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PHASED CONSTRUCTION BRIDGES: MONITORING AND ANALYSIS FOR TRAFFIC-INDUCED VIBRATION

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16. Abstract

Phased construction is a common technique utilized to allow bridges to remain partially open to traffic throughout the construction process. The segment of the bridge deck that is constructed second cures under the effect of traffic-induced vibration transmitted from the adjacent bridge-deck segment, which is open to traffic. However, subjecting bridge decks to traffic-induced vibration during early-age curing raises concerns about the durability of the decks. The primary goal of this study is to generate a fundamental understanding of the transmission of traffic-induced vibration, the extent of degradation on phased construction bridge decks, and the impact of potential mitigation measures. In this study, the response of two phased-construction bridges in Nebraska were monitored before, during, and after the second stage of phased construction. Within 6-7 hours of the second-phase pour, the two phases of the bridges converged dynamically and began to behave as a single structure. To further understand this behavior, an experimental program was executed incorporating two phased-construction specimens and one which was constructed in a non-phased manner. The phased-constructed specimens were subjected to simulated traffic-induced vibration protocols for 0 – 12 and 7 – 12 hours from the start of the pour. Within hours of the pour, significant cracks were observed in the specimen subjected to traffic for 0 –12 hours. While cracks were similarly noted for the other phased specimen, the cracks were much less extensive and did not exceed hairline widths. No cracks were observed for the non-phased specimen. Upon further evaluation, it was concluded that the critical time window of 6-7 hours during which traffic-induced vibration has the most significant impact on deck cracking corresponds to the concrete setting time. Therefore, it is recommended that phased construction bridges close for the duration of the concrete setting time to reduce premature deterioration.

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ABSTRACT

Phased construction is a common technique utilized to allow bridges to remain partially open to traffic throughout the construction process. The segment of the bridge deck that is constructed second cures under the effect of traffic-induced vibration transmitted from the adjacent bridge-deck segment, which is open to traffic. However, subjecting bridge decks to traffic-induced vibration during early-age curing raises concerns about the durability of the decks. The primary goal of this study is to generate a fundamental understanding of the transmission of traffic-induced vibration, the extent of degradation on phased construction bridge decks, and the impact of potential mitigation measures. In this study, the response of two phased-construction bridges in Nebraska were monitored before, during, and after the second stage of phased construction. Within 6-7 hours of the second-phase pour, the two phases of the bridges converged dynamically and began to behave as a single structure. To further understand this behavior, an experimental program was executed incorporating two phased-construction specimens and one which was constructed in a non-phased manner. The phased-constructed specimens were subjected to simulated traffic-induced vibration protocols for 0 – 12 and 7 – 12 hours from the start of the pour. Within hours of the pour, significant cracks were observed in the specimen subjected to traffic for 0 –12 hours. While cracks were similarly noted for the other phased specimen, the cracks were much less extensive and did not exceed hairline widths. No cracks were observed for the non-phased specimen. Upon further evaluation, it was concluded that the critical time window of 6-7 hours during which traffic-induced vibration has the most significant impact on deck cracking corresponds to the concrete setting time. Therefore, it is recommended that phased construction bridges close for the duration of the concrete setting time to reduce premature deterioration.

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CHAPTER 1 – INTRODUCTION

1.1 BACKGROUND

The United States (US) road network is essential to facilitate the transportation of people and goods across the nation and globally, which is the backbone of the US economy. Bridges are strategic connectors in the road network, which heavily suffer from critical structural deficiencies, and need to undergo a long-overdue sweeping rehabilitation process. According to the ASCE's 2021 Infrastructure Report Card, the US has more than 617,000 bridges, and 46,154 (7.5%) of those bridges are structurally deficient. On average 178 million trips are taken daily across structurally deficient bridges in the US (ASCE 2021). Furthermore, due to the current state of deteriorating infrastructure in the region and country, the number of bridges in the state and in the country in need of replacement is expected to increase. However, the complete closure of a traffic route to allow for the construction of a new bridge is often not feasible - particularly in rural Nebraska, in which truck traffic is limited to few routes and is critical to the economic vitality of the state. Typical bridge repair and replacement projects that fully close the bridge to traffic during construction have significant social and economic costs on the surrounding communities (Manning 1981, ACI 345 2013).

Phased or staged construction for the repair or replacement of bridges has emerged as a convenient alternative to alleviate the social and economic downsides associated with full bridge closure in traditional construction practices. The typical phased construction sequence for a bridge includes two stages of construction. In the first stage, a segment of the bridge, which is known as the first phase, is closed to traffic while the traffic is fully maintained on the remaining segment of the bridge, which is known as the second phase of the bridge. After the construction of the first phase of the bridge, the traffic is re-routed from the second phase to the first phase, so the construction (i.e., the second stage of construction) can begin on the second phase, as shown in Figure 1.1. It is noted that a third phase is sometimes included, which consists of a central closure pour when the first and second phases do not share a common boundary. While this is a relatively common approach in the United States, this report focuses solely on situations involving two phases without a closure pour as this is the common approach within the state of Nebraska and many others.

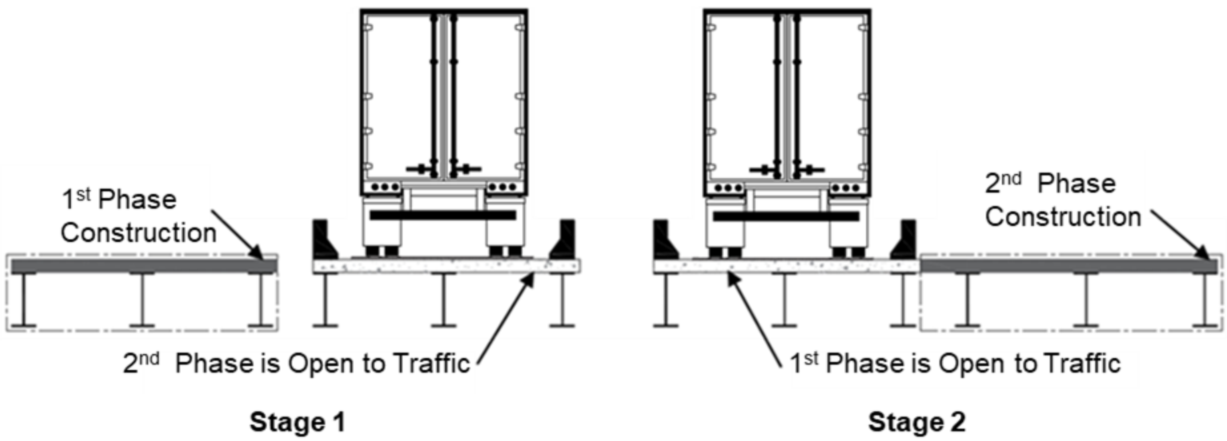


Figure 1. 1 Typical phased construction sequence of a bridge.

1.2 PROBLEM STATEMENT

Despite the practical advantages of phased construction, bridges constructed in this manner have been observed to have several constructability, serviceability, and durability issues. A common issue associated with phased construction is differential elevation, which occurs when the second phase deck does not vertically align with the first phase deck. If this differential elevation is greater than 2 inches (NDOT 2016), a closure pour is often required which extends the duration of construction and increases costs. A second issue widely associated with phased construction is premature deterioration of the second-phase deck and/or closure pour region. This premature deterioration is often evidenced by cracking of the second-phase deck along most of the span – see Figure 1.2. This extensive early-age cracking can substantially increase the costs associated with maintenance, repair, and rehabilitation over the lifetime of the bridge; and, therefore, there is a critical need to identify the causes of this early-age cracking and determine appropriate methods to mitigate premature deterioration of the deck.

There are many reasons why a concrete deck cracks, including plastic settlement, thermal shrinkage, and heavy traffic loads. However, the widespread occurrence of early-age cracking in second-phase decks where similar cracking is not observed in the first-phase decks indicates that the cause of this cracking must be directly related to the phased construction techniques. One such potential cause and the major underlying premise of this research is traffic-induced vibration, which is defined as the transfer of vibration and relative motion from the first-phase deck (which is open to traffic) to the curing concrete of the second-phase deck. This vibration can be transferred directly by the supporting formwork, transverse diaphragms (if present), the embedded reinforcing bars, and edge of the first-phase concrete. This will have two primary effects: 1) vibration of the curing concrete, and 2) deflection of the spliced reinforcing bars.

(a) (b) (c) (d)

Figure 1. 2 Evidence of cracking in second phase decks and/or closure pour regions: (a) and (b) S080-21180 2015 inspection (photos courtesy of NDOT); (c) and (d) from (Weatherer 2017).

While it is widely recognized that vibration of concrete is necessary for proper consolidation and to achieve sufficient strength, revibration or the process of vibrating concrete that was previously vibrated does not necessarily improve the performance of reinforced concrete components. Specifically, revibration is expected to improve bond strength in high-slump concrete, but it may significantly reduce bond strength in low-slump concretes (American Concrete Institute 2005), similar to mix designs used for Nebraska bridge decks. However, a more general study of bond strength due to differential deflection of rebar in curing concrete observed reductions in bond strength for deflections as low as 0.05 inch (Federal Highway

Administration 2012). In addition to impacts on the bond strength, a very recent experimental study found that prolonged revibration (e.g., 6 hours) of concrete cylinders reduced the concrete compressive strength as much as 25% and is a likely source of excessive cracking in bridge decks exposed to traffic-induced vibration (Hong and Park 2015). While it is apparent from the literature that traffic-induced vibration is a definite source of premature deterioration in phased construction, there is no clear method to mitigate this damage. This project will directly address this gap in knowledge by measuring existing levels of traffic-induced vibration in the field and directly implementing varying levels of this vibration in a laboratory experiment. Results of these experiments will provide clear guidance on how to mitigate the harmful impacts of traffic-induced vibration and enhance the durability of phased construction bridge decks.

1.3 OBJECTIVES

The primary goal of this study is to generate a fundamental understanding of the transmission of traffic-induced vibration, the extent of degradation on phased construction bridge decks, and the impact of potential mitigation measures. The specific objectives that are addressed by this research are to:

1. Synthesize the current state of practice and best practices associated with phased construction within the United States
2. Understand the characteristics of traffic-induced vibration that result in premature deterioration of concrete bridge decks in phased construction
3. Identify and recommend potential methods to mitigate deterioration due to traffic-induced vibration

1.4 REPORT OUTLINE AND SCOPE

To address the key objectives of the research, the project was sub-divided into four tasks:

1. Literature Review & Survey of State DOTs

2. Field Monitoring of Phased Construction Bridges
3. Laboratory Evaluation of Traffic-Induced Vibration
4. Synthesis and Recommendations

This report serves to outline the methods followed, results acquired, and conclusions drawn from each of these four tasks. To this end, **Chapter 1** introduces the background, problem statement, and objectives addressed by this research. The first task *Literature Review & Survey of State DOTs* is split into two chapters. **Chapter 2** summarizes the current state of knowledge associated with the premature deterioration of phased construction bridges with particular focus on the impacts of traffic-induced vibration. **Chapter 3** presents the survey that was developed to elicit the current state of practice of phased construction in the United States in addition to a synthesis of the results. The second task, *Field Monitoring of Phased Construction Bridges*, is summarized in **Chapter 4**, in which two bridges undergoing phased construction were monitored for traffic-induced vibration. **Chapter 5** presents the methods followed and results acquired during the third task, *Laboratory Evaluation of Traffic-Induced Vibration*, in which three phased construction bridge slabs were tested in the lab under varying duration of traffic-induced vibration. In conclusion, the fourth task, *Synthesis and Recommendations*, is treated in **Chapter 6**, which details the conclusions of the research and recommendations for methods to mitigate premature deterioration in phased construction bridges.

CHAPTER 2 – LITERATURE REVIEW

2.1 INTRODUCTION

Phased or staged construction for the repair or replacement of bridges is as a convenient alternative to alleviate the social and economic downsides associated with full bridge closure in traditional construction practices. The typical phased construction sequence for a bridge includes two stages of construction. In the first stage, a segment of the bridge, which is known as the first phase, is closed to traffic while the traffic is fully maintained on the remaining segment of the bridge, which is known as the second phase of the bridge. After the construction of the first phase of the bridge, the traffic is re-routed from the second phase to the first phase, so the construction (i.e., the second stage of construction) can begin on the second phase (refer to Figure 1.1). However, in the second phase of construction, the deck cures under the effect of traffic-induced vibration transmitted from the adjacent first-phase deck through reinforcement, formwork, or cross-diaphragms, which raises concerns about the structural serviceability and durability of those decks. Traffic-induced vibration causes the reinforcing bars extended from the first phase and embedded into the second phase to have differential movements during the second phase curing, which can potentially lead to accelerated degradation of the concrete-reinforcement bond in the vicinity of the construction joint (i.e., phase line) (Manning 1981, ACI 345 2013, Andrews 2013, Hong and Park 2015, Swenty and Graybeal 2012).

This chapter is intended to review the current state of knowledge regarding the impacts of traffic-induced vibration on phased construction bridge decks. The chapter is organized into three primary sections according to the study methodology. First, field inspection studies are reviewed, in which the current state of deterioration, or lack thereof, is compiled and documented for existing bridges that were constructed in a phased approach. Second, field monitoring studies are reviewed, in which the response of bridges actively undergoing phased construction is measured. Third, experimental studies are reviewed, in which the impacts of vibration and differential rebar movement are studied in a controlled laboratory environment.

The chapter concludes with a synthesis of this current knowledge so as to outline the key knowledge gaps that this research project is able to address.

2.2 BRIDGE INSPECTIONS AND OBSERVATIONS

Several studies of field inspection of existing phased-construction bridges have been conducted. Arnold et al. (1976) surveyed the performance of thirty bridges, which had been widened while maintaining traffic, for ten years since the widening of the bridges. They found that the excess water in concrete mix and low concrete cover were mainly responsible for the degradation of second-phase decks, which was attributed to traffic-induced vibration from the first-phase deck. Similar observations were made to a lesser extent in first-phase decks due to the vibration from construction activities. Accordingly, using low water-cement ratio, water reducers and adequate concrete cover was recommended for better deck performance. Montero (1980) visually inspected for cracks a bridge in Ohio after being widened while maintaining traffic, and the bridge did not show any signs of deterioration that could be definitively attributed to traffic-induced vibrations. Manning (1981) extensively reviewed the past studies of field inspections of many phased-construction bridges and visually inspected many phased-construction bridges in several states. While transverse cracking was observed in several bridges, a sample of which is shown in Figure 2.1, it was found that very few bridges exhibited degradation which could be attributed solely to traffic-induced vibrations. Manning (1981) recommended maintaining smooth riding surface and imposing traffic restrictions among other measures, to mitigate the negative impact of traffic-induced vibrations on deck performance.

Furr and Fouad (1981) visually inspected 30 bridges and only one bridge exhibited cracking and concrete spalling in the vicinity of the longitudinal joint. The joint degradation was attributed to a joint detail having the dowel bars bent 90 degrees in the horizontal plane. It was recommended that all dowel bars to extend straight from first-phase decks and be at least 24 bar diameter bars long, to avoid any deterioration of the longitudinal joints. Moreover, Furr and Fouad (1981) sampled 109 core specimens from deck areas disturbed and undisturbed by traffic of nine bridges, to be examined for deterioration through visual inspection, ultrasonic pulse velocity tests, dye tests and strength tests. The results showed that most of the core specimens from both areas had similar deterioration, therefore, the deterioration cannot be attributed solely to traffic-induced vibrations. Similarly, Deaver (1982) visually inspected 23 previously widened bridges, and none of these bridges showed any deterioration that could attributed solely to traffic-induced vibrations.

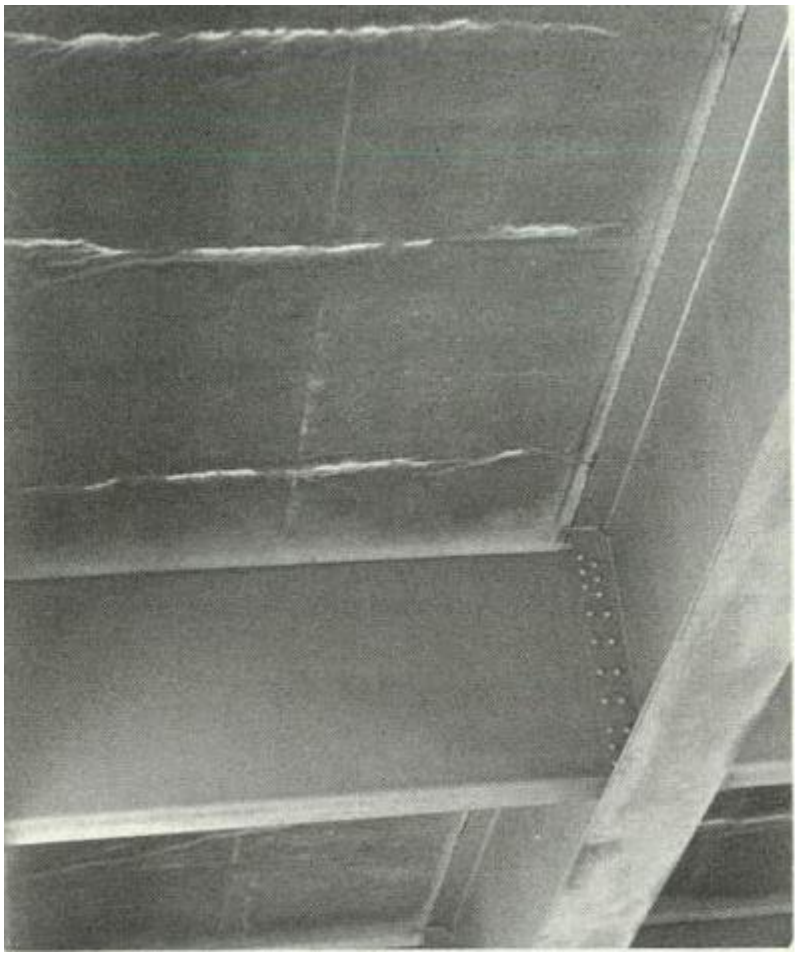


Figure 2. 1 Representative deck cracking on the underside of a deck slab (Manning 1981).

Most recently, Weatherer and Hedegaard (2019) visually inspected 41 phased-construction bridges across Wisconsin for signs of deterioration (i.e., cracks, spalls, etc.). There was no conclusive evidence that phased-construction practices cause degradation of concrete decks, as most of the deterioration signs available were highly attributed to concrete age. However, the longitudinal joint of the inspected bridges were commonly observed to be underconsolidated, likely due to the congestion in the vicinity splice, and effort should be taken to ensure proper consolidation in the vicinity of the joint. Also, no trends could be deduced between the span lengths, bridge structural systems, girder spacings, girder configurations, deck thicknesses, locations of joints between girders and longitudinal joint details, and the levels of deterioration of bridges inspected. However, 8 identical hunched slab bridges had very poor longitudinal joints, as there were very large, spalled areas exposing corroded reinforcing bars, longitudinal cracks, and delamination in the vicinity of longitudinal joints as shown in Figure 2.2. The severe deterioration was highly attributed to the shoring system used during construction, as the bridges might had experienced differential deflections between both phases decks during construction. In conclusion, several studies have looked at the performance of phased construction bridge decks in pursuit of understanding the impacts of traffic-induced vibration; however, none have been able to isolate the impacts of vibration due to confounding effects. Therefore, a review of field monitoring studies (case studies) and experimental work is included herein.

