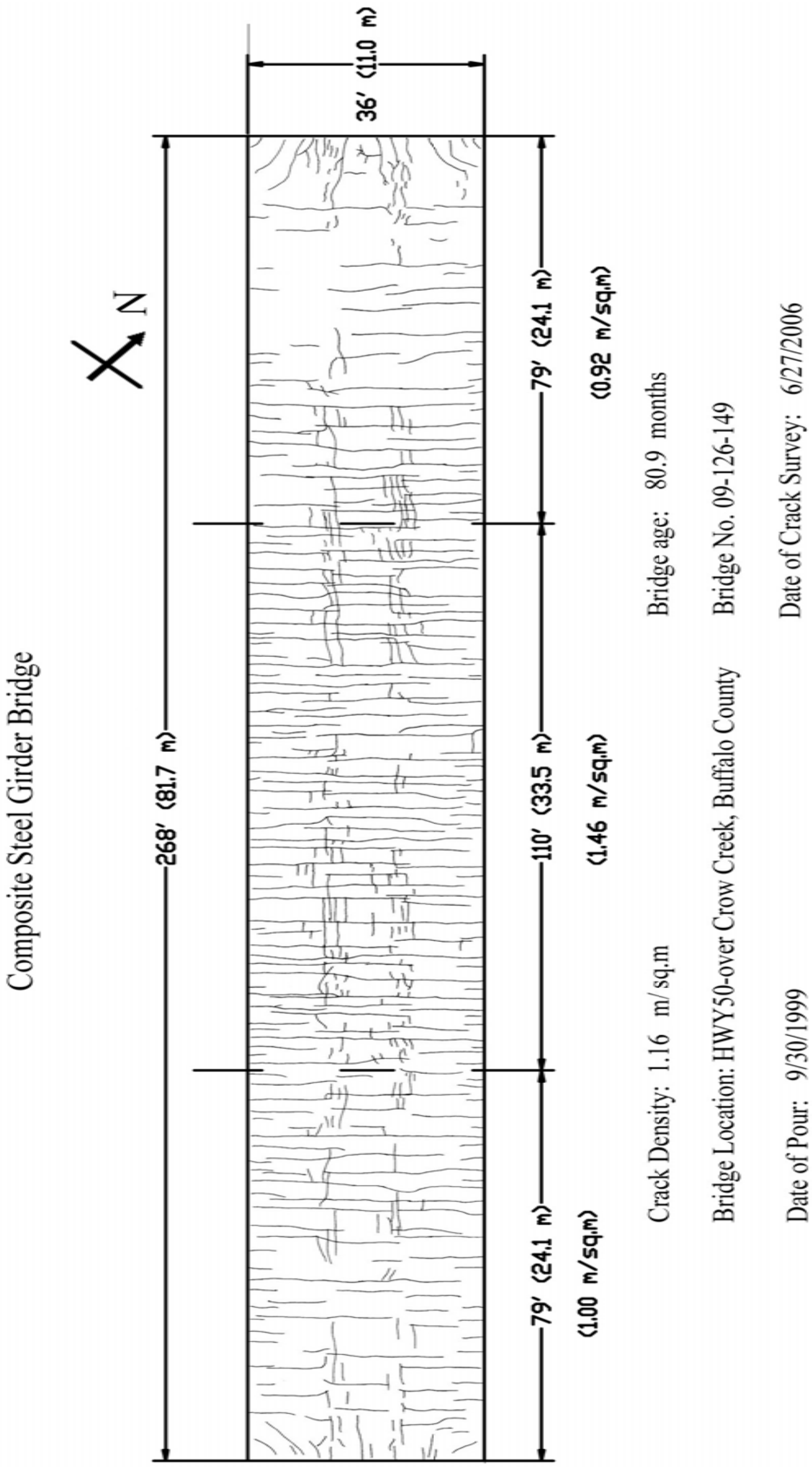


Fig. 10.7 Steel Girder Bridge #09-126-149 Crack Survey

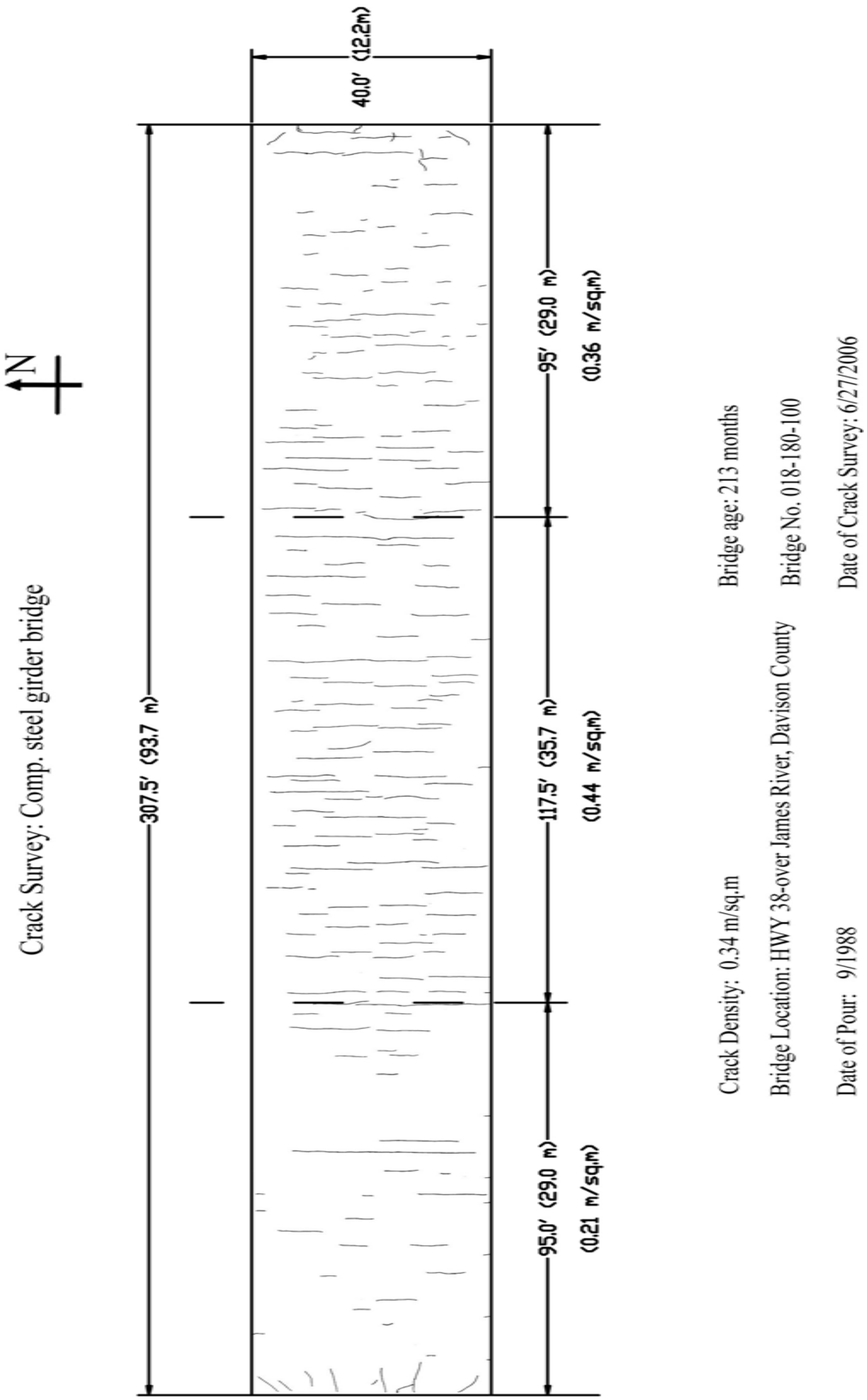


Fig. 10.8 Steel Girder Bridge #18-180-100 Crack Survey

CHAPTER 11 FINAL REPORT AND PRESENTATION

Task 9. Compare current SDDOT materials and construction specifications to the draft Pooled Fund material and construction specifications being developed for crack-free bridge decks with respect to constructability, South Dakota materials, and concrete plant capabilities and recommend best practices for future projects.

This task is addressed in several parts in other chapters of this final report. However, the advantages of using LC-HPC concrete in South Dakota bridge decks have not been fully realized in this project. Therefore, there seems to be no merit at the moment in recommending best practices for future projects because it is unsubstantiated (from a crack density point of view) that LC-HPC is resulting in reduced cracking in bridge decks compared to South Dakota A45 concrete mix.

Task 10. Prepare a final report and executive summary of the prior research, research methodology, findings, conclusions and recommendations.

The final report was submitted to SD-DOT in June 2011.

Task 11. Make an executive presentation to the SDDOT Research Review Board at the conclusion of the project.

The executive presentation for the project was done at the RRB meeting on Dec. 16, 2010.

CHAPTER 12 PETROGRAPHIC ANALYSIS

Supplemental Task S1. Perform Petrographic Analysis of Eight Concrete Cores Cut from Four Bridge Decks (two from each type) in the Two Counties

12.1 Concrete Core Details

Four inch diameter concrete cores were cut from the test slab, A45 concrete bridge deck (west bound) and LC-HPC bridge deck (east bound) for Pennington County bridges. The locations of the cores where they were cut from are shown in Figs. 12.1 to 12.2.

The pictures of the cores cut from test slab are shown in Figs. 12.3 to 12.6. The cores cut from A45 concrete bridge (west bound) are shown in Figs. 12.7 and 12.8. The cores cut from the LC-HPC bridge (east bound) are shown in Figs. 12.9 and 12.10. Physical inspection of the cores generally showed that there were more or less the same amounts of air voids on the surfaces of the cores for the three sets of cores cut from the three types of concretes (test slab, A45 bridge deck and LC-HPC bridge deck). The consolidation of concrete appeared to be the same or marginally different, if any. The paste content also seemed to be more or less same in all the concrete cores. There appeared to be very good bond between the aggregate particles and the cement paste. All the aggregate particles seemed to be fully coated with cement paste.

Fig. 12.11 and Fig. 12.12 shows three cores each cut from Brule County bridges. Physical inspection of the concrete cores cut from the test slabs, A45 concrete deck and LC-HPC deck showed no discernable difference between the microstructure of the cores. Air voids, aggregate content, and distribution, and an estimate of density, absorption and void content were determined from a petrographic analysis conducted at SDSMT. The quartzite cores (Brule County) tend to have slightly greater air void content and slightly greater total aggregate content than the limestone cores (Pennington County). The concrete consolidation for all the bridge decks was very good. The average coarse aggregate content in the cores was found to be about 40% on the slices cut and examined. The volume of permeable voids for these concretes ranged from 10.6 to 12.5%. Complete details of petrographic evaluation of the eight concrete cores are given in the following section.

12.2 Petrographic Analysis of Concrete Cores Cut from the Decks from the Two Counties

Air voids, aggregate content, and distribution, and an estimate of density, absorption and void content were determined from a petrographic analysis conducted by Dr. Edward Duke of SDSMT. The report produced by him is included in Appendix E in its entirety and was a study independent of this project and the listed investigators.

The results in Table 1 of the report suggest that the quartzite cores tend to have slightly greater air void content and slightly greater total aggregate content than the limestone cores. There does not appear to be a significant difference in air void content in the top 1 inch versus the bottom 2 inches indicating that the wet concrete consolidation was very good. The total aggregate content in the top 1 inch appears to be slightly greater than in the bottom 2 inches, regardless of coarse aggregate type. The average coarse aggregate content in the cores was found to be about 40%. The volume of permeable voids for these concretes ranged from 10.6 to 12.5%. The tests were conducted in accordance with ASSTM C642.

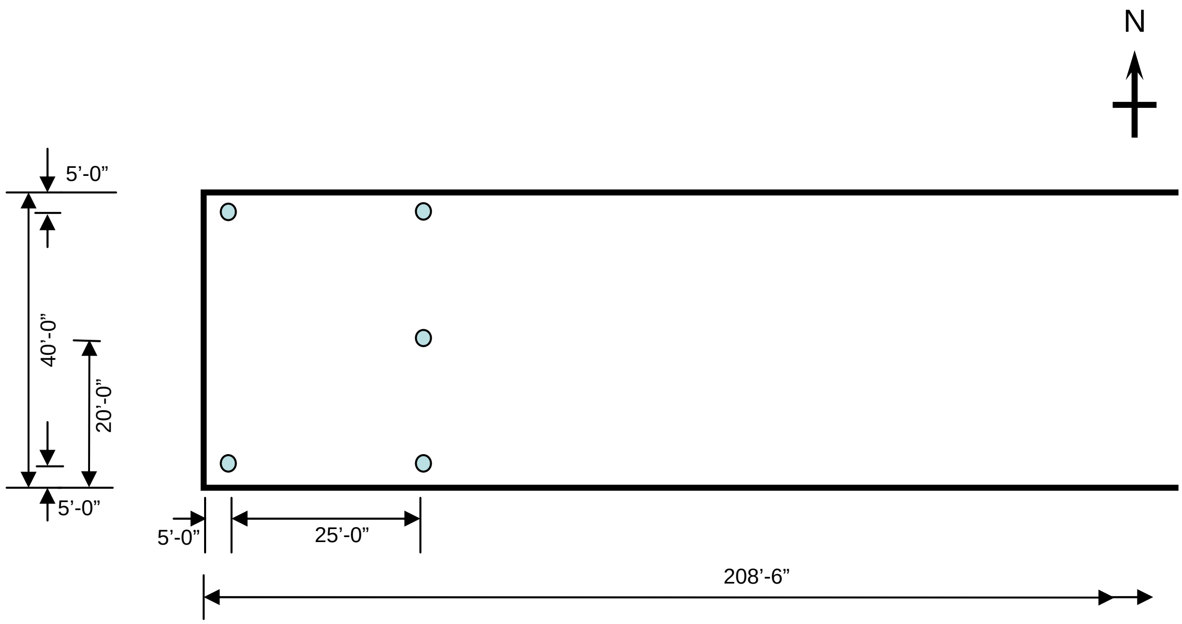


Fig. 12.1 Location of Cores cut from Pennington County Control Bridge Deck (West Bound)

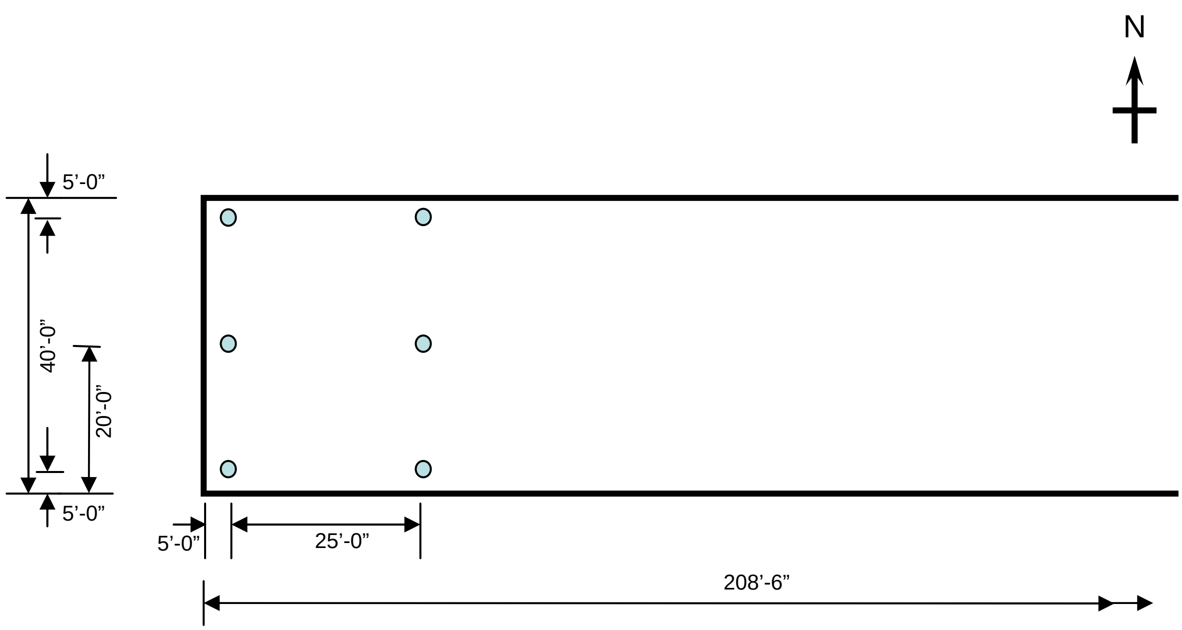


Fig. 12.2 Location of Cores cut from LC-HPC Bridge Deck (East Bound) Pennington County

Fig. 12.3 Cores cut from Test Slab (Front)

Fig. 12.4 Cores cut from Test Slab (Back)

Fig. 12.5 Cores cut from Test Slab (Front)

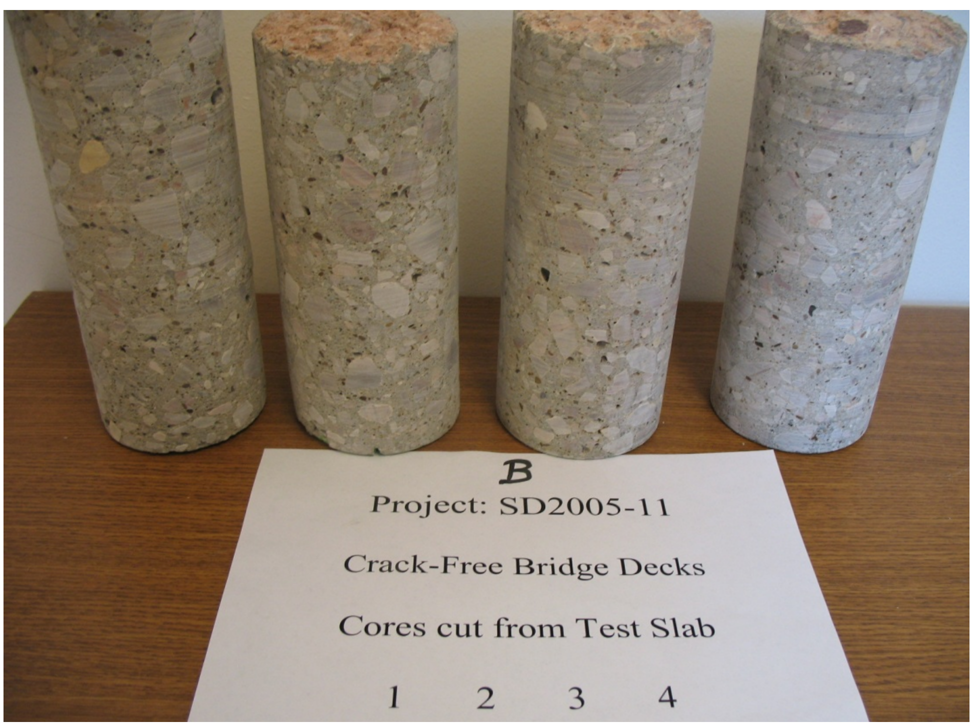


Fig. 12.6 Cores cut from Test Slab (Back)

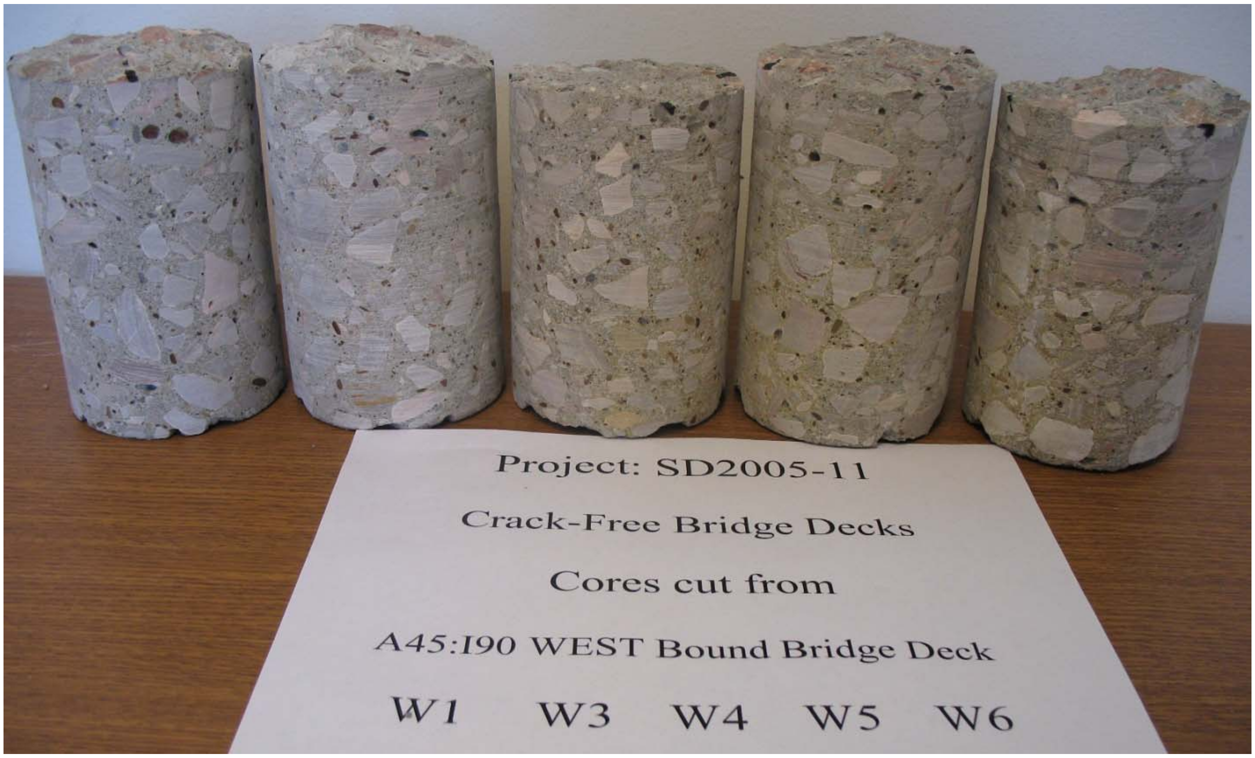


Fig. 12.7 Cores cut from Control Slab – A45 (Front)