(a) (b) (c)

Figure 2. 2 Deterioration observed in past studies: (a) spalled concrete, (b) delaminated area, and (c) large longitudinal crack in vicinity of longitudinal construction joint (Weatherer and Hedegaard 2019).

2.3 FIELD MONITORING STUDIES

Field monitoring studies of phased-construction bridges have also been conducted for further interpretation of the dynamic behavior of bridges during phased construction. Furr and Fouad (1981) instrumented nine bridges across Texas using linear potentiometers attached to the bridge girders at the middle of span, to measure the maximum differential deflection occurring between the first- and second-phases closest girders to the longitudinal joints. The recorded displacements were used to compute the transverse curvature of the decks throughout the phased construction, none of the bridge decks had transverse curvature exceed the transverse curvature required to make considerable cracking. In addition, Deaver (1982) instrumented two bridges which were undergoing widenings, using linear potentiometers to quantify the absolute and relative deflections of the girders. The new decks had been separated from the existing bridge during construction, and afterwards, were connected to the existing bridges using closure pours. The dynamic differential deflections are summarized in Table 2.1 for comparison with other studies.

Table 2. 1 Differential deflections reported in past studies between girders at the phase line following the second phase deck pour

Reference	Bridge	Girder system		Span m (ft)	Girder spacing m (ft)	Max diff. deflection mm (in.)
		Stage 1	Stage 2			
Deaver (1982)	Gordon Rd. / SR 139	C-Stl	C-Stl	24.4 (80)	2.1 (7.0)	0.25 (0.010)
	Old Dixie Rd. / SR3	S-Stl	S-Stl	21.3 (70)	1.8 (6.0)	0.30 (0.012)
Furr and Fouad (1981)	I-35 / Ave. D	C-Stl	C-Stl	18.3 (60)	2.5 (8.2)	0.81 (0.032)
	I-35 / AT&SF RR	C-Stl	C-Stl	21.3 (70)	2.5 (8.1)	1.04 (0.041)
	I-45 / FM 517	C-Stl	C-Stl	16.5 (54)	2.4 (8.0)	3.05 (0.120)
	I-10 / Dell Dale- Ave.	S-PC	S-PC	26.5 (87)	2.6 (8.4)	1.52 (0.060)
	US 75 / White- Rock Creek SB	S-PC	C-Stl	15.3 (50)	1.6 (5.4)	0.81 (0.032)
	US 75 / White- Rock Creek SB	S-PC	C-Stl	27.4 (90)	1.6 (5.4)	1.47 (0.058)
	US 84 / Leon- River	O-Stl	O-Stl	20.6 (67.5)	1.9 (6.3)	1.47 (0.058)
	Texas 183 / Elm-Fork Trinity River	C-Stl	S-PC	15.3 (50)	2.0 (6.0)	1.02 (0.040)

Weatherer and Hedegaard (2019)	B-16-123	S-PC	S-PC	19.5 (64)	3.75 (12.3)	1.10 (0.043)
	B-16-136	S-PC	S-PC	18.3 (60)	2.5 (8.0)	0.96 (0.038)

Note: C = continuous; O = overhanging; PC = prestressed concrete; S = simply supported; Stl = steel.

Moreover, Weatherer and Hedegaard (2019) instrumented two phased-construction bridges in Wisconsin after the completion of the second-phase deck placement; both bridges were instrumented using a combination of tiltmeters, accelerometers and LVDTs fixed to an instrumentation arm. The differential deflections between the adjacent girders to the phase-line (i.e., longitudinal joint) were quantified and are included in Table 2.1 for comparison with the other studies. It can be interpreted that the recorded maximum differential deflections are independent from the girder type, span length and girder spacing, and no trends can be observed between these variables. However, it is worth noting that differential deflections at the phase line have been observed in excess of 0.06 inch. This indicates that there is likely substantial movement of the extended rebar within the second phase curing concrete.

2.4 LABORATORY EVALUATIONS

Several laboratory experiments have also been conducted to quantify the impacts of prolonged traffic-induced vibrations on the bond strength between concrete and reinforcing bars and compressive strength of concrete. Harsh and Darwin (1984) examined the effects of traffic-induced vibrations on the concrete-steel bond strength and concrete compressive strength for full-depth repairs of bridge decks. It was found that traffic-induced vibration does not impact bond strength when low slump concrete is used. However, traffic-induced vibration was found to reduce the bond strength when medium (4 to 5 in.) or high slump concrete is used. . Issa (1999) performed an experimental study to determine the modulus of elasticity of concrete and curvature threshold that concrete could sustain without cracking at early ages. Dunham et al. (2007) experimentally investigated the effects of induced vibrations applied at early ages, on the attainable compressive and tensile strengths of concrete. They found that vibrations did not have severe impact on the compressive strength, but slightly reduced the tensile strength of concrete. However, the vibrations were applied using soil compacters and were applied five times over 1 or 2 minutes only, in contrast to that expected for traffic-induced vibrations, which have different frequencies and are applied for longer durations (i.e., 12, or 24 or 30 hours, etc.). Kwan and Ng (2007a, 2007b) investigated the effects of traffic-induced vibrations on curing closure pours. The test specimens were subjected to double curvature loading protocol for 24 hours with amplitudes ranging from 0.02 in. to 0.20 in. The specimens were tested to quantify any degradation in the closure strip, by a reinforcing bar pullout or contraflexure strength tests. It was concluded that traffic-induced vibrations caused a significant reduction in the bond at high amplitudes of vibrations.

Swenty and Graybeal (2012) examined the effects of relative movements between the reinforcing bars and concrete during curing on bond strength in several different embedment materials, including conventional bridge deck concretes. The bond strengths of conventional bridge deck concrete specimens, which were displaced at high amplitude, were significantly reduced. This study was conducted in association with the Federal Highway Administration (2012) and the bond strength due to differential deflection of rebar in 6-inch cube specimens observed reductions in bond strength for deflections as low as 0.05 inch. Furthermore, Andrews (2013) evaluated the effects of amplitude and time sequences of applied differential movements on the bond strength, and it was found that the differential movements had the most severe impact when it was applied between the initial and final sets of the specimen concrete. Most recently, Hong and Park (2015) executed an extensive experimentation program to evaluate the effect of traffic-induced vibrations on concrete compressive and bond strengths, including 120 concrete specimens. It was concluded that traffic-induced vibrations can have negative impacts on the compressive and bond strengths of concrete. In conclusion, past experimental studies have largely been small-scale but indicate that rebar movement and vibration during the curing process negatively impacts the bond strength, compressive strength, and tensile strength of concrete. As a result, traffic-induced vibration is a distinctly likely contributor to premature degradation of second phase decks in phased construction bridges.

In addition to these small-scale tests, two experimental tests of full-scale bridge deck specimens have been conducted to evaluate traffic-induced vibration. Fouad and Furr (1981) and Weatherer et al. (2019) both constructed full-scale bridge deck specimens in a phased-construction manner in a controlled laboratory environment, where simulated traffic-induced vibration was imparted to the specimens during curing. The test setup by Weatherer et al. (2019) is shown in Figure 2.3 and these large-scale experiments mainly tested the effect of varying the amplitude of vibration on the strength and integrity of the lap splice at the construction joint (i.e., phase line). Contradictory to the expected results, Fouad and Furr (1981) and Weatherer et al. (2019) concluded that the curvatures and differential defections of real bridges are too small to cause cracks in fresh concrete. However, it is worth noting that these studies did not begin imparting vibration until after the concrete pour was complete, which neglects a critical window time that small-scale studies had identified. Despite not concluding that cracking results from the traffic-induced vibration, the specimens by Weatherer et al. (2019) were tested to failure and allowed the imprints of the rebar to be examined in the vicinity of the phase line. This is shown in Figure 2.4. In this photograph, the "stage 1 bar imprint" corresponds to the rebar extended from the phase 1 deck into the curing phase 2 deck, while the "stage 2 bar imprint" is the rebar that is placed only within the phase 2 deck. As can be seen, the "stage 1 bar imprint" is visible, but much less distinct and appears much more disturbed. While the authors were not able to confirm, this suggests that the relative motion of the rebar within the curing concrete did indeed negatively impact the bond. Therefore, while no surface cracking was observed in these experiments, it may be possible for other bridge deck configurations.

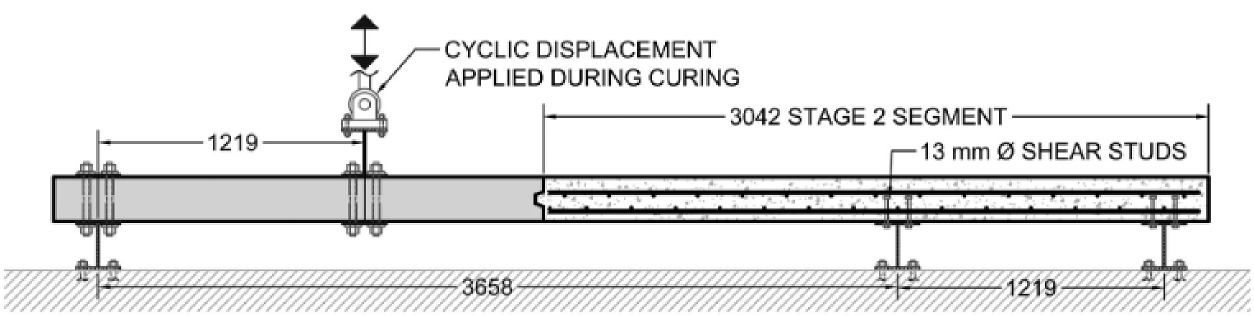


Figure 2. 3 Physical test setup by Weatherer et al (2019). Units: mm.

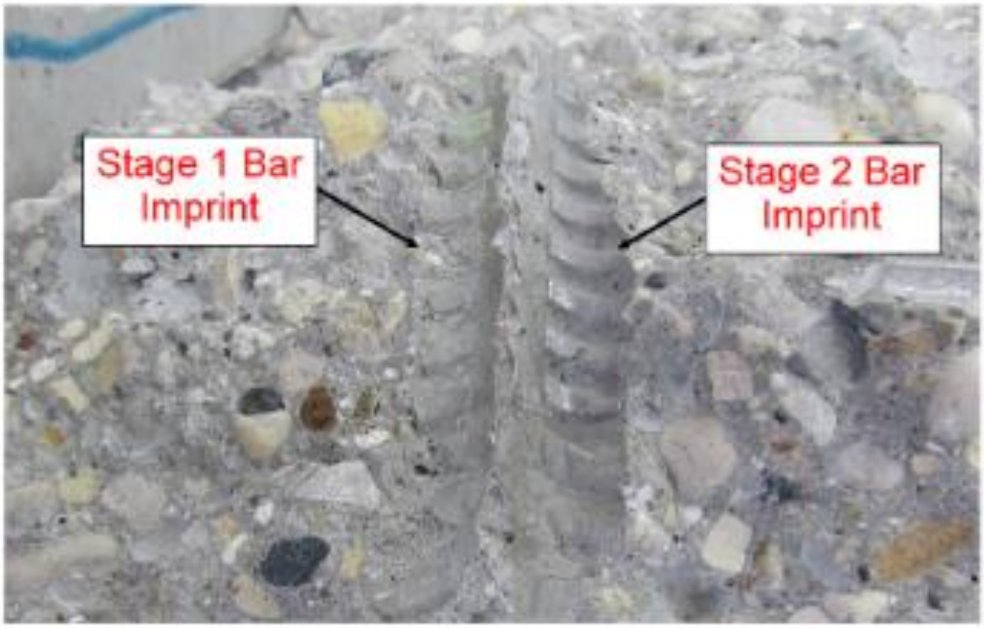


Figure 2. 4 Rebar imprints from the full-scale test of traffic-induced vibration by Weatherer et al. (2019).

2.5 SYNTHESIS AND KNOWLEDGE GAPS

Many studies have been conducted in attempts to understand the impact of traffic-induced vibration on the premature deterioration of phased construction bridge decks. Several of these studies have looked at the current state of existing phased construction bridge decks in a visual inspection approach; however, none have been able to isolate the impacts of vibration due to presence of other contributing factors. In a more detailed approach, there have been studies that monitored the response of phased construction bridges during the construction sequence. These studies have identified that differential deflections during the second phase deck pour may be exceed 0.06 inch which is a significant movement of the rebar within the curing concrete deck. However, other bridges that were monitored exhibited considerably less differential deflection and no correlations with bridge or traffic characteristics have been uncovered. To take a more controlled approach, there have been several experimental studies. These have largely been small-scale but indicate that rebar movement and induced vibration during the curing process negatively impacts the bond strength, compressive strength, and tensile strength of concrete. Significantly fewer experimental studies have been conducted at full-scale. These tests have not evidenced surface cracking as a result of traffic-induced vibration, but do provide some qualitative evidence of a reduction in the rebar-concrete bond strength. However, these tests are limited in the scenarios that they represent and questions remain for alternative phased construction bridges.

Despite the previous studies conducted to investigate the phased construction practice, several knowledge gaps remain:

1. Past field-monitoring tests were few and largely inconclusive, with primary data collection for only a limited duration of time
2. Past field monitoring tests primarily focused on the differential movement between the first and second phases without accounting for the dynamics of the entire bridge system over the construction process
3. Past large-scale experimental studies focused on varying the amplitude of the imparted vibration only without consideration of other variables such as duration of vibration or potential mitigative actions.

CHAPTER 3 – SURVEY OF DOT PRACTICE

3.1 SURVEY OBJECTIVES AND HISTORY

A survey was prepared and distributed to survey the practices and perceptions of phased construction of bridges by state departments of transportation (DOT) in the United States. The objectives of this survey were to: 1) identify construction practices associated with phased construction in the United States; 2) identify current methods used to limit premature degradation of phased construction decks; and 3) gather observations of premature degradation associated with phased construction decks. The survey was developed to gather both quantitative and qualitative information through multiple choice questions and the option for participants to provide additional written comments and/or send documentation. The survey consisted of a total of 12 questions and was in both an online and pdf format to maximize participation. The 12 questions included 9 multiple choice questions and 3 short answer questions. Questions on the survey were meant to elicit information in response to the survey's three key objectives. The full survey is provided in Appendix A.

The survey was disseminated to representatives of the state departments of transportation for all 50 states through the Subcommittee on Bridge and Structures (SCOBS) of the American Association of State Highway and Transportation Officials (AASHTO). Each SCOBS representative received an email inviting them to participate in the survey. The email was distributed in March 2020 and responses were gathered in April 2020. Of the 50 states, a total of 25 responses were received. States participating in the survey are shown in Figure 1.

3.2 OBSERVATIONS

Surveys were received by April 2020 and were subsequently analyzed by the project team. Results of the survey are presented for multiple choice questions in terms of statistics. The results are interpreted within the broader context of phased construction including any comments provided by the state DOT representative. The results of the survey are organized into four sections to address the key objectives of the survey as well as to highlight any recommendations based upon the experience of other state DOTs.

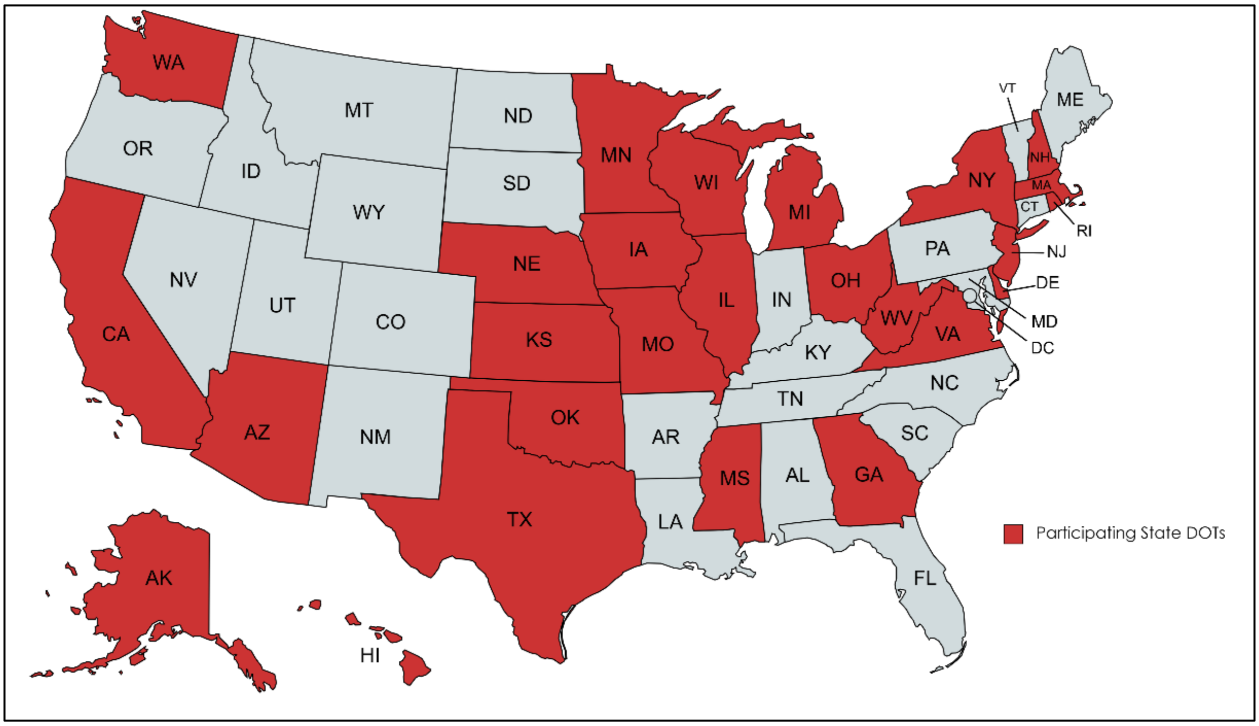


Figure 3. 1 Map of states participating in the survey of practices and perceptions regarding phased construction in the United States

3.2.1 Use and Practices of Phased Construction

Implementation and Closure Pours

Figure 3.2 – 3.7 summarize the results of the survey with respect to current uses and practices of phased construction by state DOTs. Figure 3.2 shows that the majority (72%) of DOTs often or sometimes use phased construction, while 40% of the DOTs rarely use phased construction. Figure 3.3 shows that the majority (58%) of DOTs rarely or never include a third pour (closure pour) between the two phases of the bridge. Furthermore, 20% of the DOTs include a third pour (closure pour) and 16% of the DOTs often include a third pour (closure pour). Also, 8% of the DOTs always include a third pour (closure pour). Given

that the majority of DOTs rarely or never utilize a closure pour, this research project's focus on strictly two-phase construction indicates that the findings are broadly applicable beyond Nebraska.