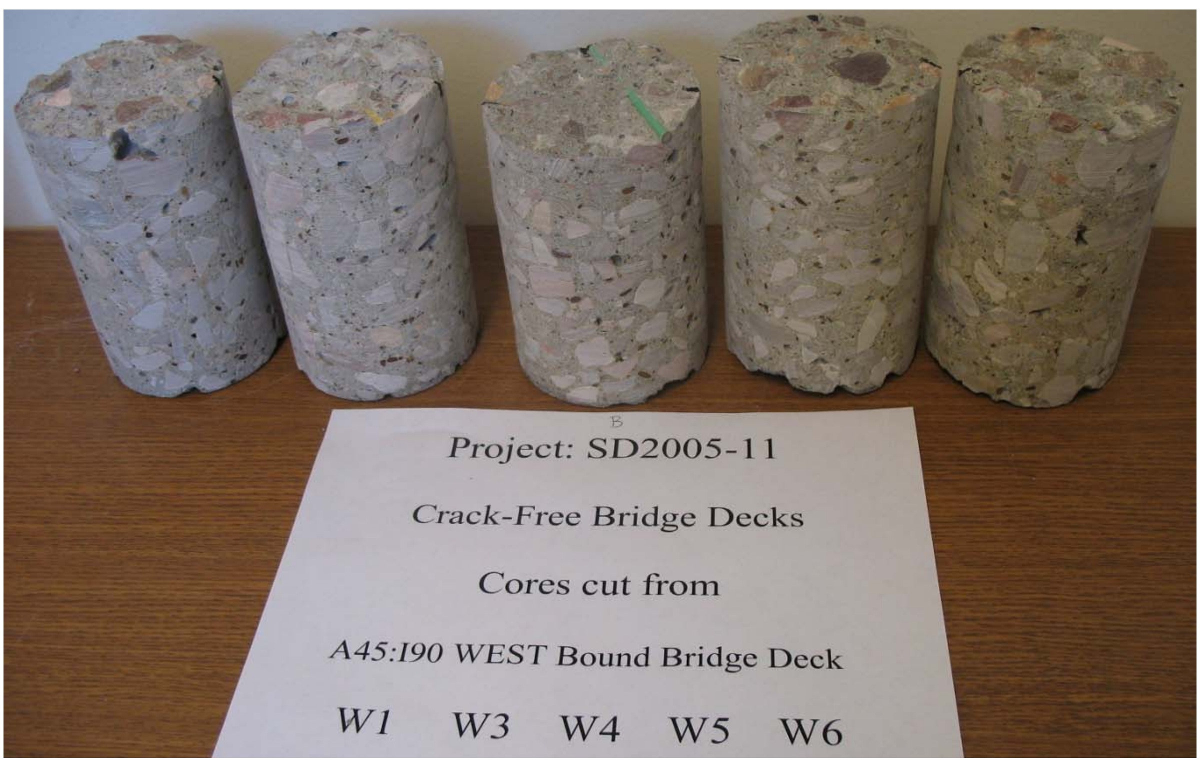


Fig. 12.8 Cores cut from Control Slab – A45 (Back)

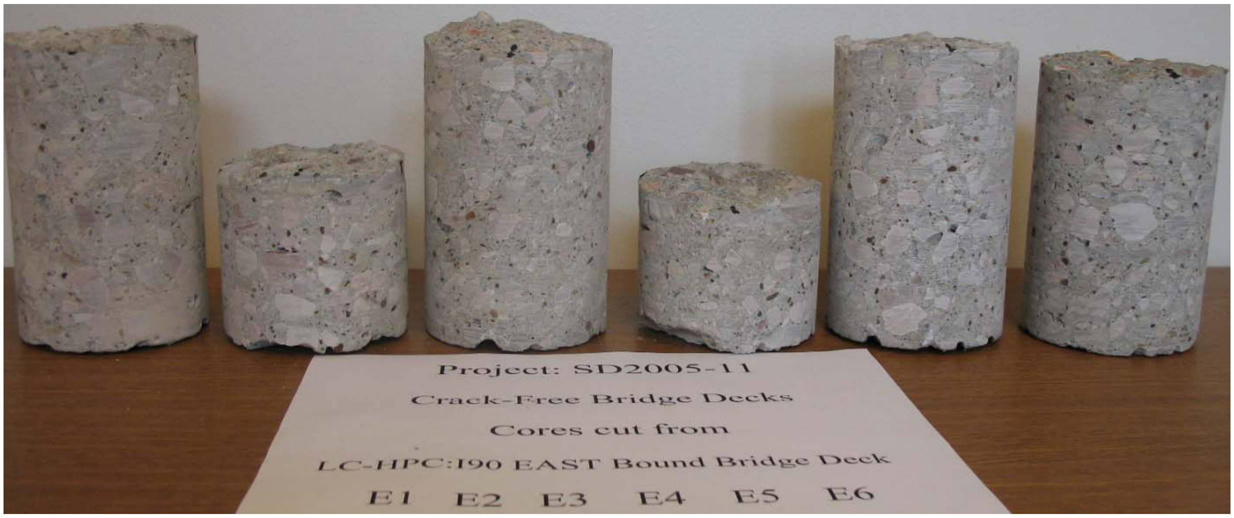


Fig. 12.9 Cores cut from LC-HPC Slab (Front)

Fig. 12.10 Cores cut from LC-HPC Slab (Back)

Fig. 12.11 Cores cut from Control Slab – A45 in Brule County

Fig. 12.12 Cores cut from LCHPC Slab in Brule County

CHAPTER 13 DISCUSSION AND CONCLUSIONS

13.1 Discussion

The following are pertinent discussion and observations from the constructability studies of the project:

(a) Mode of placement must be considered when developing mix design for LC-HPC. If the contractor intends to use concrete pump, the mix should be accordingly proportioned. While the switch to conveyor did not appear to cause problems in the Pennington bridges, maintaining the consistency of air content of concrete remains a challenging and unresolved issue.

(b) The vibrator setup with vibrators mounted on the cross beam of the finishing machine worked well. The layout of vibrators and the spacing between vibrators must be adjusted to match the width of the deck so that double vibration at the ends is avoided.

(c) Finishing remains a major problem particularly if one roller is used in the finishing machine.

(d) The amount of fogging and mist pattern need to be well regulated, because unregulated fogging causes water puddles or insufficient fogging on a hot day. The term “strike-off” needs to be well defined for any useful interpretation and implementation of the specified requirement to commence curing within 10 minutes of “strike-off”. This may also result in an undesirable increase of the w/c ratios at the deck surface.

(e) Field tests require four to five SDDOT testers continuously for LC-HPC which is more than what are required during the placement of decks with A45 concrete.

(f) It is generally perceived that production, placement, consolidation, and curing of LC-HPC in bridge decks cost more than that for A45 concrete. The extent of extra costs in implementing LC-HPC is not conclusively known.

(g) LC-HPC was placed under ideal weather conditions where the temperature was between 50 to 60° F and humidity close to 80% during much of the pour. It is unknown what the problems could be if LC-HPC is placed under hot weather conditions. The pour may have to be done at night.

(h) It is important that the contractor be able to handle the mix and to have proper weather protection available during placement. Skewing the finishing bridges with the actual deck performed and mounting the vibrators on the front of the finishing machine also worked well. Fogging needs to be modified to create a mist rather than fine water spray.

(i) Comments on importance of curing?

13.2 Conclusions

This study compares the constructability of LC-HPC with A45 concrete for bridge deck applications. Wet concrete properties, compressive strength development, and drying shrinkage properties were studied. The cores cut from the test slab, A45 concrete deck slab, and LC-HPC deck slab were physically and petrographically examined. The following conclusions are drawn from the project:

(a) It is feasible to make, place, consolidate and cure LC-HPC with all the relevant special requirements in a bridge deck placement environment. The wet concrete properties specified in the special specifications were achievable. The compressive strength and static modulus of LC-HPC were about the same as that of A45 concrete at 28 days. The drying shrinkage of LC-HPC made in the laboratory at 60 days was 100 x 10^{-6} less than that of A45 concrete. The project findings demonstrated that it is feasible to produce and place LC-HPC for bridge decks.

(b) It is important that the contractor be able to handle the mix and to have proper weather protection available during placement. Skewing the finishing bridges with the actual deck performed and mounting the vibrators on the front of the finishing machine also worked well. Fogging needs to be modified to create a mist rather than fine water spray. All phases of the deck construction: placement, consolidating, finishing, and curing must be executed efficiently to meet the placement rates outlined in the specifications.

(c) Greater effort is required for all phases of the deck construction with LC-HPC such as placement, consolidating, finishing, and curing which must be executed efficiently to meet the placement rates and more stringent specifications than the corresponding standard A45 mixes, potentially increasing the cost of placement.

(d) The cores cut from test slab, A45 deck and LC-HPC deck showed similar microstructure and physical properties, indicating that the physical structure of LC-HPC is similar to that of A45 concrete, and there was very good consolidation of concrete for all the decks. The average coarse aggregate content in the cores was found to be about 40%. The volume of permeable voids for these concretes ranged from 10.6 to 12.5%.

(e) The crack densities that were determined in this project indicate that currently used SD-DOT A45 concrete deck had a lower crack density of 0.73 m/m^2 after three years of service compared to the crack density of 0.85 m/m^2 for LC-HPC at about the same time of service in Pennington County. In Brule County, the crack density of A45 concrete bridge deck was 0.9 m/m^2 after three years of service. The crack density of LC-HPC bridge deck was also found to be 0.9 m/m^2 after three years of service. These crack densities reveal that South Dakota A45 mix performed nearly as well as or better than the proposed new LC-HPC mix for the four bridge decks evaluated in this project.

(f) For new and existing bridges in South Dakota it may concluded that:

- LC-HPC had significantly less early (before 30 days) cracking.
- Cracking increased with time.
- Steel girder bridges with optimized aggregate cracked less than prestressed girder bridges without optimized aggregate gradation.
- Both types of bridges had higher crack densities in negative moment regions and the interior spans.
- Other transportation agencies have investigated bridge deck cracking and have implemented different methods to control deck cracking. There seems to be no geographical trends for bridge deck cracking problems.

13.3 Recommendations

The procedures used in the construction of the LC-HPC bridge decks construction went according to plan. The following are the suggested recommendations from the project:

1) From the findings of this project, the researchers recommend that there is no tangible benefit in a switch from South Dakota A45 concrete mix to LC-HPC concrete for bridge decks in the State at the moment. More studies are required to demonstrate that it is beneficial before a switch to LC-HPC is considered because there may be potential cost implications that need to be established as well. Definitely, the project revealed that more efforts are need to adopt LC-HPC for routine bridge deck construction. Thorough studies are needed to prove or disprove the effects of cost implications.

2) However, it is feasible to produce and place LC-HPC for bridge decks. It is important that the contractor be able to handle the mix and to have proper weather protection available during placement. Skewing the finishing bridges with respect to the actual deck layout is a preferred option. Vibrators may be mounted on the front of the finishing machine. Fogging needs to be modified to create a mist rather than fine water spray. All phases of the deck construction, i.e., placement, consolidating, finishing, and curing must be executed efficiently to meet the placement rates outlined in the specifications in order for LC-HPC to be placed successfully per the suggested specifications.

3) Concrete mix proportions must be designed for pumping or conveyor consistent with the placement method to be used at site.

4) Care needs to be taken to design the vibrator mounting arrangement to prevent overlapping of vibration at the edge in future constructions.

5) Placement of concrete is to be avoided during severe windy conditions.

6) The data from the existing bridges was limited. It is recommended that a more in-depth study, involving the same and other bridges, be performed to make more substantial claims regarding bridge deck cracking trends.

Appendix A

Special Provisions for LC-HPC

(Developed by SD-DOT)

STATE OF SOUTH DAKOTA
DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION
FOR
LOW-CRACKING HIGH PERFORMANCE CONCRETE (LC-HPC) BRIDGE DECK

PROJECT IM 90-2(134)59 & P 2016(13)71, PCN 4259
PENNINGTON COUNTY

MARCH 30, 2005

Modify Section 460 of the Standard Specifications for Roads and Bridges as follows. These modifications apply only to the bridge deck concrete for the bridge structure designated in the plans (Structure No. 52-435-289). These modifications to Section 460 of the Standard Specification for Roads and Bridges do not apply to any other structural concrete.

Delete Section 460.1 and replace with the following:

460.1 This work shall consist of falsework and form construction, the furnishing, handling, placing, finishing, and curing the bridge deck concrete (excluding barriers), grooving the bridge deck, and the construction of a test slab. The bridge deck concrete shall be Class A45 LC-HPC and shall be measured and paid for under the bid item High Performance Concrete, Bridge Deck in accordance with Sections 460.4 and 460.5 of the Standard Specifications.

Delete Section 460.2 and replace with the following:

460.2 **A. Cement:** Section 750. Type I/II Portland Cement is required for all Class A45 LC-HPC. No substitutions will be allowed.

B. Fine Aggregate: Section 800, to include that the fine aggregate when tested for Alkali-Silica Reactivity (ASR) in accordance with ASTM C1260, except that the gradation of the material used for testing shall be as produced from the source, shall have a 14 day expansion value of less than 0.25.

C. Course Aggregate: Section 820

D. Water: Section 790

E. Admixtures: Section 751 and 752

F. Reinforcing Steel: Section 1010

G. Curing Materials: Section 821. Absorptive materials are defined as burlap. Other absorptive materials such as cotton mats may be submitted to the Engineer for approval.

H. **Mineral Admixtures:** Mineral admixtures will not be permitted in Class A45 LC-HPC.

Delete Section 460.3.A and replace with the following:

460.3.A Concrete Quality and Proportion: The Contractor shall design and be responsible for the performance of the concrete used in the bridge deck. The mix proportions selected shall produce concrete that is sufficiently workable and finishable and shall conform to the following requirements:

Class of Concrete: A45 LC-HPC

Minimum Cement Content: 534 Lbs/Yd^3 (317 Kg/m^3)

Maximum Cement Content: 563 Lbs/Yd^3 (334 Kg/m^3)

Maximum Water Cement Ratio: 0.45

The mix design shall establish a maximum water/cement ratio for the concrete mix (never to exceed 0.45).

Minimum Coarse Aggregate Content: 55%

Air Content Range: 8% ± 1%

Concrete with an air content outside of the range specified above will be subject to price adjustment except that concrete with an air content less than 6.5% or greater than 9.5% shall be rejected.

Slump at Time of Placement: 1 ½" to 3" (36mm to 75mm)

Concrete with a slump outside of the range specified above will be subject to price adjustment except that concrete with a slump ≥ 4" (100mm) shall be rejected.

Minimum 28 Day Compressive Strength: 4500 psi (31 Mpa)

Total Volume of Water and Cement shall not exceed 27% of the Mix

The absolute volume method as described in the American Concrete Institute Publication 211.1 shall be used in selecting mix proportions. The mix design shall be based upon obtaining an average concrete compressive strength sufficiently above the specified minimum 28 day compressive strength so that considering the expected variability of the concrete and test procedures no more than 1 in 10 compressive strength tests will be expected to fall below the specified strength. Mix design modifications shall only be approved after trial batches to demonstrate that the adjusted mix design will result in concrete that complies with the specified concrete properties. The Engineer will allow adjustments to the dosage rate of air entraining and Type A, F and G chemical admixtures to compensate for environmental changes during placement without a new concrete mix design or trial batch.

Satisfactory performance of the proposed mix design shall be verified by laboratory tests on trial batches. Trial batches shall be conducted in accordance with the American Concrete Institute Publication 211.1, 318 and the following:

1. The slump of each mix shall be within ¾" ± (19mm ±) of the maximum specified.

2. The air content shall be within 0.5% of the maximum specified.

The results of such tests shall be furnished by the Contractor to the Engineer at the time the proposed mix design is submitted.

All mix designs and any approved modifications thereto, including changes in admixtures, shall be approved by the Engineer prior to use. Mix design data and test results shall be recorded on a DOT Form 24 and submitted to the Engineer.

NOTE- If the method of bridge deck concrete placement is to be by a concrete pump, the mix design shall be developed for such use. It will not be permitted to make mix adjustments to the approved mix design to enhance pumpability.

Delete Section 460.3.K and replace with the following:

460.3.K **Placing Concrete:** The Contractor shall give sufficient notice before starting to place concrete to permit inspection of forms, reinforcing steel, and preparation for placing. Concrete shall not be placed without the approval of the Engineer.

Before placing concrete, all sawdust, chips, and debris shall be removed from the interior of the forms. Temporary struts, stays, braces, and supports holding the forms or screed rail in alignment shall not be buried in the concrete and shall be removed when the fresh concrete has reached an elevation rendering their service unnecessary.

The slope of chutes for concrete placement shall allow concrete to flow slowly without segregation. The delivery point of the chute shall be as close as possible to the point of deposit. Chutes and spouts shall be kept clean and shall be thoroughly flushed with water before and after each run. The flush water shall be discharged outside the forms.

Free fall of concrete shall not exceed 5 feet (1.5 meters).

The bridge deck concrete shall be placed into the forms using a conveyor, concrete pump or other approved method such that the minimum specified rate-of-pour is maintained.

Accumulations of mortar upon any reinforcing steel and/or the surfaces of forms for subsequent work shall be satisfactorily removed. Care shall be exercised to not damage surrounding concrete and to not break the concrete to reinforcing steel bond at and near the surface of the concrete while cleaning the reinforcing steel as well as during all other construction activities. Dried mortar chips, dust, and debris shall not be left in the unset concrete.

The temperature of the concrete at the time of placements shall be between 50° F (10° C) and 75° F (24° C). In periods of hot or cold weather the following shall also apply:

Cold Weather Protection: During periods of cold weather, the temperature of the concrete shall be maintained within the specified temperature range by any combination of the following:

- Heating the mixing water to a maximum of 120° F (49° C).
- Heating the aggregates to a maximum of 120° F (49° C)

The surface temperature of the forms, resteel, girders, adjacent concrete, and any other surfaces that will come in contact with the fresh concrete shall be above freezing prior to placement. Artificial heating of these elements shall be done in such a manner as to not cause damage to the element.

Bridge Deck Concrete shall not be placed when the ambient air temperature is forecast by the National Weather Service to drop below 40° F (5° C) during the curing period or when there is a probability of the ambient air temperature being more than 55° F (13° C) below the temperature of the concrete during the first 24 hours after placement unless housing and heating is provided for both the deck and the girders. Do not continue concrete operations if the ambient air temperature is less than 20° F (-7° C).

In addition to the above, housing and heating will be required for bridge deck concrete placed between November 1 and April 1.

When housing and heating is required, the Contractor shall submit a cold weather concrete protection plan to the Engineer for approval a minimum of 5 days prior to the bridge deck pour. The plan shall clearly detail the enclosure to be used, including all enclosure materials, the number and type of heat source(s) to be used, the measures that will be used to assure that the enclosure is not damaged in high winds, and the name and phone number of individual responsible for monitoring temperatures. At a minimum, the enclosure shall consist of placing insulating blankets on top of the deck while enclosing the underside of the deck and freely circulating artificial heat within the enclosure. No artificial heat source shall be used which uses an open flame or which otherwise introduces carbon dioxide into the enclosure where it can come in contact with the fresh concrete.

The Contractor shall provide remote reading indoor/outdoor type thermometers for monitoring the concrete temperatures during the protection period. The number and spacing of the thermometers shall be determined by the Engineer. Thermometers shall generally be installed to measure the internal concrete temperature at a location approximately 1” (25mm) below the top surface of the bridge deck..

The insulating blankets shall be controlled such that the concrete surface temperature is maintained between 55° F (13° C) and 75° F (24° C) during curing. The area underneath the deck shall be enclosed and heated such that the temperature of the surrounding air is as close as possible to the temperature of the concrete and within the 55° F (13° C) and 75° F (24° C) temperature range.

After curing is complete, the concrete temperature shall be lowered gradually to the ambient temperature so that the temperature drop during the first 24 hours after the end of the curing period does not exceed 40° F (5° C).