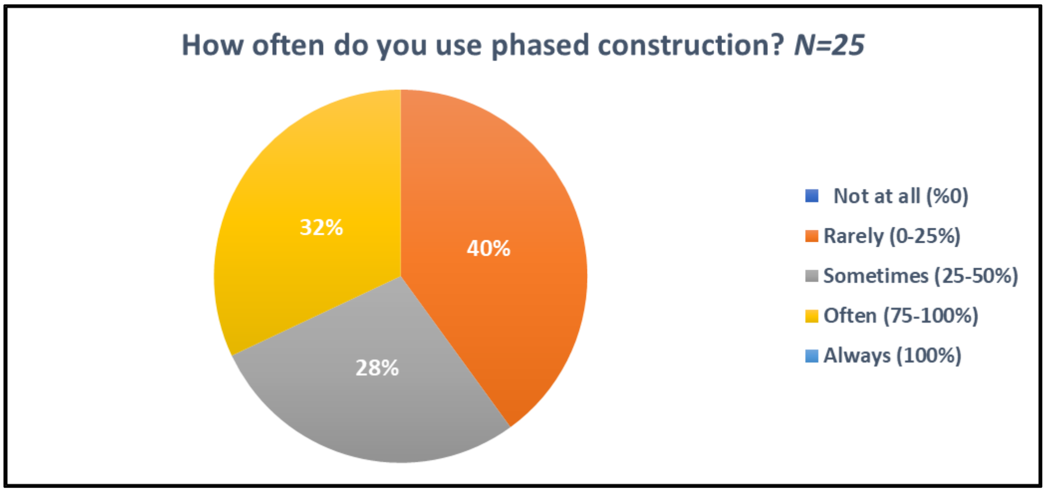


Figure 3. 2 Results of survey question #1

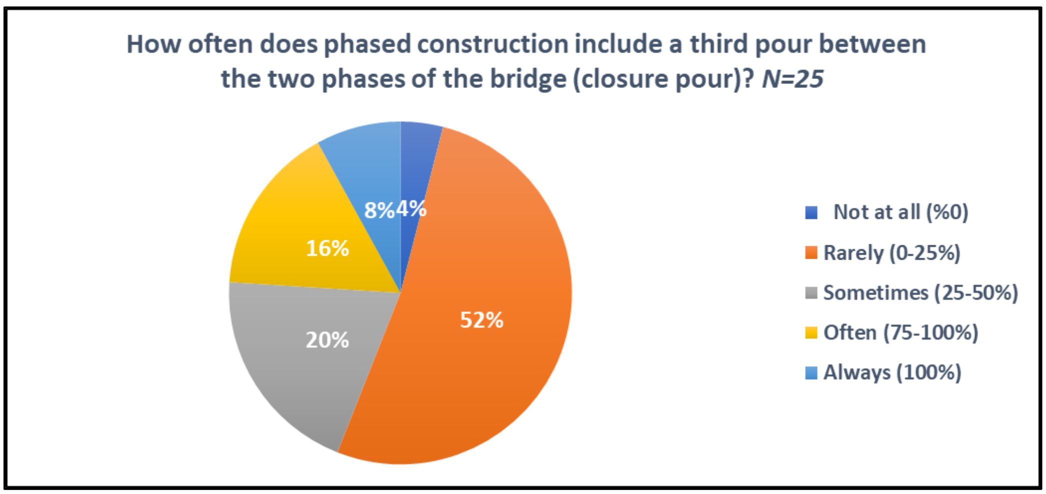


Figure 3. 3 Results of survey question #2

Curing Processes

In addition to questions surrounding the use of phased construction, information was gathered focused on varying curing processes for individual DOTs as this is another contributing source of bridge deck cracking. Figure 3.4 shows that the majority (63%) of DOTs use burlap and soaker hoses for bridge deck curing. However, 23% of the DOTs use liquid curing compounds and 14% of the DOTs use other methods of curing (i.e., cotton mat, UltraCure curing blanket or polyethylene sheeting). Similar to the findings regarding closure pours, the bridges analyzed in field monitoring tasks and in the experimental sections of this project incorporated burlap and soaker hoses for curing. Therefore, the research scope herein is broadly applicable within most state DOTs. However, Figure 3.5 shows that there is no general consensus among the DOTs on the duration of bridge deck curing. The largest percentage of DOTs (39%) have the concrete curing process for 7-10 days. Moreover, 26% of the DOTs have the concrete curing process for 10-14 days and 26% of the DOTs keep the concrete curing process for 3-7 days. A small minority (5%) of the DOTs keep the concrete curing process until the 28-day concrete compressive strength is achieved.

By analyzing the data Figures 3.4 and 3.5 together at a state-by-state level, it can be deduced that 50% of the DOTs that are using liquid curing compounds, keep the concrete curing process for 7-10 days. Moreover, 25% of the DOTs that are using liquid curing compounds, keep the concrete curing process for 3-7 days. Also, 12.5% of the DOTs that are using liquid curing compounds, keep the curing process for 10-14 days and the rest of DOTs using the liquid curing compounds, keep the curing process for 1-3 days. Furthermore, 42.86% of the DOTs that use burlap and soaker hoses (wet curing), keep the curing process for 7-10 days. In addition, 28.58% of the DOTs that that use burlap and soaker hoses (wet curing), keep the curing process for 10-14 days. However, there are 23.8% of the DOTs that use burlap and soaker hoses (wet curing), keep the curing process for 3-5 days only. 4.76% of the DOTs that use burlap and soaker hoses (wet curing) keep the curing process until the 28-day concrete compressive strength is achieved. This further emphasizes a lack of consensus in typical curing approaches.

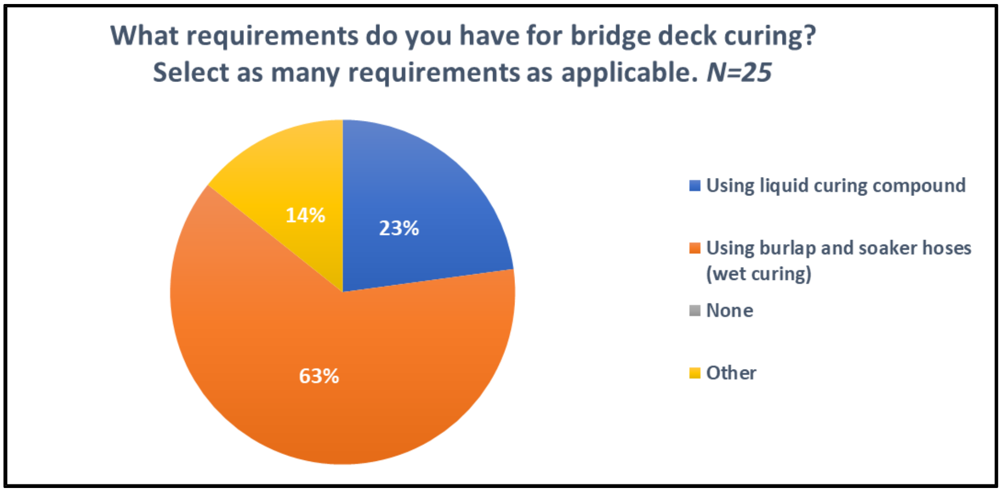


Figure 3. 4 Results of survey question #7a

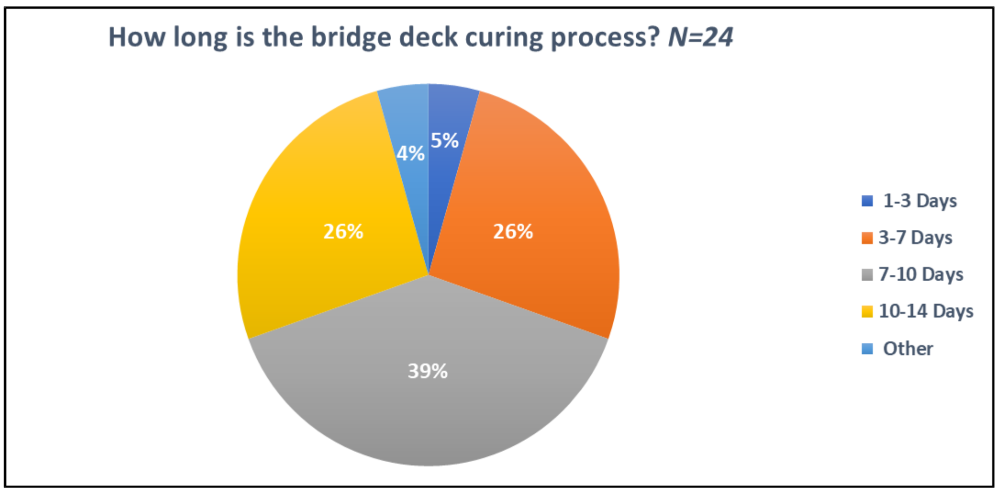


Figure 3. 5 Results of survey question #7b

Bridge and Deck Design

The design of the bridge deck and superstructure were also of interest as potential contributors to premature degradation of the deck. As a result, information was gathered associated with the rebar splicing between the two phases and the presence of transverse diaphragms. This information is presented in Figures 3.6 and 3.7. Figure 3.6 shows that the majority (56%) of DOTs tend to use lap splice for bar splicing over mechanical couplers. However, many DOTs mentioned they use mechanical couplers when the lap splice length cannot be satisfied according to AASHTO LRFD specifications. Figure 3.7 shows that there is no general consensus among the DOTs on the stage when the transverse diaphragms are connected.

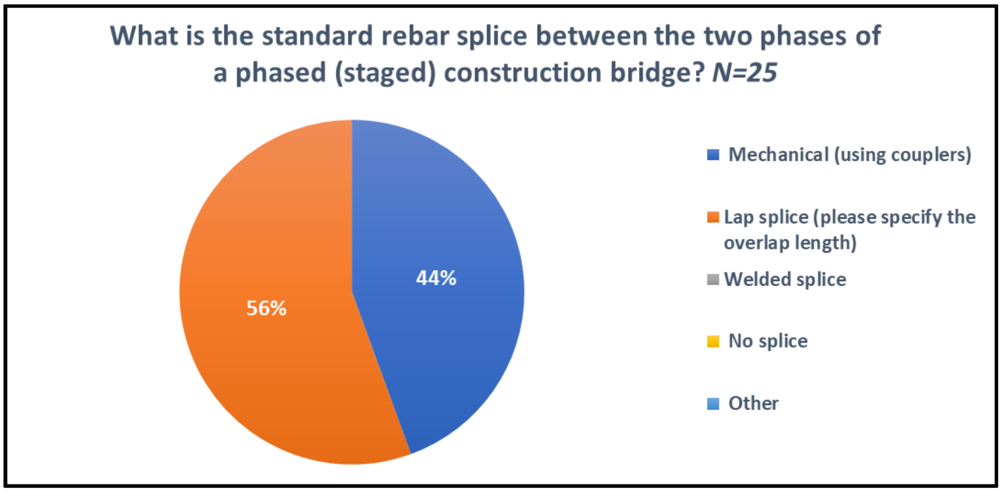


Figure 3. 6 Results of survey question #8

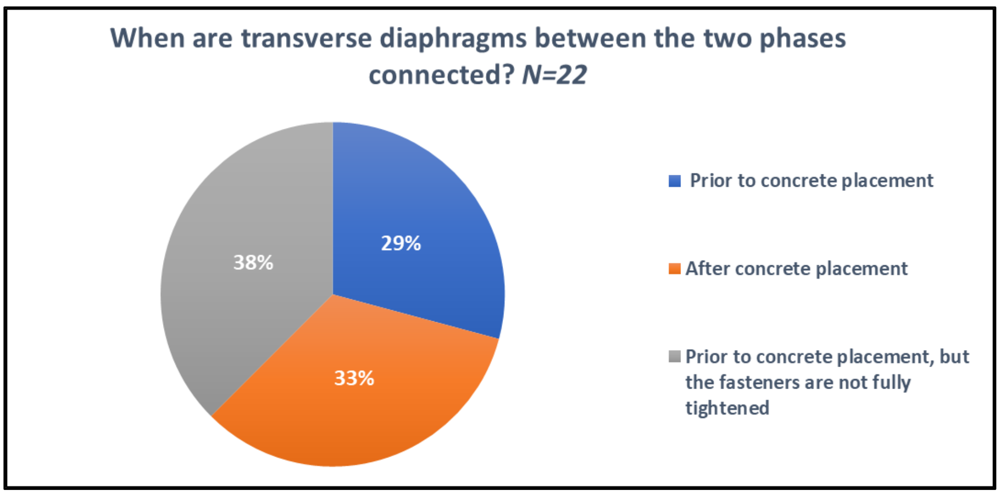


Figure 3. 7 Results of survey question #9

3.2.2 Restrictions during Phased Construction

Traffic Restrictions

The survey aimed to gather information regarding the range of traffic restrictions imposed during phased construction as a way to understand the extent of traffic-induced vibration. Figure 3.8 shows that 31% of the DOTs do not impose any traffic restrictions and the most imposed traffic restriction by the DOTs, is setting a speed limit for vehicles during concrete curing. However, some of the DOTs mentioned that the speed limit for vehicle is imposed due to safety requirements for construction sites; and other DOTs mentioned that they close the nearest lane to phase line for facilitating the concrete placement process. That is, these restrictions are in place for reasons other than reduction of traffic-induced vibration. Moreover, some DOTs impose traffic restrictions only during the concrete placement of the third pour (closure pour). In addition, one of the DOTs places traffic restrictions only in rare circumstances, such as constructing a bridge with long-span welded plate girders; due to a concern that the deflection of girders under traffic load will affect the reinforcing steel bond at the phase line.

Figure 3.9 shows that there is no general consensus among the DOTs that are imposing the traffic restrictions, on how long those restrictions are in place. Some DOTs impose restrictions during the placement of concrete only and other DOTs impose restrictions throughout a 14 day curing period or project duration. However, some DOTs keep the restrictions in place until the newly poured concrete attains certain compressive strength (i.e., 2500 psi, or full strength).

By analyzing the data Figures 3.8 and 3.9 together at a state-by-state level, it can be deduced that 55.6% of the DOTs that impose a speed limit for vehicles during concrete curing, keep it in place for more than 5 days or until concrete attains certain compressive strength. Moreover, 22.2% of the DOTs that are imposing a speed limit for vehicles, keep this limit for 1 day and the rest keep the speed limit for 12 hours or less. Furthermore, 75% of the DOTs that set a load limit for trucks during concrete curing, keep this limit for more than 5 days or until concrete attains certain compressive strength. Also, 25% of the DOTs that set a load limit for trucks during concrete curing, keep this limit for 12 hours or less. In addition, 42.8% of the DOTs that close the nearest lane to the phase line during concrete curing, close it for 3-5 days. While, 28.6%

of the DOTs that close the nearest lane to the phase line during concrete curing, close it for more than 5 days or until concrete attains certain compressive strength, and the rest of DOTs that close the nearest lane, close it for 1 day or less.

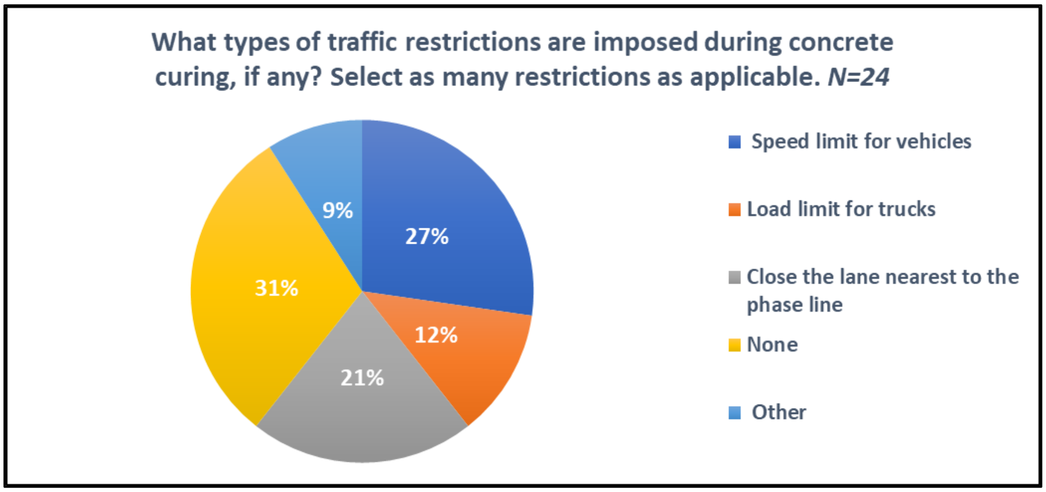


Figure 3. 8 Results of survey question #3a

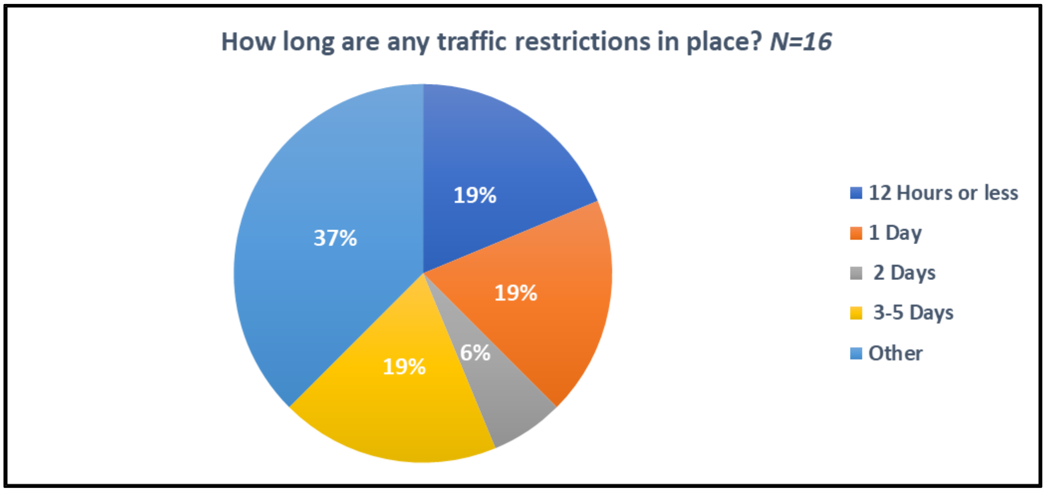


Figure 3. 9 Results of survey question #3b

Construction Restrictions

Figure 3.10 shows that the majority (64%) of DOTs impose a limit on the use of heavy equipment on the new deck for a period of time following curing. In addition, 16% of the DOTs do not impose any restrictions on construction operations during concrete curing. Figure 3.11 shows that 50% of the DOTs impose restrictions on construction operations during concrete curing for durations different than the options provided in the multiple-choice question. Comments from several DOTs indicated that these restrictions are in place for 14 days or until concrete attains certain compressive strength (i.e., 3000 psi). However, a large minority (39%) of the DOTs keep construction operations restrictions in place for 6-10 days.

By analyzing the data Figures 3.10 and 3.11 together at a state-by-state level, it can be deduced that 56.25% of the DOTs that impose limits on the use of heavy equipment on the new deck during concrete curing, keep it in place for more than 10 days or until concrete attains certain compressive strength (i.e., 3000 psi). 37.5% of the DOTs that are imposing limit on the use of heavy equipment on the new deck during concrete curing, keep this limit for 6-10 days and the rest keep this limit for 3-5 days. 50% of the DOTs that set limit on the use of heavy equipment near the phase line, keep this limit for more than 10 days or until concrete attains certain compressive strength (i.e., 3000 psi). 25% of the DOTs that limit the use of heavy equipment near phase line during concrete curing, keep this limit for 6-10 days. However, there are 25% of the DOTs that are setting a limit on the use of the heavy equipment near the phase line, keep this limit for 3-5 days only.

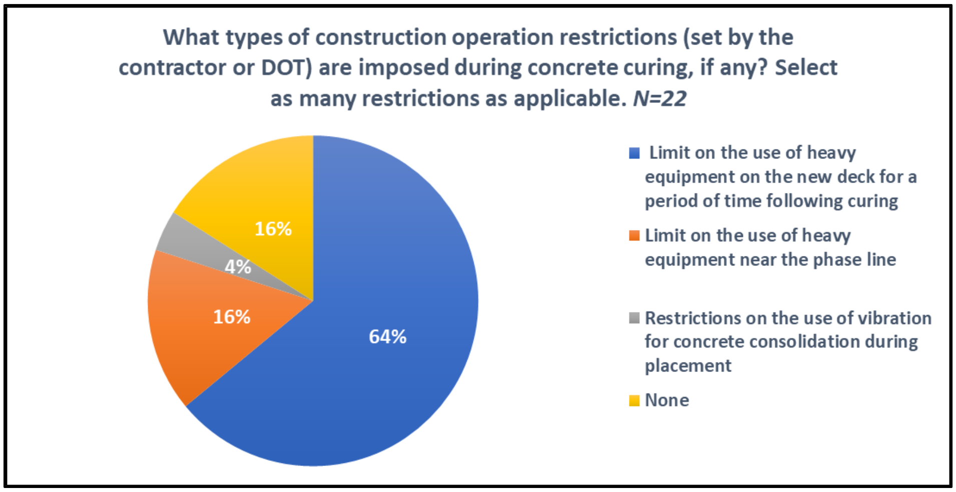


Figure 3. 10 Results of survey question #4a

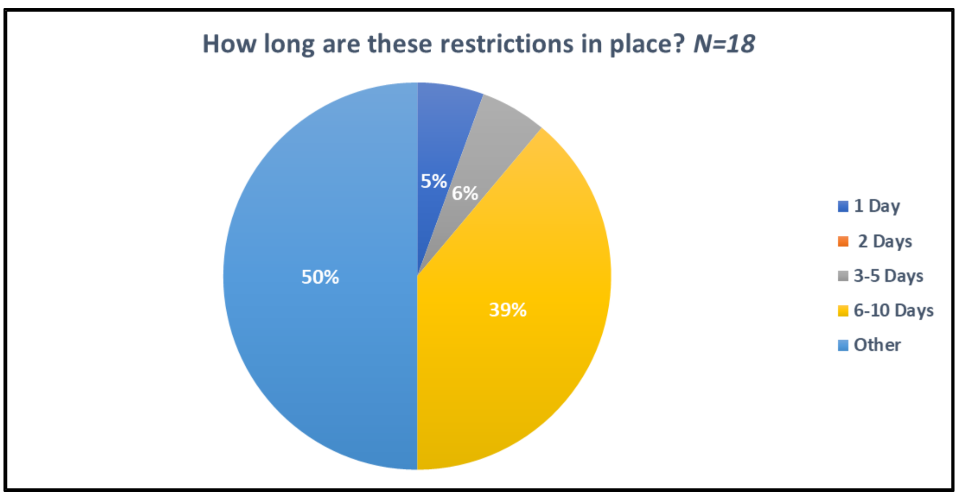


Figure 3. 11 Results of survey question #4b

3.2.3 Degradation Observations

Deck Cracking Presentation

Figure 3.12 shows that the majority (59%) of DOTs mentioned that the decks of phased construction bridges have similar cracking, as compared to the decks of non-phased bridges. Also, 36% of the DOTs mentioned that the decks of phased construction bridges have more cracking than the decks of non-phased bridges. However, 5% of the DOTs (1 state) mentioned that the decks of phased construction bridges have less cracking, as compared to the decks of non-phased bridges.

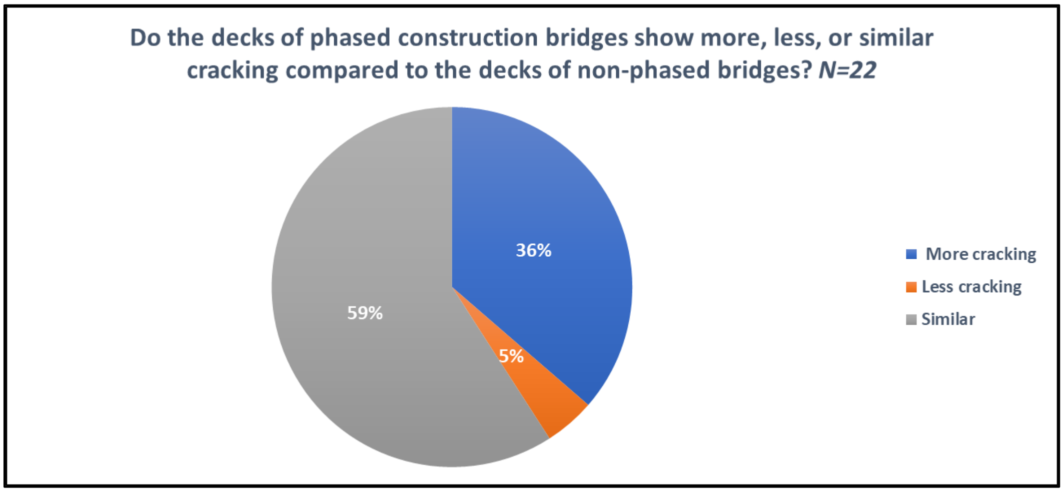


Figure 3. 12 Results of survey question #5a

By analyzing Figures 3.3 and 3.12 together at the state-by-state level, it can be deduced that 100% of the DOTs that do not include the third pour (closure pour) mentioned that the decks of phased construction bridges, show more cracking than the decks of non-phased construction bridges. Similarly, 50% of the DOTs that rarely include the closure pour mentioned that the decks of phased construction bridges, show more cracking than the decks of non-phased construction bridges. However, 50% of the DOTs that rarely include the closure pour mentioned that the decks of phased construction bridges, show similar cracking to the decks of non-phased construction bridges. Given that the phased bridges without a closure pour subject a large area to traffic-induced vibration, this data serves to further motivate the problem being studied in this project and provides some level of clarification regarding why field observation studies have had such inconclusive results.

Furthermore, 20% of the DOTs that sometimes include the closure pour mentioned that the decks of phased construction bridges show more cracking than the decks of non-phased construction bridges. On the other hand, 80% of the DOTs that sometimes include the closure pour mentioned that the decks of phased construction bridges, show similar cracking to the decks of non-phased construction bridges. 25% of the DOTs that often include the closure pour mentioned that the decks of phased construction bridges, show more cracking than the decks of non-phased construction bridges. 50% of the DOTs that often include the closure pour mentioned that the decks of phased construction bridges, show similar cracking to the decks of non-phased construction bridges; and the rest of the DOTs that often include the closure pour mentioned that the decks of phased construction bridges, show less cracking than the non-phased construction bridges. Finally, 100% of the DOTs that always include closure pour mentioned that the decks of phased construction bridges, show similar cracking to the decks of non-phased construction bridges.

Figure 3.13 shows that 50% of the DOTs mentioned that phase 2 deck (second pour) show the most cracking of the whole bridge deck, but one of the DOTs mentioned that the closure pour shows the most cracking, if it is used. However, 22% of the DOTs mentioned that phase 1 deck (first pour) have the most cracking of the whole bridge deck. By analyzing Figures 3.3 and 3.14 together at the state-by-state level, it can be deduced that 100% of the DOTs that do not include the third pour (closure pour) mentioned that phase 2 deck, show more cracking than phase. Moreover, 77.8% of the DOTs that rarely include the closure pour mentioned that phase 2 decks, show more cracking than phase 1 and closure pour decks. However, 11.1% of the DOTs that rarely include the closure pour mentioned that phase 1 decks, show more cracking than phase 2 and closure pour decks; but the rest of the DOTs that rarely include the closure pour mentioned that closure pour decks, show more cracking than phase 1 and 2 decks. Furthermore, 66.7% of the DOTs that sometimes include the closure pour mentioned that closure pour decks, show more cracking than phase 1 and 2 decks; but the rest of the DOTs that sometimes include the closure pour mentioned that phase 1 decks, show more cracking than phase 2 and closure pour decks. Also, 66.7% of the DOTs that often include the closure pour mentioned that phase 1 decks, show more cracking than phase 2 and closure pour decks; but the rest of DOTs that often include the closure pour mentioned that closure pour decks, show more cracking than phase 1 and 2 decks.

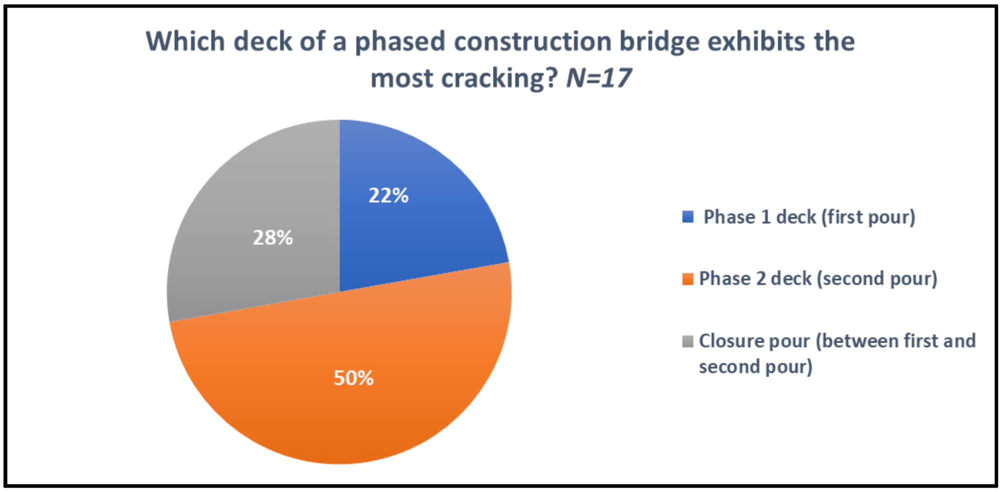


Figure 3. 13 Results of survey question #5b

Figure 3.14 shows that the biggest percentage of DOTs (35%) are not sure of when cracks are initially observed after the bridge pour. However, 31% of the DOTs mentioned that cracks are initially observed during 1-3 months after the bridge deck pour. Also, 26% of the DOTs initially observed the cracks within 2 weeks after the bridge pour. Some DOTs mentioned that they are unable to determine when cracks initially occur; as they are covered by membrane and pavement.

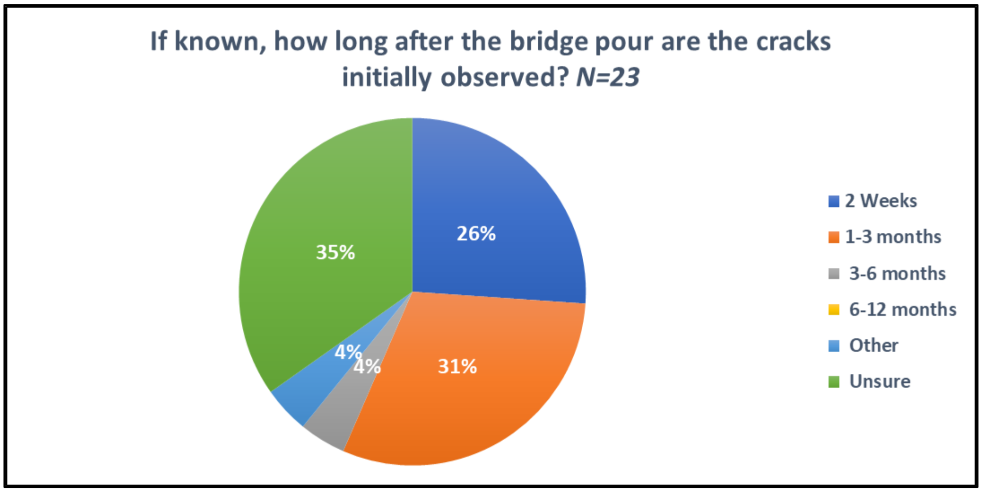


Figure 3. 14 Results of survey question #5c

Surface Treatment

Figure 3.15 shows that the majority (56%) of DOTs leave the deck surface at the phase line joint untreated and exposed. However, 36% of the DOTs seal the deck surface at the phase line and 8% of the DOTs overlay the deck surface of the joint at the phase line with membrane and waterproofing, or wearing surface.

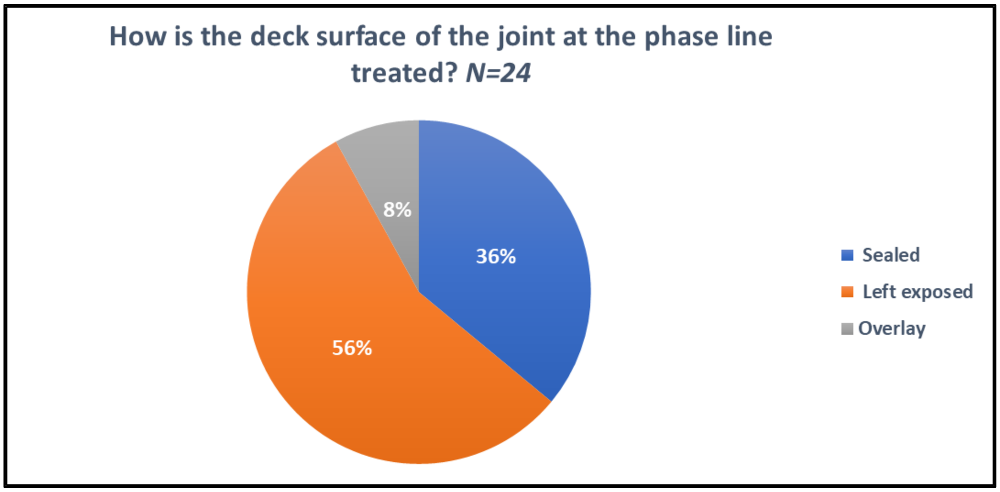


Figure 3. 15 Results of survey question #5d

Observations of Correlations by DOTs

Figure 3.16 shows that the majority (79%) of DOTs mentioned that there is no correlation between the bridge geometry and structural system, or rebar splicing method and the degradation of bridge decks. However, 21% of the DOTs mentioned that there is a correlation between the bridge structural and geometric characteristics and the degradation of bridge decks. Three of the DOTs that responded to this question mentioned that steel bridges have more deck degradation than other bridge types. One DOT mentioned that the dead load deflection of steel bridges cannot be controlled nor the steel girders can be tied, when the spans exceed 150 feet. Another DOT mentioned that the bridge skewness contributes to the deck degradation. In a more unique note, one DOT mentioned that more cracks are observed in bridges using PPC Bulb-Tee beams and high performance concrete and these issues are currently being researched. One DOT also mentioned that unsealed joints result in premature deterioration of the bay containing the phase line. Another DOT mentioned that the bridges using prestressed beams do not deflect a lot; hence those bridges have less cracking.

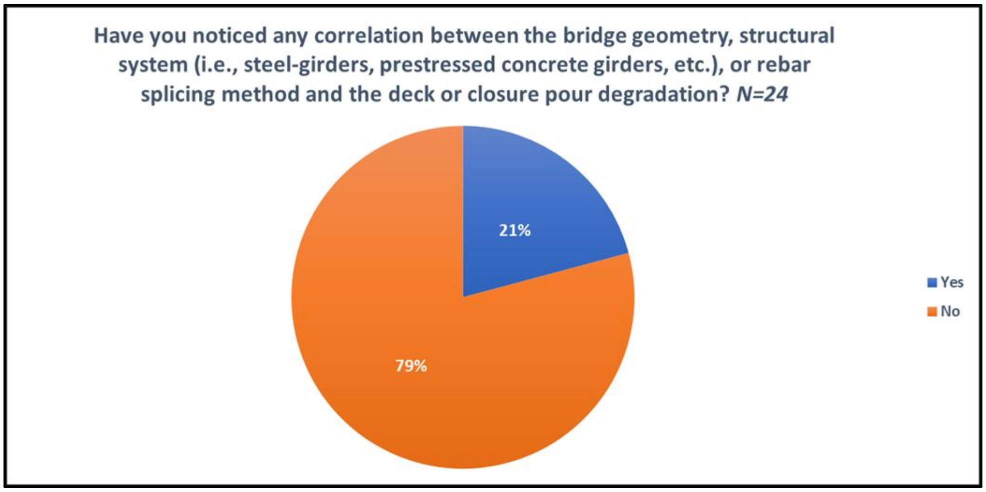


Figure 3. 16 Results of survey question #6

3.2.4 Summary and Recommendations

A key finding of this survey is that the practices in Nebraska are largely similar to those in many other states, which renders this research broadly applicable beyond state borders. In addition, research on traffic-induced vibration in phased construction is further motivated by the results of this survey. Specifically, the large majority of DOTs that utilize phased construction regularly and rarely or never specify closure pours tended to observe more significant cracking in phased construction bridge decks than other DOTs. However, by analyzing Figure 3.2 – 3.16, no clear relations can be drawn between the trend of cracking and degradation of the decks of the phased construction bridges and the different measures the DOTs apply during and after the construction of phased-bridges (i.e., traffic and construction operation restrictions, method of curing, including third pour (closure pour), concrete curing procedure, etc.).

Several DOTs provided recommendations for enhancing the durability of phased construction bridges and reducing the sources of deck degradation. These recommendations are summarized here:

- One DOT mentioned to remove the traffic for at least 24 hours from the adjacent lanes after concrete placement and to place a drip edge in the underside of the deck near the phase line to protect beams

- Others reduce the usage of phased construction bridges as much as possible, and to ensure the closure pour is over a beam, with a minimum width of 3 feet.
- Others recommend the use of elastomeric or polymer fibers or nonmetallic fibers in concrete in the closure pour and deck when there is a concern of cracking or deflection differences.
- Others require the use of shrinkage reduction chemical admixture for deck concrete and proper combination of water and curing compound.
- Others recommended to keep the phases of bridge separated as much as possible during concrete placement of the second phase, by completing the installation of diaphragms until after the curing of the second phase.
- Others recommend installing the stage 2 portion of mechanical connectors and to not tie rebar laps until after the curing, which is in an effort to reduce the cracking.
- Another DOT recommends to eliminate the keyway detail along the longitudinal joint (phase line), to have a better consolidation of concrete along joint interface and to use sealant (i.e., methacrylate) along the longitudinal joint.
- Another DOT recommends scheduling a healer/sealer or epoxy overlay a few years after completing a phased deck.