Hot Weather Protection: When the ambient temperature is above 90° F (32° C), the forms, reinforcing steel, girders, adjacent concrete, and any other surfaces which will come in contact with the fresh concrete shall be cooled to below 90° F (32° C) by means of water spray or other approved methods.

During periods of hot weather, the temperature of the concrete shall be maintained within the specified temperature range by any combination of the following:

- Shading the materials storage areas and/or the production equipment.
- Cooling the coarse aggregates by sprinkling with water.
- Cooling the aggregates or water by refrigeration.
- Replacing a portion or all of the mix water with ice that is flaked or crushed to the extent that the ice will completely melt during mixing of the concrete.
- Liquid nitrogen injection.

Consolidation: The bridge deck concrete shall be consolidated by means of a mechanical device on which internal (spud or tube type) vibrators of the same type and size are mounted. A minimum of 6 vibrators shall be mounted so as to provide a maximum insertion spacing of 12” (300 mm) on centers. This spacing may not be changed without the permission of the Engineer. Operation of the vibrators shall provide 12” (300mm) longitudinal spacing of insertion points along the length of the concrete placement. The vibrators shall meet the following requirements:

Diameter of Head: 1 ¾” (44mm) to 2 ½” (64mm)
Frequency of Vibration: 8000 to 12000 vibrations per minute (under load)
Average Amplitude: 0.02” (0.6mm) to 0.05” (1.27mm)
Radius of Action: 7” (180mm)

The Contractor shall provide a copy of the manufacturer's specifications for the type and brand of vibrators being used.

The vibrators shall be mounted on the mechanical device in such a manner that they will enter the concrete vertically under the influence of their own weight, and shall be provided with adequate flexibility to work themselves around the reinforcing steel. The mechanical device may be mounted on the finish machine or upon an independent framework capable of being propelled along the screed rails. Movement of personnel through the vibrated concrete will not be permitted.

Delete Section 460.3.M.4 and replace with the following:

460.3.M

4. **Bridge Deck Finish:** The concrete shall be placed slightly higher than the finished surface of the deck. Immediately after the concrete has been placed and consolidated, the surface shall be struck off and finished with an approved finish machine with a carpet drag and metal pan mounted to the finish machine. Hand finish work behind the finish machine shall be kept to a minimum. Tining of the bridge deck concrete will not be permitted. Local irregularities may be finished with a float, roller or other approved device provided that it does not become excessive. The addition of water or pre-curing material to assist in finishing operations is not permitted. The finishing machine shall be a self-propelled rotating auger and cylinder type with one rotating cylinder. Finish machines with multiple cylinders will not be allowed. The finish machine shall span the concrete placement width. The single cylinder and auger shall spread and consolidate the concrete to the established profile by traversing the placement width, transverse to the roadway centerline. The finish machine shall be capable of forward and reverse motion under positive control, and be capable of raising the cylinder and auger to clear the concrete surface when traveling in reverse. Any modifications to the factory product will require approval by the Engineer. The portion of the bridge deck adjacent to the barriers shall be neatly finished to a true surface with a wooden float.

   After the concrete curing period, the bridge deck surface shall be tested for smoothness in accordance with SD417. The permissible longitudinal and transverse surface deviation shall be 1/8" (3mm) in 10 ft. (3 meters). Any portion of the deck showing variation from the template of more than 1/8" (3mm) shall be either ground, at the expense of the Contractor, to an elevation that will be within the allowable tolerance or be accepted under the provisions of Section 5.3 of the Standard Specifications for Roads and Bridges. Necessary grinding shall be accomplished with specially prepared circular diamond blades mounted on a horizontal shaft. Area that have been ground shall not be left smooth or polished, but shall have a uniform texture equal in roughness to the surrounding unground concrete.

Bridge Deck Grooving: The bridge deck shall be grooved following the curing period. The grooves shall be perpendicular to centerline of the deck, unless otherwise noted. On skewed bridge decks, the grooving shall not result in a "staggered" pattern at the bridge ends. The grooving shall be accomplished using a mechanized multi-blade saw capable of sawing 1/8" wide by 3/16" ($\pm$ 1/16") deep grooves. The spacing between the individual grooves shall be randomly spaced and shall vary between 5/8" and 1-5/8" with 50% of the spaces being 1 inch or less. Overlapping of the grooves shall not be allowed. The 12 inch width of the bridge deck adjacent to each curb shall be left ungrooved. Bridge Deck Grooving will be measured to the nearest 0.1 Square Yard. Bridge Deck Grooving will be paid for at the contract unit price per square yard. Payment will be full compensation for labor, equipment, tools, materials and all incidentals required to complete the work.

Additional Special Requirements for Bridge Deck Concrete:

The Class A45 LC-HPC concrete being placed and finished shall be protected from damage due to rapid evaporation. Protection from evaporation shall consist of continuous fogging during and after concrete placement until the specified curing method begins. Concrete placement will not be allowed when conditions on the bridge deck are such that the evaporation rate, as determined in Chapter 2 of the American Concrete Institute Manual of Concrete Practice 305R, is estimated to equal or exceed 0.2 lb/sq ft/hr (1 kg/sq m/h), or is predicted to exceed that rate during the course of placement. The effects of the required continuous fogging will not be considered in the estimation of the evaporation rate. Prior to placing concrete, the air temperature and humidity will be measured at a location approximately 12" (300mm) immediately above the bridge deck. Wind speed may be measured on the bridge deck or estimated using information from the nearest weather station. Concrete temperatures may be those actually measured from the previous days run or estimated from aggregate, cement and water temperatures as specified by the Engineer. When general area evaporation conditions are estimated to be above 0.2 lb/sq ft/hr (1 kg/sq m/h)**,** the Contractor may proceed by using measures such as wind breaks, cooling the concrete, etc. to create and maintain environmental conditions on the bridge deck that are satisfactory for concrete placement.

Delete Section 460.3.N and replace with the following:

460.3.N **Curing Concrete:** All newly placed bridge deck concrete shall be cured so as to prevent loss of water. Concrete curing shall begin within 10 minutes of concrete strike-off, and the surface shall not be allowed to dry from the time strike-off has occurred until the end of the curing period. If the concrete surface begins to dry before the curing materials can be applied, the surface of the concrete shall be kept moist by a fog spray that is applied so as not to damage the surface.

Curing of the bridge deck concrete shall continue uninterrupted for a minimum of 14 days. All falsework and formwork shall remain in place during the 14 day curing period. Curing shall be by the Water With Waterproof Cover Method as described below:

Water With Waterproof Cover Method: The concrete shall be kept continuously wet at all times during the curing period. Absorptive materials shall be presoaked by immersing them in water until they are saturated. Saturated absorptive materials conforming to Section 460.2.G as modified by this special provision shall cover the entire concrete surface within 10 minutes of concrete strike-off. If wet burlap is used, apply one layer within 10 minutes of concrete strike-off followed by a second layer, at right angles to and within 5 minutes of the first layer. If the materials cannot be applied because the surface of the concrete is too plastic, then the surface must be kept continuously wet with fogging, spraying, or other approved wetting technique. Do not allow the surface to dry after strike-off, or during any time during the cure period.

Fog mist shall be applied continuously from the time of concrete strike-off until the concrete is covered with saturated absorptive material in such a way as to prevent drying of the concrete surface. Maintain the fogging to produce a “gloss to semi-gloss water sheen” on the concrete surface until the curing materials are applied. Reduce fogging only if excess water accumulates on the surface and begins to run. Fogging equipment shall be mounted on the finish machine or other equipment that may immediately follow the finish machine and shall be capable of placing fog over the entire width of the placement area. Hand held fogging equipment is required to be on the project and immediately available for use, however, the use of hand held fogging equipment is restricted to use on areas that are not reachable by the mounted fogging equipment or in the event of an unanticipated malfunction of the mounted equipment. The fog spray shall be produced from nozzles that atomize the droplets and are capable of keeping a large area damp without depositing excess water. Use high pressure equipment that generates at least 1200 psi at 2.2 gpm (8.3 MPa at 8.3 L/min), or low pressure equipment having nozzles capable of supplying a maximum flow rate of 1.6 gpm (6.1 L/min). Water shall not be allowed to drip, flow, or puddle on the concrete surface during fog misting, placing of absorptive material, or at any time before the concrete has achieved final set.

The absorptive materials shall be maintained in a fully wet condition using a misting hose until the concrete has set sufficiently to allow foot traffic. At that time, soaker hoses shall be placed on the absorptive material to cover the entire concrete surface and maintain continuous saturation of all absorptive material. Running water shall be continuously applied to the soaker hoses until such time that the entire bridge deck is covered with polyethylene sheeting. Joints in the polyethylene sheeting shall be overlapped a minimum of 6” (150mm) and shall be tightly sealed with pressure sensitive tape, mastic, glue, or other approved method such that a waterproof cover is maintained over the entire concrete surface. The polyethylene sheeting shall be secured so that it will not be damaged or displaced in windy conditions. Should any portion of the waterproof cover be damaged before completion of the curing period, the damaged area shall be immediately repaired.

The Contractor will be required to inspect the concrete surface regularly to ensure that all areas remain wet for the entire 14 day curing period. During periods of high winds or adverse weather conditions, the Contractor shall perform inspections at a minimum rate of every six hours.

During the 14 day wet curing period, only foot traffic will be allowed on the bridge deck. Work to place reinforcing steel, forms for bridge barrier, or concrete for bridge barrier is allowed 3 days after the concrete is placed, provided that the curing is maintained. No stockpile of materials will be allowed on the bridge deck during the 14 day curing period.

Within 30 minutes of removing the absorptive curing materials at the end of the curing period, apply two coats of liquid membrane forming curing compound to the concrete surface as follows (Linseed oil base emulsion curing compound shall not be used):

> A power spray applicator of the proper size and design such that the applicator has sufficient capacity and the proper spray nozzles to provide a uniform application at the specified rate shall be used.
>
> Prior to and during application, the curing compound shall be thoroughly mixed to a uniform consistency in accordance with the manufacturer's recommendations.
>
> The application rate of each coat shall be as per the manufacturer's recommendations with a minimum rate of 1 gallon per 245 sq ft (1 liter per 6 sq meters).
>
> Apply the first coat of curing compound when no free water remains on the concrete surface but while the surface is still wet. If the concrete becomes dry prior to the application of the first coat of curing compound, thoroughly wet the concrete surface with water applies as a fog spray.
>
> Spray the second coat immediately after and at right angles to the first coat.
>
> Protect the curing membrane against marring for a period of at least 7 days. Give any marred or otherwise disturbed surfaces an additional coating. Should the curing membrane be subjected to continuous injury, the Engineer may require wet burlap, polyethylene sheeting, or other approved impermeable material to be applied at once.

Delete Section 460.3.O and replace with the following:

460.3.O The Class A45 LC-HPC bridge deck concrete shall be protected as outlined in Section 460.3.K as revised in this Special Provision.

Add Section 460.3.T

460.3.T **Test Slab for Bridge Decks Poured with Class A45 LC-HPC Bridge Deck:** To demonstrate the Contractor's ability to furnish, handle, place, finish and cure Class A45 LC-HPC concrete in compliance with the specifications, a test slab shall be constructed. The test slab shall be constructed using the approved mix design, and shall be cast not later than 30 day prior to the scheduled bridge deck placement. The concrete shall be supplied by the same supplier that will be furnishing the concrete for the bridge deck.

The Contractor shall cast a concrete slab of the same width and depth of the bridge deck to be placed and of a minimum length of 35 ft. (10 meters). The location of the test slab shall be proposed by the Contractor and must be approved by the Engineer.

The test slab shall be constructed with the same concrete that is to be placed in the deck and shall be finished and cured as required by the contract documents using the same personnel, methods, and equipment that the contractor intends to use for the bridge deck placement. The Contractor may use any approved concrete placement method for the trial slab, except that if a concrete pump is proposed for use to place concrete on the bridge deck pour, a concrete pump shall also be used to place concrete for the trial slab. This is required to assure that the approved mix design will result in a concrete mix suitable for concrete pump placement.

Not less that one day after placing the concrete for the trial slab, the Contractor shall remove four full depth 4" (100mm) diameter cores, one from each quadrant of the trial slab. Curing may be discontinued immediately prior to coring. Coring of the trial slab shall be witnessed by the Engineer. Immediately after coring, the cores will be forwarded to the Engineer for visual inspection of the degree of consolidation. The Engineer will retain the cores for purposes of future analysis.

Placement of Class A45 LC-HPC concrete in the bridge deck will not commence until permission is granted by the Engineer. Permission to place the bridge deck concrete will be based on the Contractor's ability to adequately place, finish and cure the concrete and on verification by the Engineer that adequate consolidation was achieved. Granting of permission to place concrete will be given or denied within 24 hours of receiving the cores from the Contractor.

Removal and disposal of the test slab, if required, shall be included in the cost of the test slab.

At the completion of the trial slab, a post-construction conference shall be held with all parties that participated in the planning and construction being present. Any problems, issues, and successes for the trial slab construction shall be discussed and recorded at the meeting. All problems and issues shall be adequately addressed before proceeding with the bridge deck placement.

Measurement for the Trial Slab will be made per each. Payment will be full compensation for all labor, equipment, tools, materials and all other items of work required in furnishing, forming, placing, finishing, curing, protecting, coring, removal, disposal, and all other items incidental to the Trial Slab.

Add Section 460.3.U

460.3.U **Frequency of Testing:** Sampling and testing by the Department will be in accordance with the Materials Manual except as modified as follows:

First Three Truckloads: Fresh (Plastic) concrete tests will be performed on the concrete from the first three truckloads of concrete for any individual concrete placement and will be used for the purposes of acceptance. Sampling of the concrete in this application will be at the beginning of the batch after 5 gallons ± of concrete has been discharged from the mixing drum.

Subsequent Truckloads: Fresh (Plastic) concrete tests for the purposes of acceptance will be performed on the concrete from all truckloads of concrete subsequent to the first three truckloads at the following frequency:

Slump:	One out of every two conveyances
Air Content:	One out of every four conveyances
Unit Weight:	One out of every four conveyances
Temperature:	Every Conveyance

Sampling of concrete for all truckloads after the first three truckloads will be in accordance with the Materials Manual.

* * * * *

Appendix B

Mix Proportions

BIRDSALL SAND & GRAVEL

A Division of Pete Lien & Sons, Inc.
P.O. Box 767 Rapid City, SD 57709-0767 PH:605-342-9250 FX:605-394-7246

AGGREGATE PLANTS: Oral, Wasta, Creston, and Blunt, SD
READY MIX CONCRETE PLANTS: Rapid City, Sturgis, Spearfish, Lead, Ft. Pierre, SD and Gillette, WY
CONCRETE PUMPING SERVICE: Based in South Dakota and Wyoming

DATE: 8/02/05

PLEASE DELIVER THE FOLLOWING PAGE(S) TO:

NAME: DAVE DAILEY

COMPANY: HEAVY CONSTRUCTORS

PHONE: ______ FAX: 342-8262

FROM:

NAME: Adam Gilliland

DIRECT PHONE LINE 939-2789

TOTAL NUMBER OF PAGES (INCLUDING COVER PAGE): 3

COMMENTS:

Mix Designs - Exit 60

SDRMCA

NATIONAL STONE, SAND & GRAVEL ASSOCIATION

Contractor Concrete Mix Design

Company Name: Birdsall Sand & Gravel

Prepared by: Adam D. Gilliland ADG

Title of Preparer: Concrete Sales Representative

Mix # (for DOT use): ______

Class of Concrete: Class A45 Concrete

Company mix reference#: 952150

MATERIALS:

		Sp. Gr.	Absorption	F.M.
Fine Aggregate (type, source):	Natural Sand, Birdsall Sand & Gravel Creston Pit	* 2.64	1.1	2.90
(state, county and legal description):	SD, Penn., T2S R12E Section 9			
Coarse Aggregate (type, source):	Limestone Ledge Rock, Pete Lien & Sons,	* 2.67	0.5	
(state, county and legal description):	SD, Penn., T2N R7E Section 20, 21			
Med. Aggregate (type, source):	N/A	*		
(state, county and legal description):				
Cement (brand, type, source):	GCC Dacotah Cement, Type I/II, Rapid City	3.15		
Fly Ash (brand, type, source):	ISG Resources, Modified F, Coal Creek, ND	2.55		
Water (source):	Birdsall Sand & Gravel	1.00		

Admixtures (brand, type): Quantity/yd^3 or oz/cwt.◊ =

- Master Builders MBVR, Air Entraining Agent — 6 oz. / CY (Approx.)
- Master Builders Pozzolith 322-N, Water Reducing Agent — 3 oz. / cwt.

◊ = of cementitios material

DESIGN MIX PROPORTIONS:

W/C Ratio: 0.42

		lb/yd^3	Absolute Volume (ft^3) – ŧ
Cement		570	2.90
Fly Ash		125	0.79
Coarse Aggregate*		1710	10.26
Fine Aggregate*%	40	1155	7.01
Water		283	4.54
Air Content		6.5%	1.76
Other:			
TOTAL		3852	27.26 (≅27ft^3)

ŧ-absolute volume = (lb. Of product)÷ [(Sp. Gr.)×(62.4)]
*-Saturated Surface Dry Basis %-enter the percent of total aggregate

TRIAL MIX TEST DATA: Attach Supporting Lab Test Documents. Aggregate: {sieve analysis, absorption, fineness modulus, LA abrasion, specific gravity, soundness, colormetric, course % particles passing 200, % particles less than 1.95 sp. gr.} Trial Batch: {mix weights, slump, air content, unit weight, actual aggregate moisture, actual w/c ratio, cylinder compressive strengths, strength gain curve}.

Comments: Project – Exit 60, Penn. Co. Structures and Pavement
IM 90-2(134)59 and P2016(13)71, PCN 4259 & 6227

Distribution: SDDOT Conc. Engr.
SDDOT Area Engr.
SDDOT Region Matls. Engr.

Contractor Concrete Mix Design

Company Name: Birdsall Sand & Gravel

Mix # (for DOT use):

Prepared by: Adam D. Gilliland ADG

Class of Concrete: Class A45 LC-HPC Bridge Deck

Title of Preparer: Concrete Sales Representative

Company mix reference#:

MATERIALS:

		Sp. Gr.	Absorption	F.M.
Fine Aggregate (type, source):	Natural Sand, Birdsall Sand & Gravel Creston Pit	* 2.62	1.1	2.85
(state, county and legal description):	SD, Penn., T2S R12E Section 9			
Coarse Aggregate (type, source):	Limestone Ledge Rock, Pete Lien & Sons,	* 2.67	0.5	
(state, county and legal description):	SD, Penn., T2N R7E Section 20, 21			
Med. Aggregate (type, source):	Limestone Ledge Rock, Pete Lien & Sons	* 2.67	0.5	
(state, county and legal description):	SD, Penn, T2N R7E Section 20, 21			
Cement (brand, type, source):	GCC Dacotah Cement, Type I/II, Rapid City	3.15		
Fly Ash (brand, type, source):	N/A			
Water (source):	Birdsall Sand & Gravel	1.00		

Admixtures (brand, type):	Quantity/yd³ or oz/cwt.◊ =
Master Builders MBVR, Air Entraining Agent	0.8 oz. / cwt
Master Builders Piolyheed 1020, Mid-Range WRA	5.0 oz. / cwt.
Master Builders Glenium 3030 NS, High-Range WRA	7.5 oz. / cwt

◊ = of cementitios material

DESIGN MIX PROPORTIONS:

W/C Ratio: 0.43

	lb/yd³	Absolute Volume (ft³) – ŧ
Cement	563	2.86
Fly Ash		
Coarse Aggregate*	1375	8.25
Medium Aggregate*	457	2.74
Fine Aggregate*% 40	1204	7.36
Water	242	3.88
Air Content	8.0%	2.16
Other:		
TOTAL	3841	27.25 (≅27ft³)

ŧ-absolute volume = (lb. Of product)÷ [(Sp. Gr.)×(62.4)]

*-Saturated Surface Dry Basis %-enter the percent of total aggregate

TRIAL MIX TEST DATA: Attach Supporting Lab Test Documents. Aggregate: (sieve analysis, absorption, fineness modulus, LA abrasion, specific gravity, soundness, colormetric, course % particles passing 200, % particles less than 1.95 sp. gr.) Trial Batch: (mix weights, slump, air content, unit weight, actual aggregate moisture, actual w/c ratio, cylinder compressive strengths, strength gain curve).

Comments: PROJECT – Exit 60 Bridge Deck – Pennington Co.
IM-P 90-2(134)59 PCN 4259 & 6227

Distribution: SDDOT Conc. Engr.
SDDOT Area Engr.
SDDOT Region Matls. Engr.

Appendix C

Pooled Fund Project Crack Survey Protocol

BRIDGE DECK SURVEY SPECIFICATION

DRAFT

1.0 DESCRIPTION.
This specification covers the procedures and requirements to perform bridge deck surveys of reinforced concrete bridge decks.