CHAPTER 4 – FIELD MONITORING OF PHASED CONSTRUCTION BRIDGES

4.1 OVERVIEW

The goal of this task is to quantify the characteristics of traffic-induced vibration for typical Nebraska bridges and construction practice. Specific information to be determined includes the amplitude, frequency, and duration of traffic-induced vibration throughout the construction process. It is well understood that the stiffness of the curing deck will significantly increase from the time of pour until the time when the deck is opened to traffic. For this reason, the vibration characteristics are anticipated to vary over the construction time and continuous monitoring beginning before the pour and the beginning of curing was sought. Two phased-construction bridges in Nebraska were monitored at the field before, during and after the two stages of phased construction. System identification and signal processing techniques were applied, to analyze the field-recorded data and closely monitor the changes in the dynamic characteristics of both bridges throughout the phased-construction stages. For further interpretation of the transmission of vibrations from the first-phase to the second-phase decks due to traffic events or construction operations throughout and after the phased-construction stages, the maximum bridge responses (i.e., accelerations and displacements) were quantified at different locations across the bridges.

4.2 BRIDGE SITES

Two bridges were identified for monitoring, which were undergoing phased construction. These bridges were identified in consultation with the NDOT Technical Advisory Committee for active phased construction during Summer and Fall 2019. The bridges are described in detail in the following sub-sections: Brunswick Viaduct (S014 17044) and Silver Creek (S030 35969). The Brunswick Viaduct bridge is referred to herein as Bridge 1 and was undergoing a deck replacement project. The Silver Creek bridge is referred to herein as Bridge 2 and was a bridge replacement project.

4.2.1 Bridge 1: Brunswick Viaduct (S014 17044)

The first bridge monitored was the Brunswick Viaduct Bridge, Bridge 1, with structure ID S014 17044 and Control Number: 32229. This bridge is located near Brunswick, NE, at lat/lon of 42.34039, -98.028995 on State Highway 14 over a railroad crossing. This is a 3-span continuous steel girder bridge where the first and third spans are 33.67 ft and the central span is 44.17 ft. The bridge underwent a deck replacement in summer 2019. Traffic during the phased construction was subject to a reduced speed limit of 45 mph and was limited to one lane only in alternating directions. Details regarding the construction phasing are provided in Figure 4.1 with corresponding dates in Table 4.1. An aerial photograph of the bridge site just prior to the asphalt overlay of the deck replacement is included in Figure 4.2.

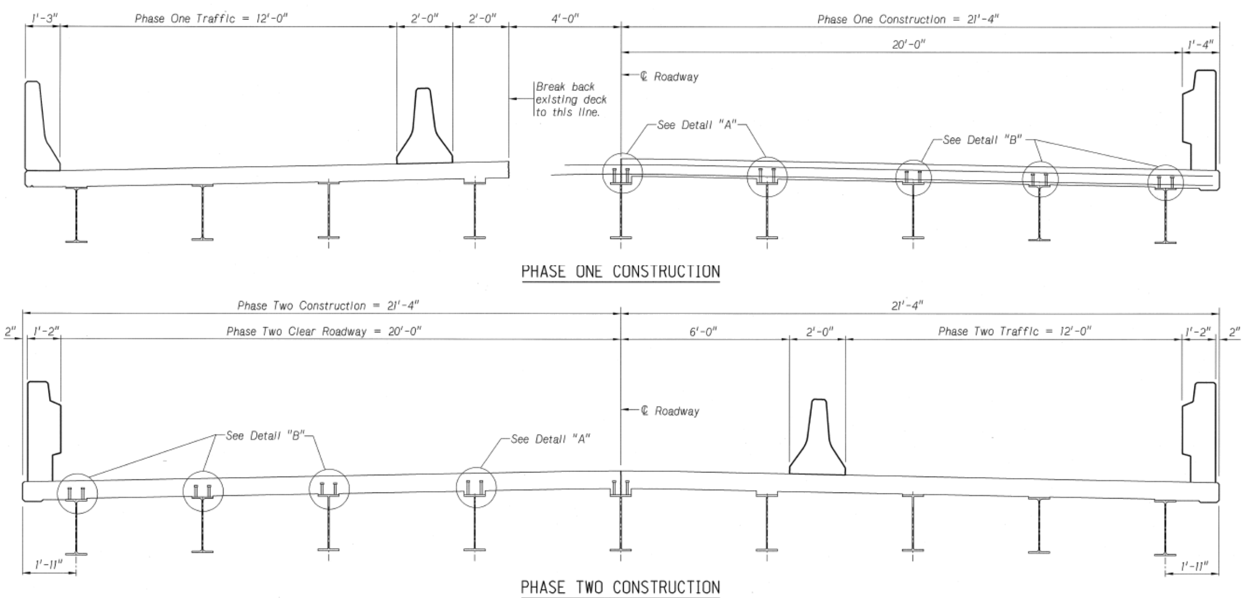


Figure 4. 1 Phasing of the deck replacement at Brunswick Viaduct Bridge (Bridge 1)

Table 4. 1 Dates of phased construction activities for Brunswick Viaduct Bridge (Bridge 1)

Activity	Date
Phase 1 Deck Pour	6/18/2019
Phase 1 Deck Curing (Site Visit)	7/20/2019
Phase 2 Deck Pour	8/29/2019
Phase 2 Deck Curing (Site Visit)	9/21/2019
Asphalt Overlay	9/23/2019

Figure 4. 2 Aerial photo of the Brunswick Viaduct Bridge (Bridge 1) after deck replacement and prior to asphalt overlay

4.2.2 Bridge 2: Silver Creek (S030 35969)

The second bridge monitored was the Silver Creek Bridge, Bridge 2, with structure ID S030 36969 and Control Number: 42745. This bridge is located at lat/lon of 41.3113, -97.6735 just southwest of Silver Creek, NE, on State Highway 30 over water. This was originally a continuous steel girder bridge, but was replaced as a 2-span simply-supported prestressed inverted tee bridge in Fall 2019. Both spans of the new bridge are 65 ft. Similar to Bridge 1, a reduced speed limit of 45 mph was posted; however, 2 lanes of traffic remained open throughout the construction unlike at Bridge 1. Details regarding the construction phasing are provided in Figure 4.3 with corresponding dates in Table 4.2. An aerial photograph of the bridge site just prior to the phase 2 deck pour is included in Figure 4.4.

Table 4. 2 Dates of phased construction activities for Silver Creek Bridge (Bridge 2)

Activity	Date
Phase 1 Deck Pour	8/9/2019
Phase 1 Deck Curing (Site Visit)	9/21/2019
Phase 2 Deck Pour	11/15/2019
Phase 2 Deck Curing (Site Visit)	11/21/2019

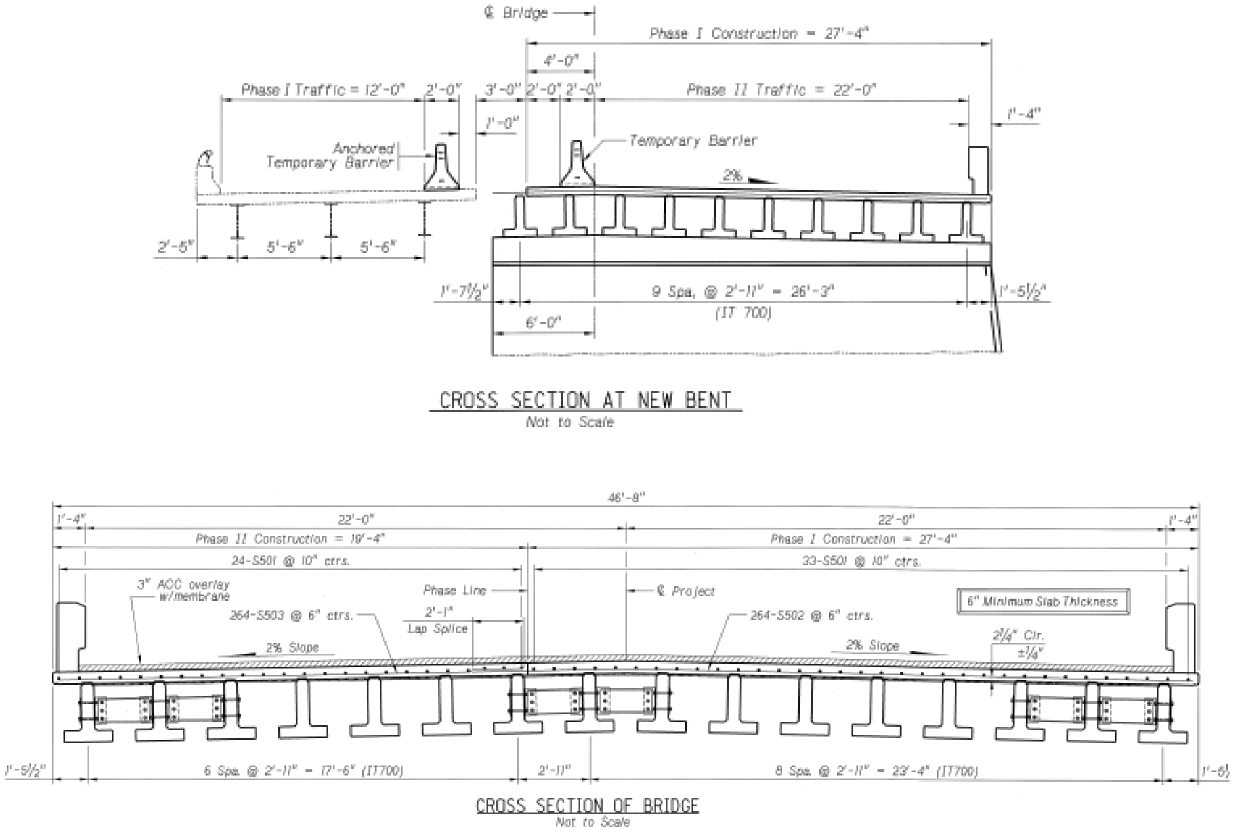


Figure 4. 3 Construction phasing at Silver Creek Bridge (Bridge 2)

Figure 4. 4 Aerial photo of phase 2 deck pour at Silver Creek Bridge (Bridge 2)

4.3 MONITORING PLAN

Bridge 1 and Bridge 2 were monitored before, during and after phased-construction stages to quantify the impact of traffic-induced vibrations. A total of 7 field monitoring tests were conducted, as summarized in Table 4.3. Note that the test ID begins with a descriptor for the construction stage of the bridge when the test was conducted (i.e.,"T1" for the deck pour of the 1^{st} phase), followed by the bridge number (i.e., "BR1" for bridge 1 or "BR2" for bridge 2). The test durations presented in Table 4.3 are corresponding to the lengths of data considered in data analysis and they are not necessarily the actual durations of data collected.

Table 4. 3 Summary of field monitoring tests and durations

Test ID	Construction Stage	Bridge 1		Bridge 2	
		Duration (Hrs.)	Date	Duration (Hrs.)	Date
T1-BR1/BR2	Deck pour of the 1^{st} Phase	1.5	6/18/19	2.75	8/9/19
T2-BR1	Before the deck pour of the 2^{nd} Phase	1.25	8/26/19	-	-
T3-BR1/BR2	Deck pour of the 2^{nd} Phase	8.8	8/29/19	8.71	11/15/19
T4-BR1/BR2	Deck curing of the 2^{nd} Phase	0.62	9/6/19	1.16	11/21/19

Note: BR1= bridge 1; and BR2= bridge 2 (i.e., T1-BR1 refers to the test of bridge 1 conducted at the deck pour of the 1^{st} phase construction stage)

During each test, the acceleration at various positions along the bridge was monitored. The acceleration was monitored on girders, on the deck, and on rebar to fully characterize the dynamics of the structure as well as the transfer of vibration from one phase to another. High-sensitivity accelerometers were incorporated for this task and utilized at a sampling frequency of 2048 Hz to ensure that the dynamic characteristics of the structure could be captured. Given that the sensors to be placed on rebar would be sacrificial, different models of accelerometers were utilized in this field monitoring campaign to balance high quality data with cost effectiveness. PCB 393B04 piezoelectric accelerometers were used to instrument girders and deck, as these are non-sacrificial, as well as to instrument reinforcing bars before concrete pouring. The PCB 393B04 sensors have a sensitivity of (±10%) 1000 mV/g, measurement range of ±5g, and frequency range of (±5%) 0.06 – 450 Hz. PCB 603C01 and PCB 352C34 piezoelectric accelerometers were used as sacrificial sensors to be embedded in concrete to monitor the vibration of the rebar. These models have a sensitivity of (±5%) 100 mV/g, measurement range of ±50g, and frequency range of (±5%) 0.5 Hz – 10 kHz. The sensors were attached to concrete girders using epoxy and to steel girders and deck surfaces using magnets. When the sensors were attached to rebar, a combination of epoxy and clamps were utilized. In addition, the rebar sensors were prepared to be embedded in concrete, by tightly wrapping a waterproofing tape around the sensors, as shown in Figure 4.5.

The sensor layouts for each test and for each bridge are provided in Figures 4.6 – 4.11. In each layout, only the instrumented spans are shown so that they are presented in a detailed view. As shown, both phases of construction were instrumented during each test. Sensors were only included on the rebar when associated with tests on the second phase deck pour. The sensor placement aimed to focus data collection on a cross-section of the bridge so that the measurements could be used to understand the transfer of vibration from one phase to another as well as to understand the general dynamic characteristics of the bridge, which are expected to vary more significantly in this direction. While larger signal-to-noise ratios would be expected for sensor placement closer to mid-span, placement was limited by water and height access limitations at each site.

(a) (b) (c)

Figure 4. 5 (a) PCB 603C01 sensor; (b) waterproofed sensor fixed to rebar; and, (c) protected sensors fixed to rebar for T3-BR2.