2.0 SURVEY REQUIREMENTS.

a. Pre-Survey Preparation.
(1) Prior to perfomring the crack survey, related construction documents need to be gathered to produce a scaled drawing of the bridge deck. The scale must be exactly 1 in. = 10 ft (for use with the scanning software), and the drawing only needs to include the boundaries of the deck surface.
(3) The scaled drawing should also include compass and traffic directions, deck stationing, and a scaled 5 ft by 5 ft grid on the deck.
(4) For curved bridges, the scaled drawing need not be curved, i.e., the curve may be approximated using straight lines.
(5) Coordinate with traffic control so that at least one side (or one lane) of the bridge can be closed during the time that the crack survey is being performed.

b. Preparation of Surface.
(1) After traffic has been closed, station the bridge in the longitudinal direction at ten feet intervals. The stationing shall be done as close to the centerline as possible. For curved bridges, the stationing shall follow the curve.
(2) Prior to beginning the "crack survey," mark a 5 ft by 5 ft grid using lumber crayons on the portion of the bridge closed to traffic corresponding to the grid on the scaled drawing. Measure and document any drains, repaired areas, unusual cracking, or any other items of interest.
(3) Starting with one end of the closed portion of the deck, begin tracing cracks that can be seen while bending at the waist. After beginning to trace cracks, continue to the end of the crack, even if this includes portions of the crack that were not initially seen while bending at the waist. Areas covered by sand or other debris need not be surveyed. Trace the cracks using a different color crayon than was used to mark the grid and stationing.
(4) At least one person shall check "Over the marked portion of the deck for any additional cracks. The goal is not to mark every crack on the deck, only those cracks that can initially be seen while bending at the waist.

c. Weather Limitations.
(1) Surveys are limited to days when the expected temperature during the survey will not be below 60° F .
(2) Surveys are further limited to days that are forecasted to be at least mostly sunny for a majority of the day.
(3) Regardless of the weather conditions, the bridge deck must be completely dry before the survey can begin.

3.0 BRIDGE SURVEY.

a. Crack Surveys.

Using the grid as a guide, transfer the cracks from the deck to the scaled drawing. Areas that are not surveyed should be marked on the scaled drawing. Spalls, regions of scaling, and other areas of special interest need not be included on the scale drawings but should be noted.

b. Delamination Survey.
At any time during or after the crack survey, bridge decks shall be checked for delamination. Any areas of delamination shall be noted and drawn on a separate drawing of the bridge. This second drawing need not be to scale.

c. Under Deck Survey.
Following the crack and delamination survey, the underside of the deck shall be examined and any unusual or excessive cracking noted.

Appendix D

Petrographic Evaluation of Concrete Core Cut from the Bridge Decks

EVALUATION OF CRACK-FREE BRIDGE DECKS

South Dakota DOT Project SD2005-11

Supplemental Task S1.

Perform Petrographic Analysis of Eight Concrete Cores Cut from Four Bridge Decks (two from each type)

Engineering and Mining Experiment Station
South Dakota School of Mines and Technology

Scope

The work consists of petrographic examination of eight concrete cores to determine air void and aggregate content and distribution and application of ASTM C642 to estimate density, absorption, and void content.

Materials

The eight cores (approx. 4 inch diameter) are designated as follows: NC, C5, #3 EB, #5 EB, 3C, W4, C2, and W6 #6.[1] Each core was cut on a diamond saw into two vertical half-cores. Photographs of the half-core surfaces are shown in Figure 1.

Air void content and distribution

Content of entrained and entrapped air voids was estimated using petrographic point counting. Thin sections measuring 1.5 in. x 2.5 in. were prepared from the top and bottom of each core for a total of 16 thin sections. During preparation, thin sections were vacuum-impregnated with blue epoxy to aid in the identification of air voids. Photomicrographs of portions of each thin section are shown in Figure 2. Point counts were conducted focusing on the part of the thin sections that included the top 1 inch and bottom 2 inches of each core. A minimum of 500 points were counted on each thin section.

The following features were tallied during point counting:

entrained air voids (generally spherical and less than 1 mm in diameter)
entrapped air voids (generally irregular and larger than 1 mm in diameter)
aggregate particles (all coarse and fine aggregate, undifferentiated)
paste (all paste components, undifferentiated)

Table 1 shows the estimated air void content and distribution in the samples. Separate columns show entrained air as a percentage of paste, combined entrained and entrapped air as a percentage of paste, and combined entrained and entrapped air as a percentage of paste and aggregate (i.e., all components of core). Percent total aggregate (combined coarse and fine) is also shown.

The results in Table 1 suggest that the quartzite cores tend to have slightly greater air void content and slightly greater total aggregate content than the limestone cores. There does not appear to be a significant difference in air void content in the top 1 inch versus the bottom 2 inches. The total aggregate content in the top 1 inch appears to be slightly greater than in the bottom 2 inches, regardless of coarse aggregate type.

[1] Labels on cores were difficult to read. These designations may differ from actual core identifications.

Aggregate content and distribution

Content of coarse aggregate in the top 1 inch and bottom 2 inches of each core was estimated using a modification of methods in ASTM C856 (see also St. John et al., 1998). Photographs of the cores were enlarged and printed on paper. Coarse aggregate particles were cut out of the sheets of paper and weighed. The remaining paper, which represents the paste and fine aggregate content, was also weighed and the percent coarse aggregate calculated. Coarse aggregate was identified on the basis of size (minimum dimension generally > 5 mm or 3/16 in), shape (generally angular), and rock type (e.g., limestone or quartzite). Table 2 shows the estimated coarse aggregate content and distribution in the eight cores. The method is considered to be very precise; however, the overall accuracy is limited by the relatively small sample size. The number of aggregate particles intersected on the cut surface of a single core may not adequately represent the concrete as a whole.

Accepting possible limitations of sample representativeness, the results in Table 2 suggest that the average coarse aggregate content in the cores is very close to 40% (volumetric). Core C2 appears to have the highest aggregate content (average 43.9%) and core 3C appears to have the lowest (average 36.5%). There appears to be no systematic difference between the average content in quartzite versus limestone cores (40.1% and 39.2%, respectively). There is also no significant difference in average aggregate content in the top 1 inch versus the bottom 2 inches (39.2% and 40.4%, respectively). There is some indication, however, that aggregate is concentrated in the top of the limestone cores but in the bottom of the quartzite cores.

ASTM Method C642, Standard Test Method for Density, Absorption, and Voids in Hardened Concrete

The eight cores were tested according the ASTM C642. Table 3 shows the results for the eight core samples. The procedure is summarized here.

The oven-dry mass, saturated mass after immersion, saturated mass after boiling, and immersed apparent mass are determined experimentally as outlined in ASTM C642, Standard Test Method for Density, Absorption, and Voids in Hardened Concrete.

From these data, the absorption after immersion, absorption after immersion and boiling, dry bulk density, bulk density after immersion, bulk density after immersion and boiling, apparent density, and volume of permeable pore space are calculated as follows.

Absorption after immersion (%) = $[(B-A)/A] \times 100$

Absorption after immersion and boiling (%) = $[(C-A)/A] \times 100$

Bulk density, dry = $[A/(C-D)]\cdot\rho = g_1$

Bulk density after immersion = $[B/(C-D)]\cdot\rho$

Bulk density after immersion and boiling = $[C/(C-D)]\cdot\rho$

Apparent density = $[A/(A-D)]\cdot\rho = g_2$

Volume of permeable pore space (%) = $(g_2 - g_1)/g_2 \times 100$ or $(C-A)/(C-D)/100$

where:

A = Mass of oven-dried sample in air, grams

B = Mass of surface-dry sample in air after immersion, grams

C = Mass of surface-dry sample in air after immersion and boiling, grams

D = Apparent mass of sample in water after immersion and boiling, grams

g_1 = Bulk density, dry, Mg/m^3

g_2 = Apparent density, Mg/m^3

ρ = Density of water = 1 Mg/m^3 or 1g/cm^3

The results in Table 3 indicate that the volume of permeable voids ranges from 12.47% in core #3 EB to 10.61% in cores C5 and #5 EB.

References Cited

ASTM C642-97, 1997, Standard test method for density, absorption, and voids in hardened concrete: Philadelphia.

ASTM C856-95, 1995, Standard practice for petrographic examination of hardened concrete: Philadelphia.

St. John, D.A., Poole, A.B., and Sims, I., 1998, Concrete petrography: New York, John Wiley, 474 p.

County	Description	Sample I.D.
Pennington	West Bound A45	#W4 and W6 #6
	East Bound LC-HPC	#3 EB and #5 EB
Brule	West Bound A45	NC and 3C
	East Bound LC-HPC	C2 and C5