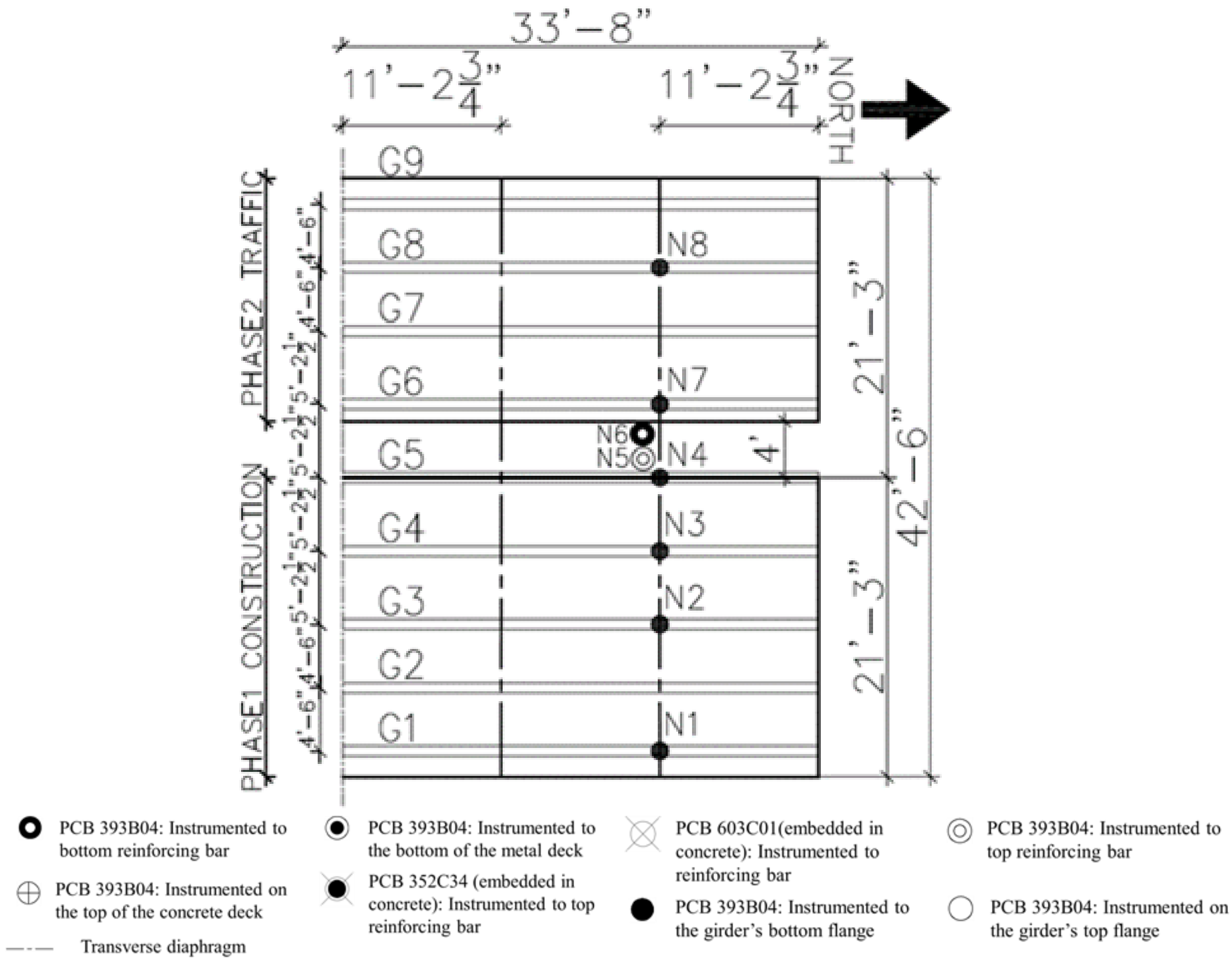


Figure 4. 6 Instrumentation layout for test T1-BR1

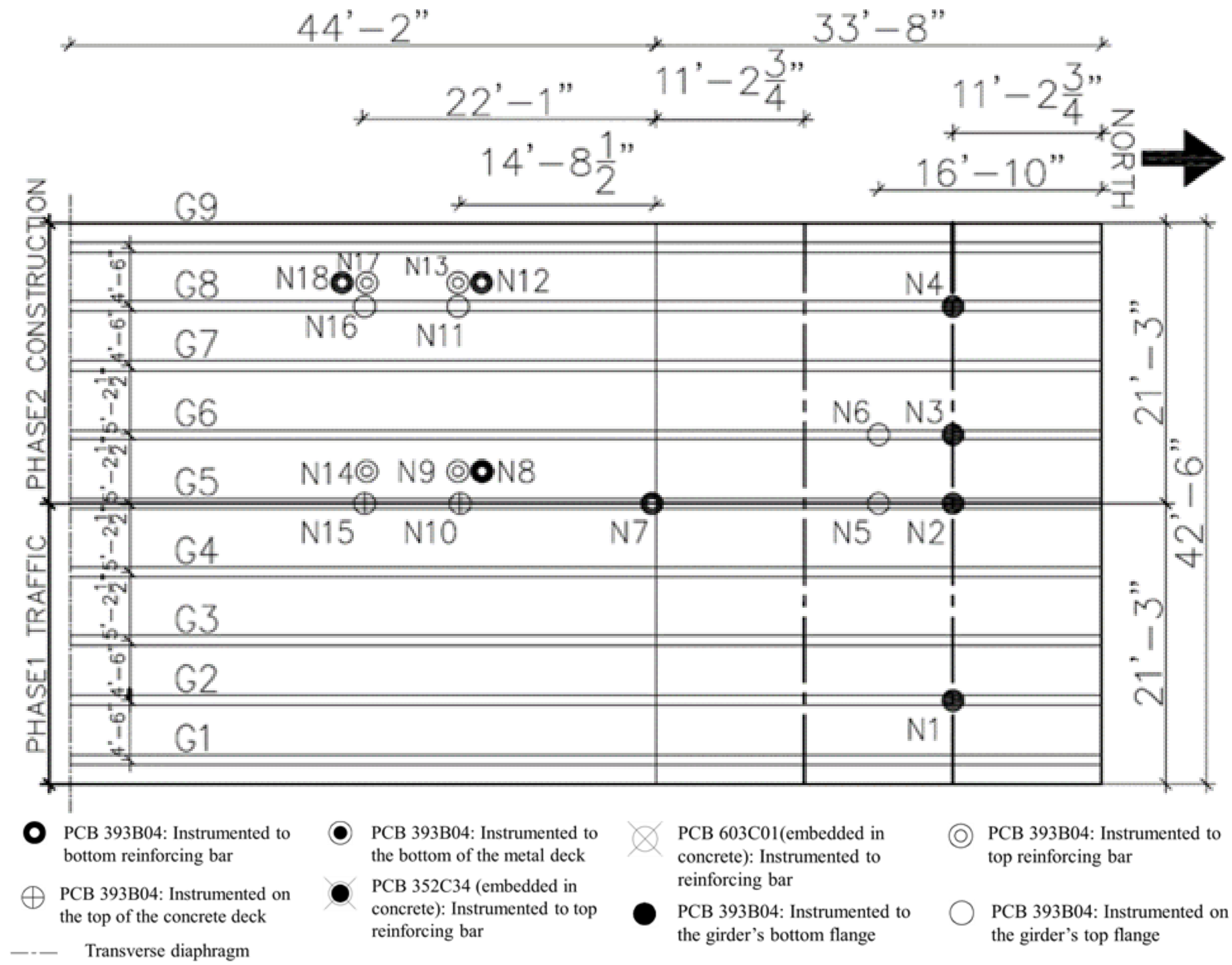


Figure 4. 7 Instrumentation layout for test T2-BR1

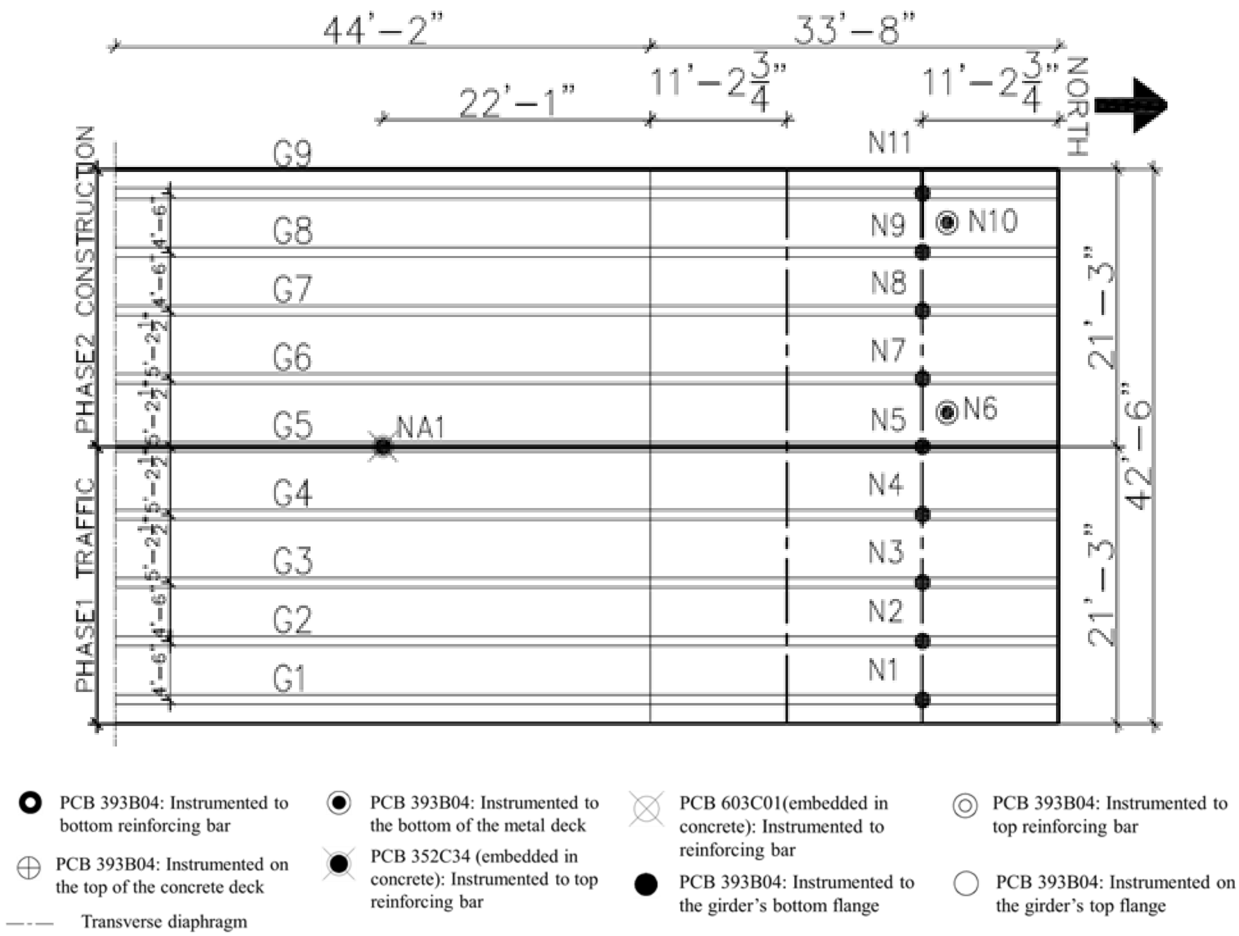


Figure 4. 8 Instrumentation layout for test T3-BR1

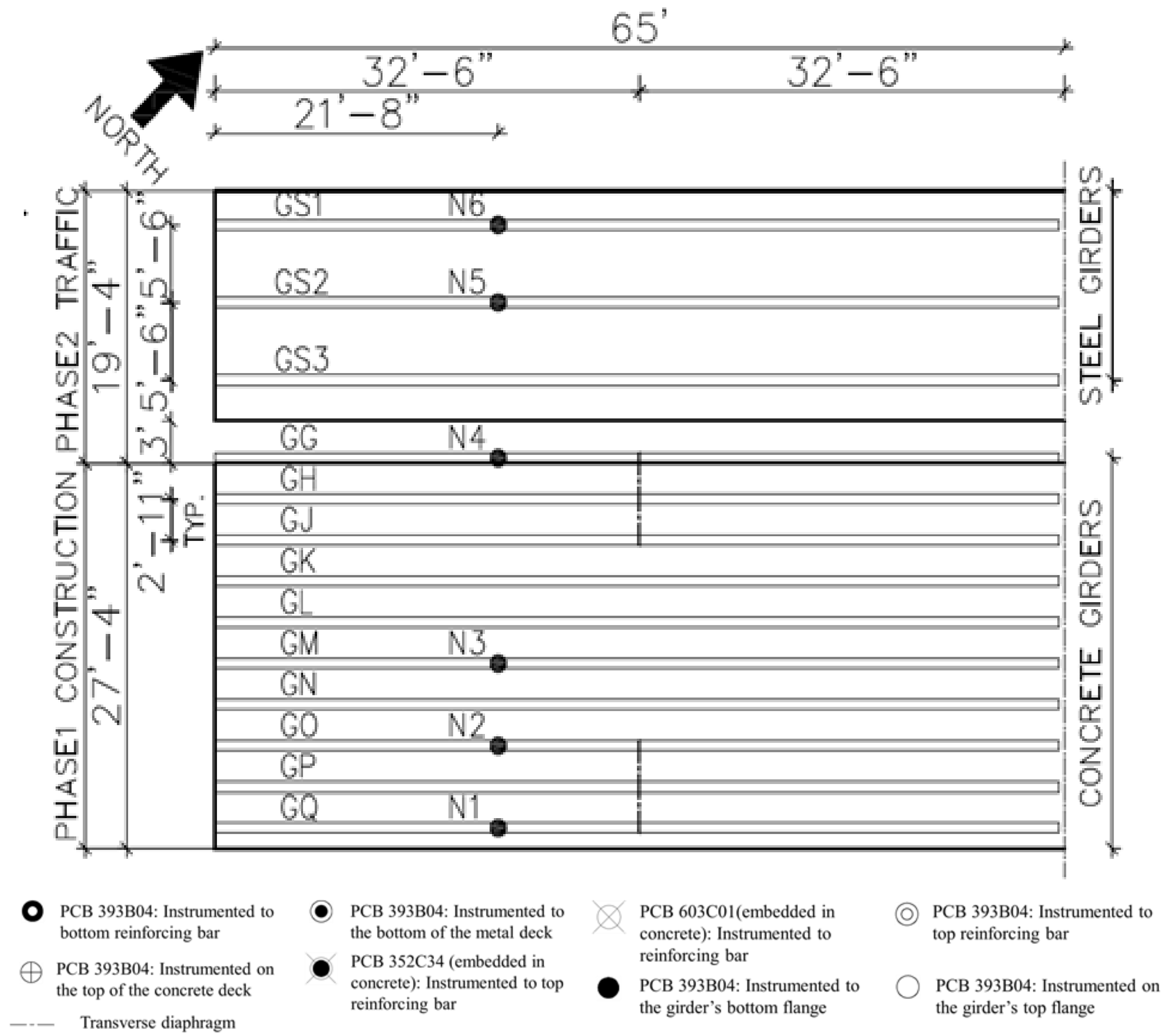


Figure 4. 9 Instrumentation layout for test T1-BR2

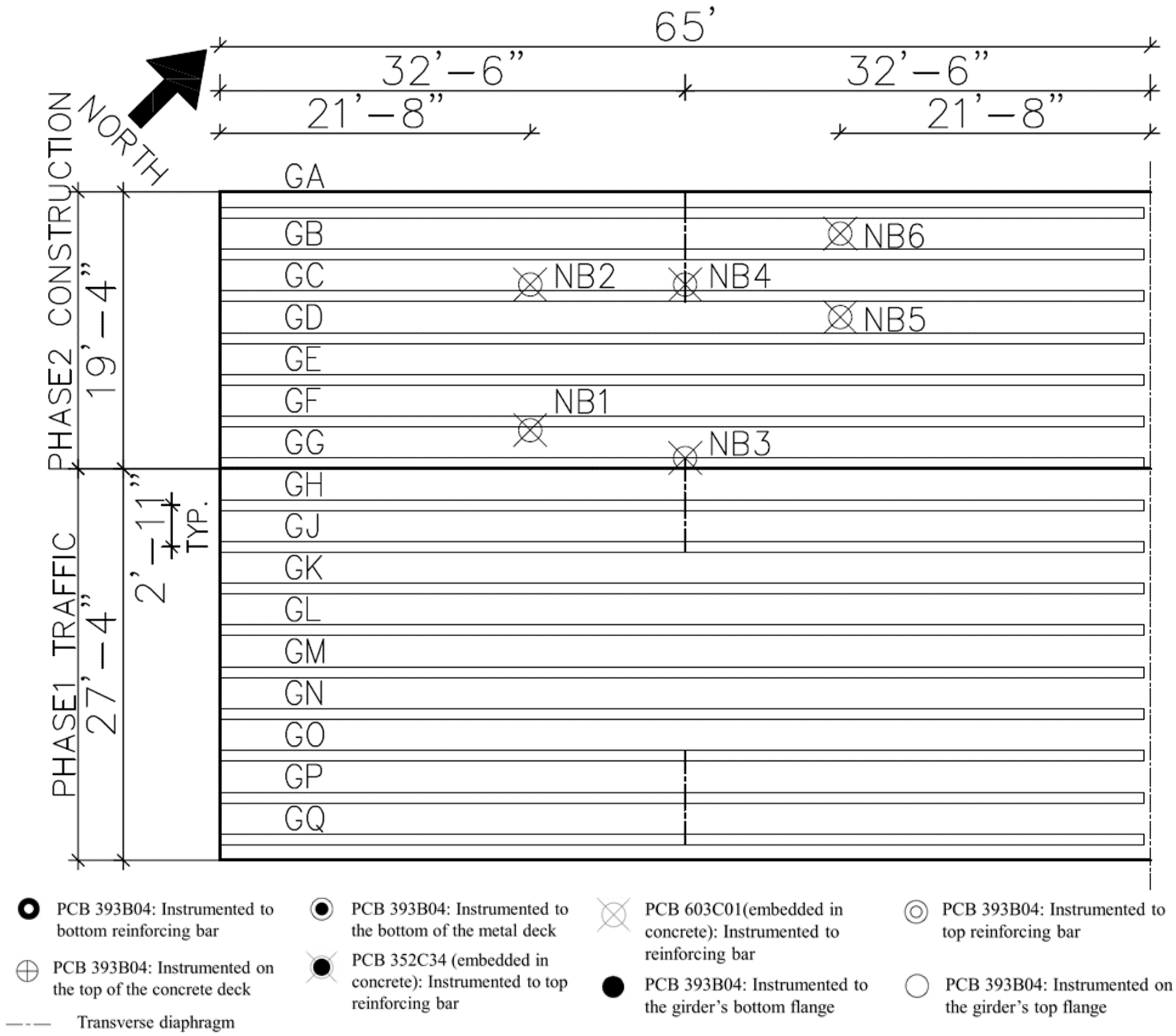


Figure 4. 10 Instrumentation layout for the rebar during T2-BR2, T3-BR2, and T4-BR2

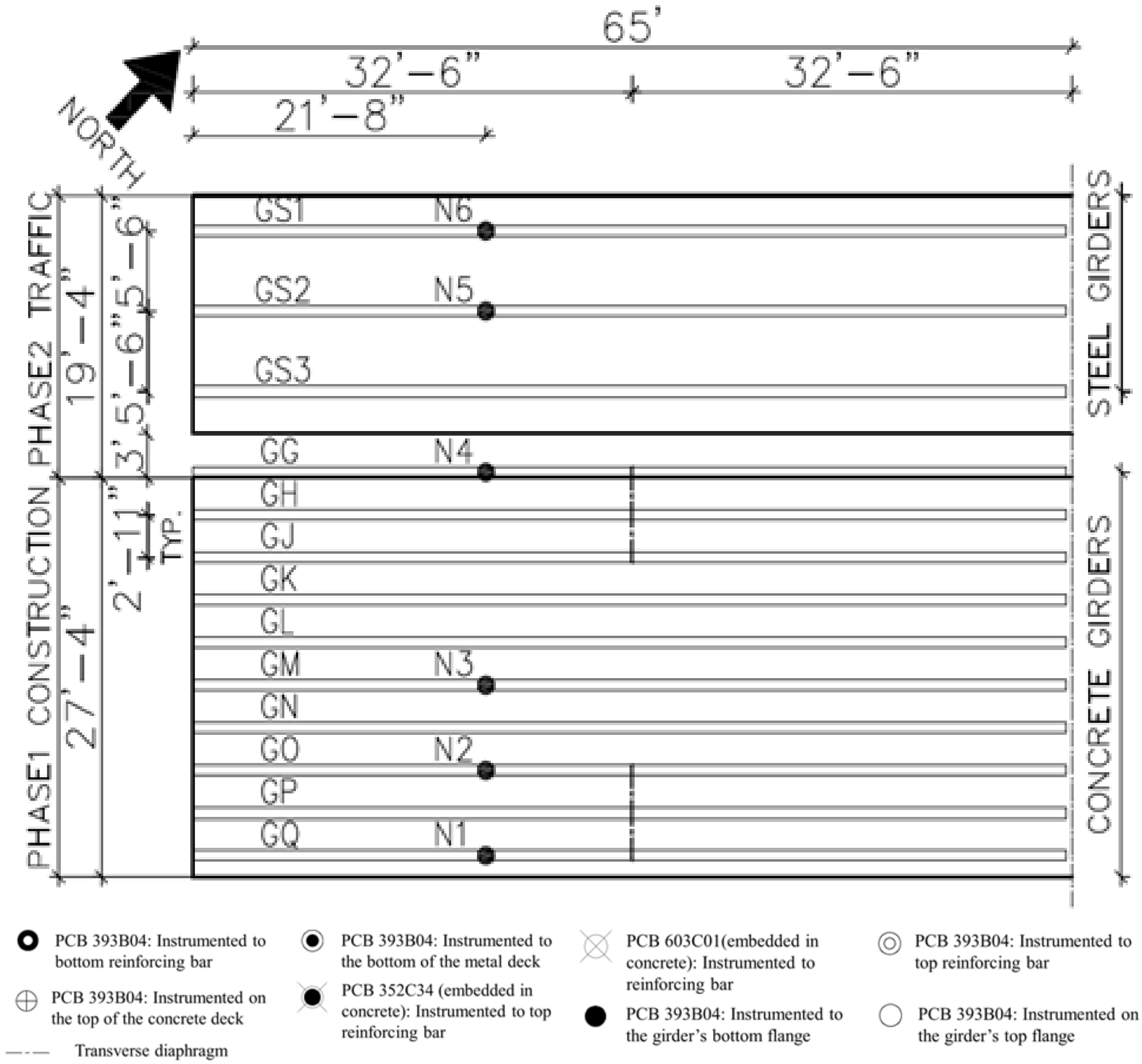


Figure 4. 11 Instrumentation layout for girders during T2-BR2, T3-BR2, and T4-BR2

4.4 RESULTS

4.3.1 System Identification

The main objective of the system identification task was to extract the dynamic characteristics of both the first and second phases of the monitored bridge from the raw acceleration data of the tests. The interpretation of the system identification results can aid in understanding the evolution of the dynamic characteristics throughout the construction process. For a robust estimation of the dynamic characteristics from the acceleration data, the Stochastic Subspace Identification (SSI) technique was used for the estimation of the global dynamic characteristics (i.e., Operational Deflected Shapes (ODSs), natural frequencies, and damping ratios). The ARTeMIS Modal pro software (Structural Vibration Solutions 2020) was used for this task. The acceleration data were divided into approximately 65-minute data segments (whenever applicable) and the sampling frequency of all the data segments was down-sampled from 2048 to 128 Hz for processing. To closely monitor the changes that occurred to the dynamic characteristics of both phases separate from one another, the data for each construction phase was analyzed individually.