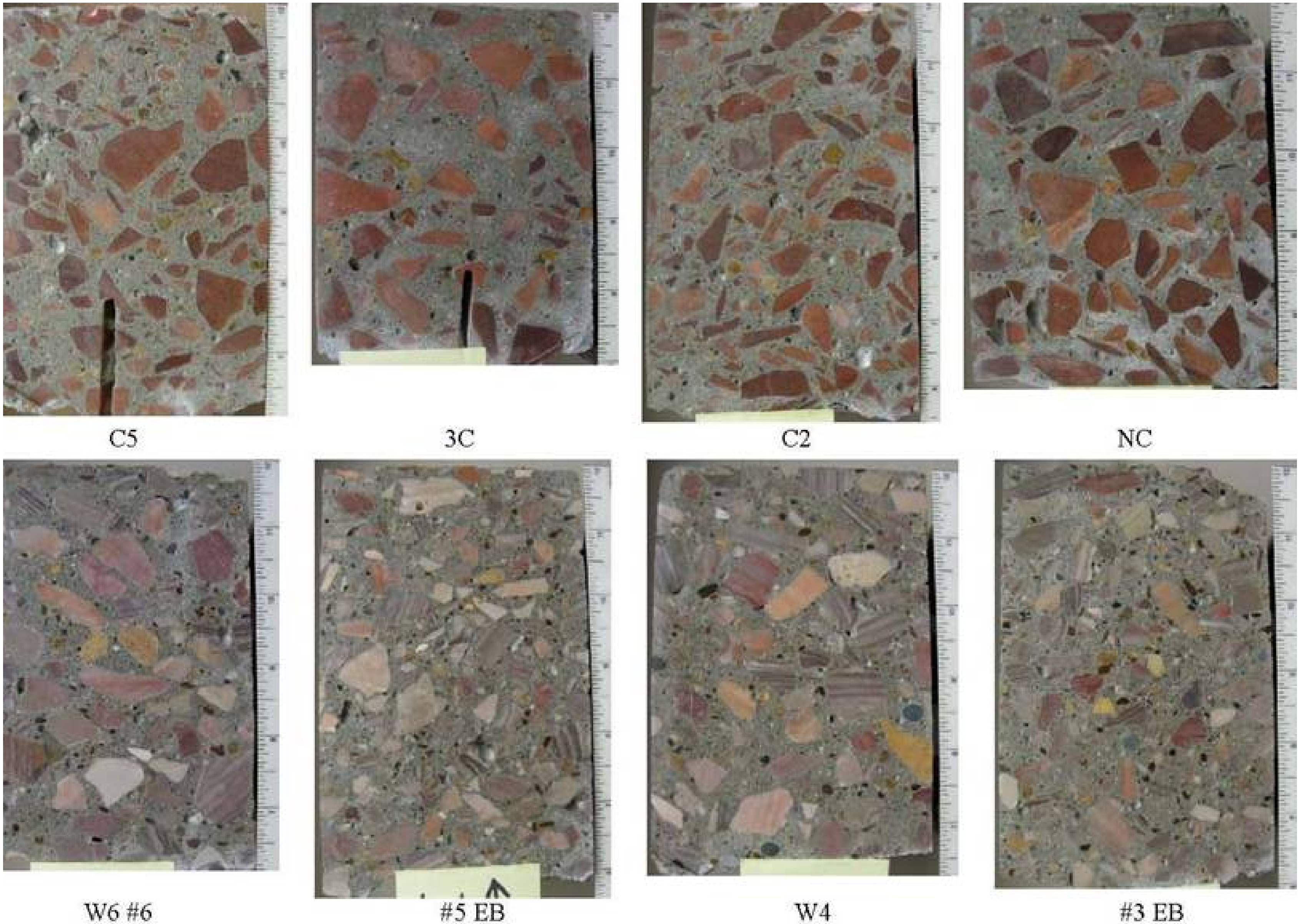


Figure 1. Photographs of cut surface of half-cores (wear surface is at top in all photos). Scale in inches at right of each core.

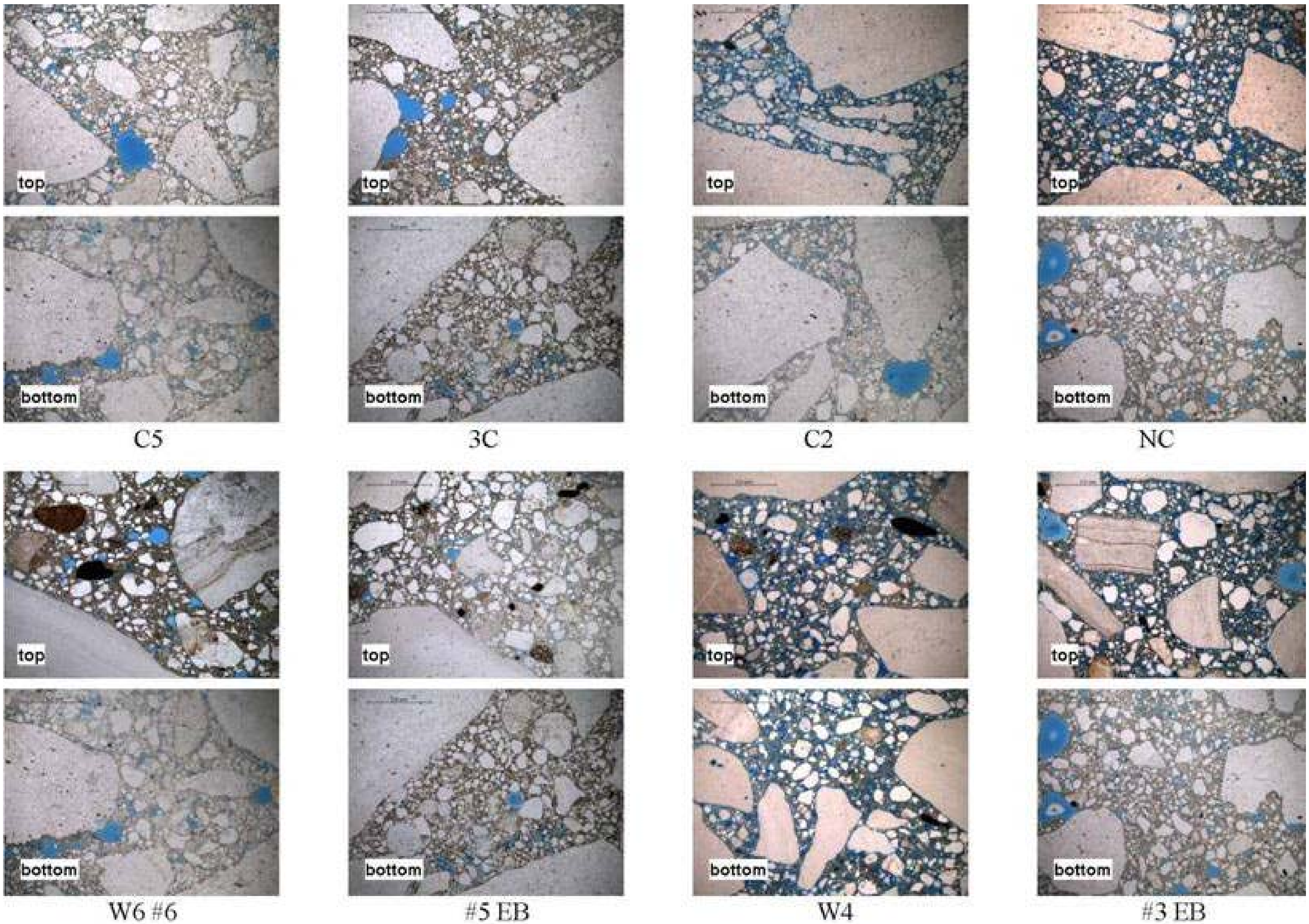


Figure 2. Photomicrographs of concrete thin sections. Blue epoxy shows air voids. Scale bar is 5 mm. Wear surface is at top in all photos.

Table 1. Air void content and distribution.

I.D.		coarse aggregate	% entrained/ (entrained+paste)	% (entrained+entrapped)/ (entrained+entrapped+paste)	% (entrained+entrapped)/ (entrained+entrapped+paste+ aggregate)	% aggregate (coarse+fine)
W6 #6	top 1 in.	limestone	8.6	12.9	4.5	65.4
	bottom 2 in.	limestone	12.0	15.9	6.1	61.6
#5 EB	top 1 in.	limestone	7.6	10.7	3.0	71.6
	bottom 2 in.	limestone	11.9	15.8	4.8	69.8
W4	top 1 in.	limestone	16.3	18.7	6.7	64.0
	bottom 2 in.	limestone	17.6	20.4	9.1	55.4
#3 EB	top 1 in.	limestone	12.0	17.4	5.8	66.9
	bottom 2 in.	limestone	3.9	8.1	2.4	70.4
C5	top 1 in.	quartzite	10.9	12.9	3.2	75.0
	bottom 2 in.	quartzite	18.2	20.0	5.4	73.1
3C	top 1 in.	quartzite	9.7	14.7	4.4	69.9
	bottom 2 in.	quartzite	14.1	19.4	6.3	67.7
NC	top 1 in.	quartzite	22.0	23.3	7.9	66.0
	bottom 2 in.	quartzite	10.4	16.7	5.8	65.0
C2	top 1 in.	quartzite	28.1	32.1	8.9	72.3
	bottom 2 in.	quartzite	15.4	20.6	7.3	64.6
		average (all)	13.7	17.5	5.7	67.4
		average limestone	11.2	15.0	5.3	65.6
		average quartzite	16.1	19.9	6.2	69.2
		average top	14.4	17.8	5.6	68.9
		average bottom	13.0	17.1	5.9	66.0
		average limestone top	11.1	14.9	5.0	67.0
		average quartzite top	17.7	20.7	6.1	70.8
		average limestone bottom	11.4	15.1	5.6	64.3
		average quartzite bottom	14.6	19.2	6.2	67.6

County	Description	Sample I.D.
Pennington	West Bound A45	#W4 and W6 #6
	East Bound LC-HPC	#3 EB and #5 EB
Brule	West Bound A45	NC and 3C
	East Bound LC-HPC	C2 and C5

Table 2. Coarse aggregate content and distribution.

I.D.		aggregate type	% aggregate
W6 #6	top 1 in.	limestone	36.9
	bottom 2 in.	limestone	45.3
#5 EB	top 1 in.	limestone	41.1
	bottom 2 in.	limestone	39.2
W4	top 1 in.	limestone	42.4
	bottom 2 in.	limestone	36.8
#3 EB	top 1 in.	limestone	40.3
	bottom 2 in.	limestone	34.3
C5	top 1 in.	quartzite	30.8
	bottom 2 in.	quartzite	44.6
3C	top 1 in.	quartzite	38.6
	bottom 2 in.	quartzite	34.4
NC	top 1 in.	quartzite	38.5
	bottom 2 in.	quartzite	45.9
C2	top 1 in.	quartzite	44.8
	bottom 2 in.	quartzite	43.1

average (all)	39.8
average limestone	39.5
average quartzite	40.1
average top	39.2
average bottom	40.4
average limestone top	40.2
average quartzite top	38.2
average limestone bottom	38.9
average quartzite bottom	42.0

County	Description	Sample I.D.
Pennington	West Bound A45	#W4 and W6 #6
	East Bound LC-HPC	#3 EB and #5 EB
Brule	West Bound A45	NC and 3C
	East Bound LC-HPC	C2 and C5

Table 3. Results of ASTM C642 Standard Test Method for Density, Absorption, and Voids in Hardened Concrete

Sample I.D.	Absorption after immersion	Absorption after immersion & boiling	Bulk density, dry, g/cm^3	Bulk density after immersion, g/cm^3	Bulk density after immersion & boiling, g/cm^3	Apparent density, g/cm^3	Volume of permeable voids
NC	4.33%	5.06%	2.26	2.36	2.38	2.55	11.37%
C5	4.09%	5.00%	2.19	2.27	2.29	2.45	10.61%
#3 EB	4.43%	5.72%	2.18	2.27	2.30	2.49	12.47%
#5 EB	4.23%	4.75%	2.19	2.29	2.30	2.45	10.61%
3C	4.29%	5.34%	2.18	2.28	2.30	2.47	11.74%
W4	4.57%	4.90%	2.24	2.34	2.35	2.52	11.11%
C2	4.37%	4.96%	2.21	2.31	2.32	2.48	10.89%
W6 #6	4.10%	5.03%	2.23	2.32	2.34	2.51	11.32%

County	Description	Sample I.D.
Pennington	West Bound A45	#W4 and W6 #6
	East Bound LC-HPC	#3 EB and #5 EB
Brule	West Bound A45	NC and 3C
	East Bound LC-HPC	C2 and C5

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