The most unique outcome of the SSI analyses was the determination of the first two identifiable natural frequencies of the first and second phases for each bridge, as shown in Figures 4.12 and 4.13. The data points at time "0" correspond to the data segment that included the pouring activities of the second phase. Therefore, the data points before time "0" represent the response of both phases before the commencement of the second phase pour, and similarly, all the data points after time "0" represent the response of both phases during the curing of the second phase. The natural frequencies included in this plot were verified against the lowest two natural frequencies obtained from manual peak-picking task for validation. In addition, Modal Assurance Criterion (MAC) values were computed between the ODSs to ensure that the modes plotted at one time correspond to the same mode at subsequent times. Figures 4.12 and 4.13 show that the natural frequencies of both phases plunged at time 0, due to the addition of a great amount of mass (i.e., freshly poured concrete of the second phase) during the pour. Moreover, the SSI results showed that over the first 4-5 hours of curing, the first two identifiable natural frequencies of the first phase were higher than those of the second phase by nearly an order of 1-2 Hz. This discrepancy happened because the second-phase deck had a very low stiffness during the first few hours of curing before the final setting of concrete. Therefore, the transverse stiffness of the second phase was substantially reduced, which made the second phase more flexible than the first phase. Hence, both phases behaved independently from each other. However, the plots indicate that both phases continued stiffening over time, and within 6-7 hours of the pour completion of this bridge the first and second phases converged dynamically (i.e., both phases share the same natural frequencies). This key finding identifies a critical window of curing for the second-phase deck (i.e., within 6-7 hours of the pour). Applying traffic-induced vibration during this window is very critical as both phases still act as two independent systems. Consequently, closing the first phase to traffic until both phases stiffen and start behaving as a single system can potentially enhance the durability of the phased-construction bridges. This hypothesis is tested in the experimental section of this research.

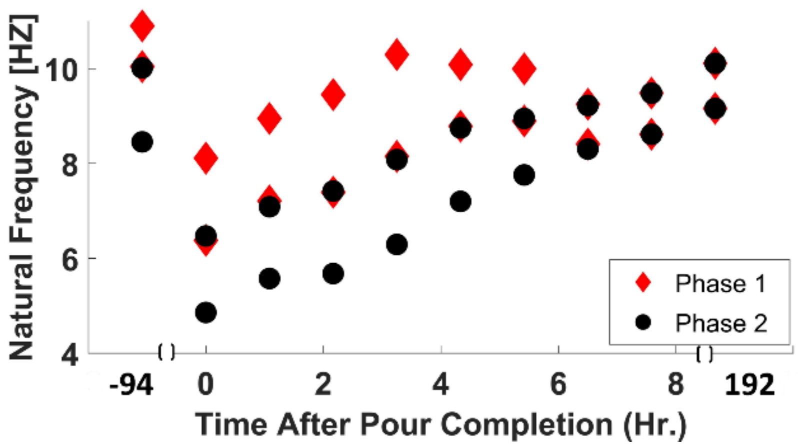


Figure 4. 12 Lowest two natural frequencies of each phase of Bridge 1 as a function of time with respect to the start of the second phase deck pour.

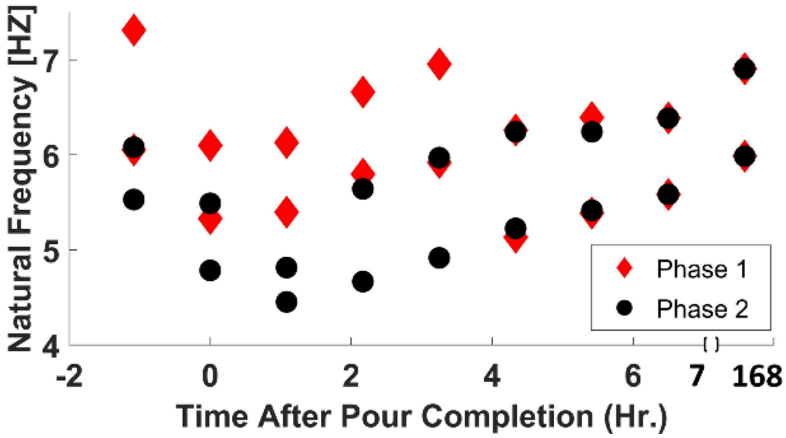


Figure 4. 13 Lowest two natural frequencies of each phase of the Bridge 2 as a function of time with respect to the start of the second phase deck pour.

4.3.2 Peak Acceleration and Displacement

The next step in analyzing the field monitoring data was to extract the peak accelerations and displacements of the girders and deck reinforcement before, during and after the stages of phased construction. The extracted peak accelerations and displacements were used to quantify the traffic-induced vibrations transmitted from the first to the second phases of both bridges, through the bridge diaphragms, phase-line edges of the first-phase decks, and deck reinforcement. To extract the peak accelerations corresponding to passing traffic or ongoing construction operations, a unified procedure was followed. First, it was assumed that the peak accelerations due to passing traffic or ongoing construction operations are greater than 0.0035g and the duration between two consecutive peaks shall be at least 15 seconds, to avoid the interference between two consecutive events. The cutoff acceleration and buffer duration were defined based on manually inspecting time history segments, which were including construction operations and passing traffic events. These thresholds were efficient in capturing peak accelerations corresponding to ongoing construction operations or passing traffic events. The acceleration time history segments were numerically integrated twice yielding corresponding displacements, from which the peak values could then be extracted. The results of this procedure are analyzed in the following sub-sections.

Traffic vs Construction Vibration

The peak accelerations of first-phase girders during the first phase deck pour are influenced only by construction activities, such as the presence of construction workers, bidwell, and the placement of concrete. On the other hand, the peak accelerations of second-phase girders during the second-phase deck pour are influenced by construction activities and traffic-induced vibration as a result of the spliced rebar tying the second-phase to the first-phase, which is open to traffic. Table 4.4 presents the 95th percentile peak acceleration for a central girder and an exterior girder for both of these scenarios for Bridge 2. 95th-percentile quantities are utilized to reduce the influence of outliers while still evidencing the higher end of the observations. Note that similar data was unable to be determined for Bridge 1 due to sensor failures during the first-phase deck pour. Considering the peak accelerations in this table, it can be seen that the presence of traffic increases the peak vibration by 125 – 140%. This indicates that a significant amount of vibration is contributed by the construction activities themselves; however, traffic-induced vibration is not negligible and considerably increases the vibration present during the second phase curing.

Table 4. 4 Peak acceleration with and without traffic-induced vibration

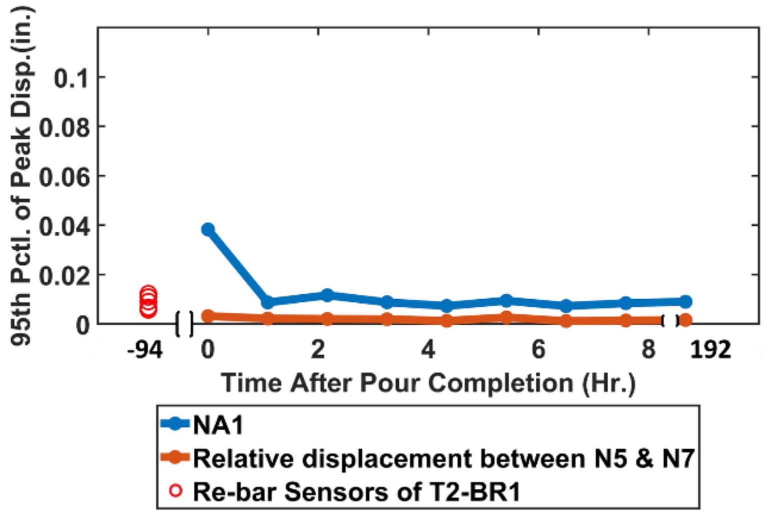
Girder	**95th Percentile Peak Acceleration**	
	Phase 1 Girders during Phase 1 Deck Pour	**Phase 2 Girders during Phase 2 Deck Pour**
Central	0.016 g	0.022 g
Exterior	0.023 g	0.029 g

Relative Rebar Motion

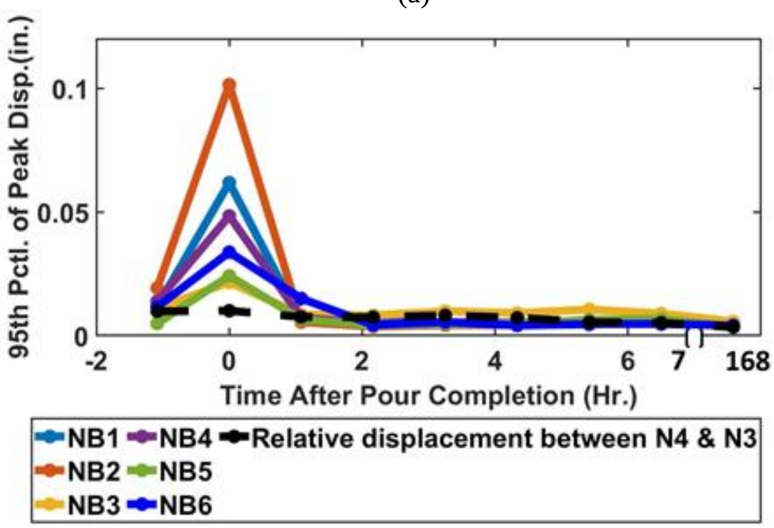
The peak displacements were extracted from first and second phase girders and reinforcing bars of both bridges. Figure 4.14 shows the summary plots of the 95th-percentile peak displacements of reinforcing bars before, during and after the concrete pour of the second-phase decks. In addition, the 95th-percentile peaks of the relative displacement between the phase-line girder and closest second-phase girder were determined and are included in this set of plots. For further interpretation and monitoring of the relative motion between the second-phase girders and the deck reinforcing-bars, the relative displacements were calculated and are shown in Figure 4.14c. In all, these plots indicate that the highest absolute and relative displacements in all cases occur during the time of the concrete pour (time = 0 on the plots). This makes sense given that the reinforcing bars were directly subjected to impact loads due to the stepping of the workers as well as pouring and vibration of concrete.

Considering Figure 4.14a, the relative displacements between adjacent girders at the phase line were effectively zero at Bridge 1 before, during, and after the second phase pour. This is plausible as the girders were instrumented near the abutment and at the location of continuous transverse diaphragms. Hence the sensors were placed at a relatively very stiff location and is likely not representative of the behavior of the entire bridge. On the other hand, considering Figures 4.14b and c for Bridge 2, non-negligible relative displacements between adjacent girders at the phase line were observed. Bridge 2 did not have transverse diaphragms between the first and second phases; and, therefore, the sensors were able to pick up the relative motion between the girders and rebar. This motion, however, is still relatively small with the 95^{th}-percentile peak relative displacement of only 0.01 inch between adjacent girders at the phase line. As time passes from the start of the second phase deck pour, the concrete of the second-phase deck hardens and the bridge systems stiffened. As can be seen in Figures 4.14a and b for both bridges, the displacements of the rebar and the relative displacements between adjacent girders at the phase line approached zero within approximately one hour after the pour completion.

As shown in Figure 4.14c, the peak relative displacements between embedded rebar and the girder below at the center of the second phase were nearly double that at the phase line. This may indicate that reduced bond strength may occur further from the phase line than originally anticipated. This behavior can be attributed to the stiffer rebar near the phase line. However, nearly an hour after the pour completion, the relative displacements near the phase line were actually higher than those near the center of the second phase. This is plausible since the response of the bridge was mainly governed by passing traffic after the pour completion and the source of traffic-induced vibration was the first phase. Therefore, the closest reinforcing bars and second-phase girders to the phase-line were more excited than the central second-phase girders and reinforcing bars. Eventually, when the concrete hardened and system stiffened after 4-5 hours from the pour completion of the second-phase deck, the relative displacements converged to nearly zero. This trend is similar to that observed in the SSI analysis (see Figure 4.12 and 4.13), and it indicates that the potential source of early-age deck degradation (i.e., relative motion between the reinforcing bars and the second-phase girders) could be eliminated after 4-5 hours from the pour completion, where the first and second phases would start behaving as one system.



(a)



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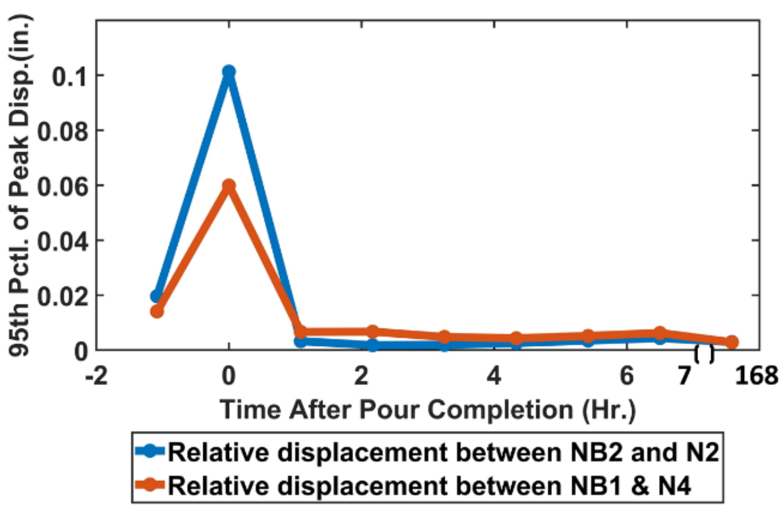


(c)

Figure 4. 14 95th-percentile peak displacements of reinforcing bars as a function of time after the second phase deck pour for (a) bridge 1; and (b) and (c) bridge 2.

CHAPTER 5 – EXPERIMENTAL TESTING

5.1 OVERVIEW

As discussed in the field monitoring chapter, it is hypothesized that closing phased-construction bridges during the second phase of construction for the first few hours of curing will enhance the durability and the strength of the concrete-rebar bond and bridge deck. Therefore, the main goal of this section is to experimentally quantify the effectiveness of closing phased-construction bridges to traffic during the second phase of deck construction for the first 6-7 hours of deck curing on the durability of the phased-construction bridges. To this end, the final objective of this project was to replicate a phased-construction scenario of bridges in a controlled lab environment for further investigation of the key findings of the field monitoring task and to draw conclusions about the effect of maintaining traffic during concrete curing on the deck durability and strength. An experimental program incorporating the testing of 3 large-scale bridge deck specimens monotonically until failure, where two specimens were constructed in a phased manner (i.e., specimens were subjected to simulated traffic-induced vibrations during concrete curing), and one specimen was constructed in a non-phased manner (i.e., the specimen was not subjected to traffic-induced vibration during concrete curing), was executed to accomplish the experimental task's objective. The responses of the specimens and reinforcing bars along the lap splice were monitored using displacement and strain sensors.

5.2 EXPERIMENTAL PROGRAM

The experimental program included the testing of three full-scale bridge deck specimens, where two specimens were phased-constructed, and the other specimen was non-phased-constructed. To construct a phased-constructed specimen, firstly, a deck segment, which is known as the first phase, had been constructed and cured without being subjected to any source of simulated traffic vibration. Then, after the hardening of the first phase, the remaining deck segment, which is known as the second phase, was constructed and attached to the hardened first phase, and cured under the effect of the simulated traffic-induced vibration. Both phased-constructed specimens were subjected to the same simulated traffic-induced vibration displacement protocol, but one of the phased-constructed specimens was immediately subjected to the vibration from the beginning of the second-phase deck concrete pour, while the other phased-constructed specimen was subjected to the vibration 6-7 hours after the beginning of the second-phase deck concrete pour. Furthermore, an additional specimen was fully constructed in a non-phased manner, where the whole specimen was cast at once without being subjected to simulated-traffic induced vibration during curing. The non-phased-constructed specimen response then was compared to the response of the phased-constructed specimens. The specimen design was intended to replicate a typical bridge in Nebraska in terms of dimensions, concrete mix, formwork design, and reinforcement design and details.

To quantify the effect of the simulated traffic-induced vibration on the specimen's concrete-rebar bond strength, integrity and strength, each specimen was tested monotonically until failure in a 3-point bending test. Reinforcing bars were instrumented with several strain gauges along the lap splice to monitor the change in the average bond stress along the reinforcing bars during the tests. Also, LVDTs were used to monitor the differential displacements between the first- and second-phase decks during the application of the simulated traffic-induced vibration displacement protocols.

5.2.1 Dynamic Test Setup

The dynamic test setup, as shown in Figure 5.1, was designed to impart simulated traffic-induced vibration protocols to the hardened first-phase deck using a vertical 110-kip MTS hydraulic actuator over the first hours of curing of the second-phase deck. The test setup intended to mimic a typical phased-construction scenario of a bridge, but in a controlled laboratory environment. The phased-constructed specimens were connected to four W8X24 beams, which acted as bridge girders. The W8X24 beams were bolted to *strong girders*, which are necessary to maintain the appropriate height of the specimen in the lab while providing no flexibility to the setup. The *strong girders* were firmly connected to the lab's floor. The specimens were connected to the W8X24 beams using 0.75-in diameter bolts, which acted as shear studs.

Four PVC pipes were embedded in the first-phase deck to facilitate the actuator's attachment to the first-phase deck using 0.75-in diameter threaded rods. Figure 5.2 shows the dynamic test of a phased-constructed specimen at the lab, where the simulated traffic-induced vibration protocols were imparted while the second-phase deck was curing in the formwork.

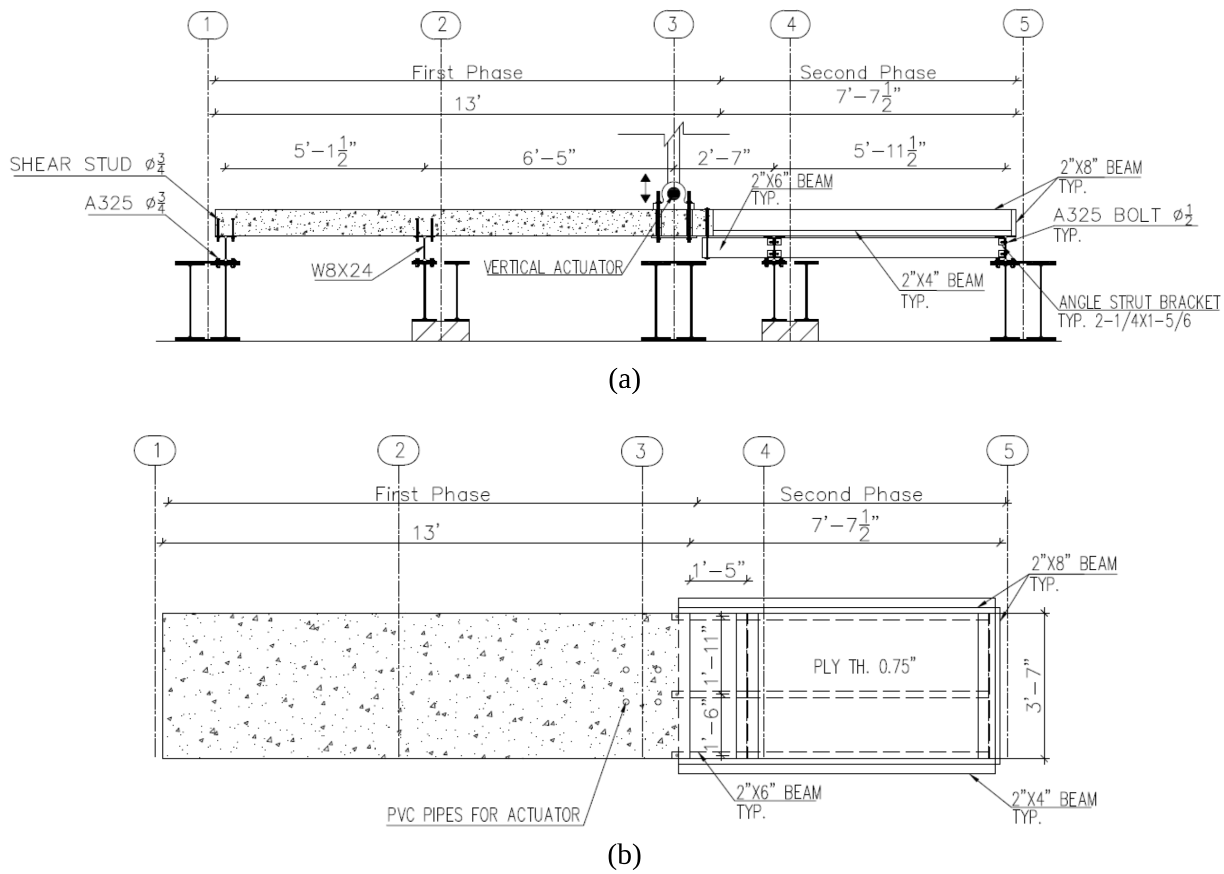


Figure 5. 1 (a) Elevation and (b) plan view of the dynamic test setup and formwork details of the second-phase deck.

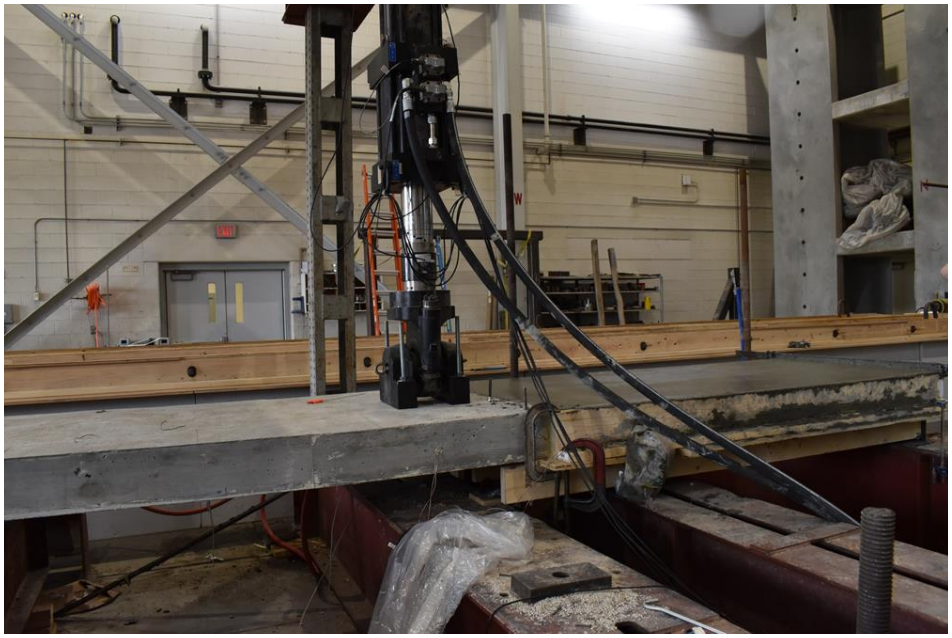
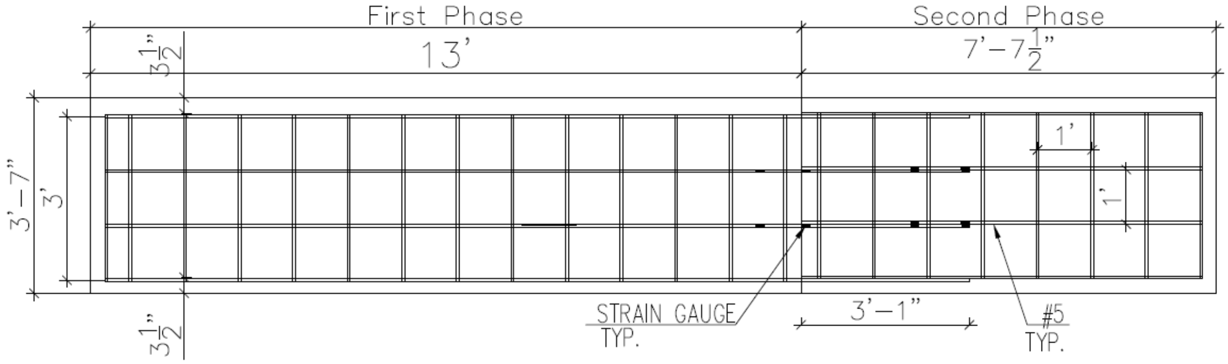


Figure 5. 2 Photo of dynamic test setup

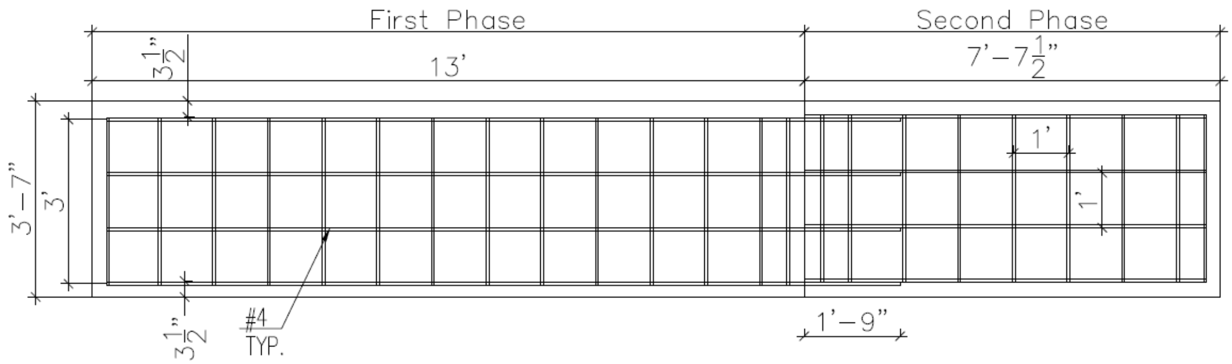
5.2.2 Specimen Design

All the specimens were 247.5-in long, 43-in wide, and 7.25-in thick. The specimen's dimensions and spacing between the W8X24 beams were chosen to replicate a typical transverse strip of an actual bridge deck supported on girders. Regarding the phased-constructed specimens, the first-phase decks were 13-ft long first phases and the second-phase decks were 7-ft 7.5 in long. The typical concrete mix specified by NDOT for bridge decks, 47 BD, was used for casting all the specimens. The mix has a minimum compressive strength at 28 days of 4000 psi and a maximum slump of 4 in. The concrete was acquired from local ready-mix suppliers. Burlap was used for the wet curing of the specimens for 10 days, which is the same as the procedure specified for curing in the field in Nebraska.

The reinforcement of the specimen was designed and detailed using the empirical method as per the AASHTO LRFD bridge design specifications (AASHTO 2017), and the bridge office policies and procedures of NDOT. Figure 5.3 shows the top and bottom reinforcement details of the specimen. All the reinforcing bars used were Grade 60. The lap splice lengths of the bottom and top reinforcing bars were 37- and 21-in long, respectively. All the specimens had the same reinforcement details.



(a)



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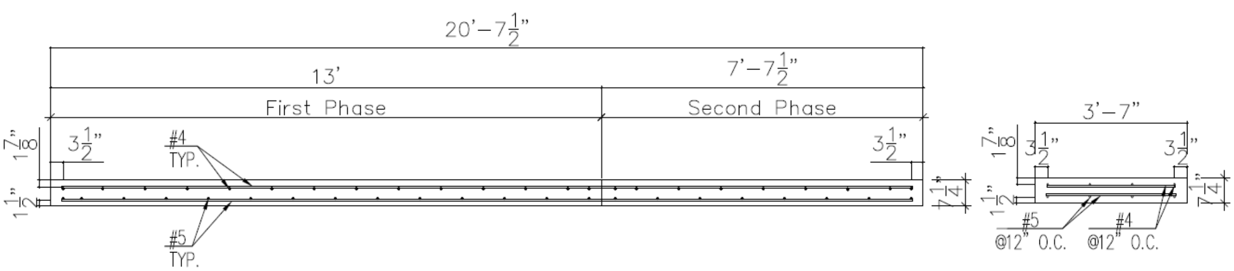
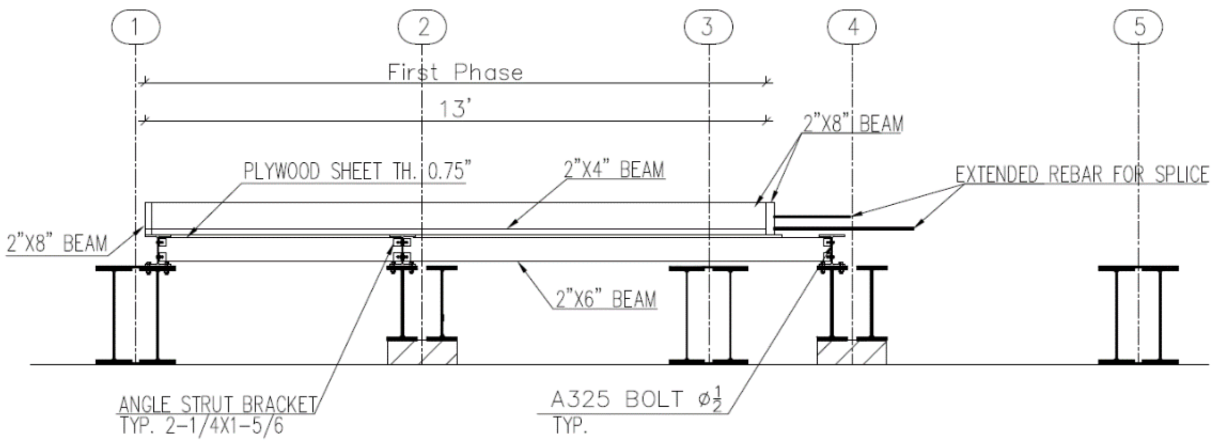


(c)

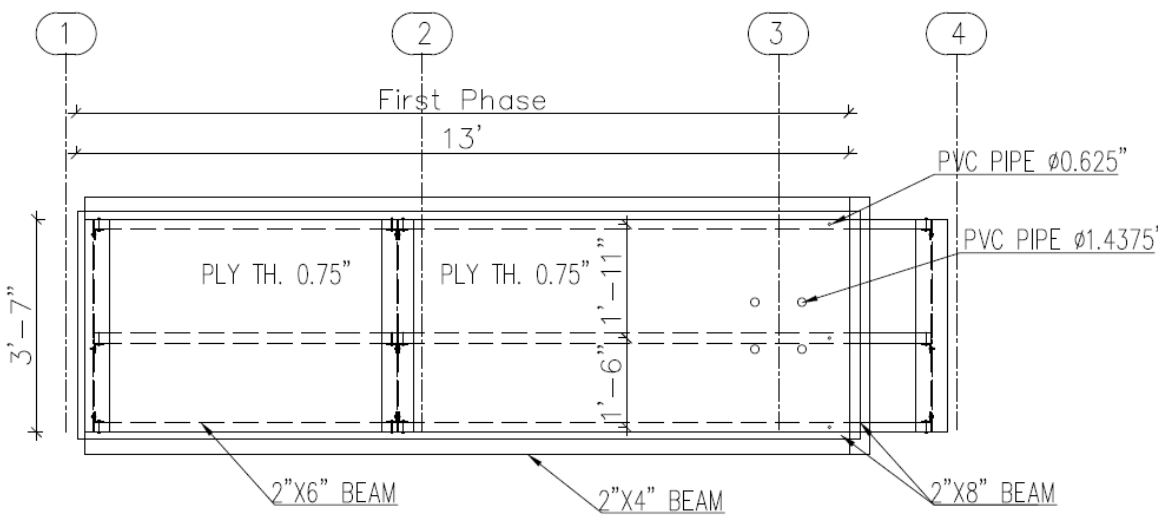
Figure 5. 3 Plan view of the specimen's (a) bottom and (b) top reinforcement details; and (c) side views of the specimen's reinforcement details.

5.2.3 Formwork Design

The formwork used for the phased-constructed specimens was designed to simulate the formwork used for phased-construction bridges in the field. Figure 5.4 includes the formwork details of the first-phase deck while the formwork for the second-phase deck is included in Figure 5.1. . The formwork consisted of 0.75-in thick plywood sheets supported on the 2X6 wood beams spanning between the W8X24 beams. The formwork sidewalls were made of 2X8 wood beams, which were reinforced at the bottom with 2X4 wood beams. Three 0.50-in diameter steel threaded rods passing through embedded PVC pipes at the phaseline edge of the hardened first phase deck were used to attach the second phase formwork (i.e., 2X6 wood beams) to the hardened first phase deck.



(a)



(b)

Figure 5. 4 (a) Elevation and (b) plan view of the first phase of construction formwork.

5.2.4 Dynamic Displacement Loading Protocol

A displacement protocol was developed to simulate the motion of the first-phase rebar extended into the second-phase deck during and after a typical second phase of construction of bridge decks. As mentioned in Chapter 4, the rebar in the vicinity of phaseline was monitored during and after the pour of the second-phase deck of the Silver Creek bridge (CN: 42745). The rebar response in the event of truck passes was extracted and closely studied statistically for the development of the displacement protocol for the dynamic tests. As shown in Figure 5.5, the building unit of the developed protocol consisted of four wavelets, where the first three shorter wavelets were 1.365 seconds long and represented the passes of single trucks, while the fourth longer wavelet was 2.275 seconds long and represented the pass of several trucks back-to-back. The waveform of the wavelet was similar to a sine wave with unequal positive (upward) and negative (downward) amplitudes. The positive amplitude (i.e., uplift) simulated the typical response of the deck in a multi-span bridge when trucks travel on the adjacent spans. Based on the results of the numerical analyses conducted by Weatherer (2019), the positive amplitude was chosen to be 40% of the negative amplitude. The delay between the wavelets was chosen to be 10 seconds, which was selected as the median rate of the truck passes of several bridges in Nebraska on the I-80 route during rush hour (personal communication). The negative amplitude of the protocol was 0.125 in, which was selected to match the highest differential deflection between the first and second phases recorded in the past studies (Furr and Fouad 1981; Weatherer 2019). During the dynamic tests, the building unit of the displacement protocol was repeated continuously until the end of the test, however, the protocol amplitude was decaying over the test duration. The amplitude decay was due to the reduction in the reinforcing bar motion that occurs as concrete hardens, as discussed in the field monitoring task. Table 5.1 shows the amplitude decay pattern used for the dynamic tests as percentages of the initial amplitude (i.e., +0.05/-0.125 in).

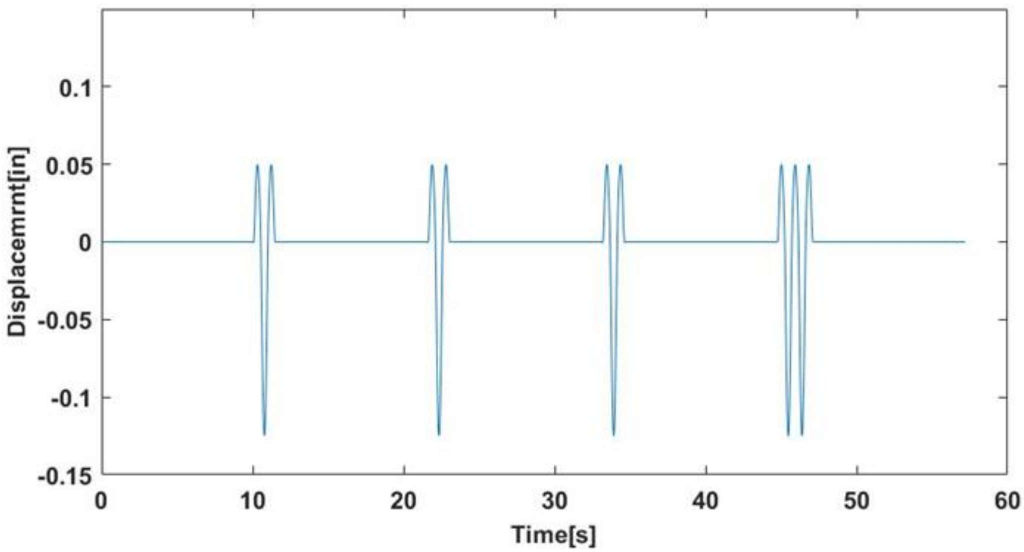


Figure 5. 5 Simulated traffic-induced vibration displacement protocol for 0-3 hours.

Table 5. 1 Amplitude decay pattern for the displacement protocol

Concrete age [Hours]	Amplitude [%]	Negative amplitude [in]	Positive amplitude [in]
0-3	100	0.125	0.05
3-7	80	0.1	0.04
7-12	55	0.06875	0.0275

5.2.5 Test Matrix

The test matrix of the experimental program included the testing of three specimens, as shown in Table 5.2. Specimen 0 was constructed in a non-phased manner (i.e., the specimen was fully cast at once) and was not subjected to any simulated traffic-induced vibration during casting nor curing. However, specimens 1 and 2 were constructed in a phased manner and underwent dynamic tests of different vibration durations. The simulated traffic induced-vibration displacement protocol was imparted to the first-phase deck of specimen 1 for 12 hours continuously starting from the beginning of the concrete pour of the second-phase deck. While the impartment of vibrations to the first-phase deck of specimen 2 began right after the final setting of the second-phase deck (i.e., after 6.7 hours from the start of the concrete pour); details on the concrete setting time test for the concrete mix 47 BD can be found in section 5.2.8. The dynamic test of specimen 1 simulated a typical phased construction scenario, where the bridge remains open to traffic during the second phase of construction. While specimen 2 simulated the scenario of a phased-construction bridge that was closed to traffic during the second phase construction until the final setting time of the second-phase deck was reached. Based on the field monitoring task, it is believed that resuming the traffic to phased-construction bridges after the final setting time of the second-phase deck will greatly reduce any potential early-age deck degradation due to maintaining traffic during the second phase of construction. Therefore, the main goal of the test matrix was to quantify to effectiveness of closing the first-phase decks to traffic until the second-phase decks reach the final setting on enhancing the structural integrity and durability of phased-construction bridges.

Table 5. 2 Test matrix

Specimen No	**Description**
0	Baseline (No vibration)
1	Traffic-Induced vibration imparted from start of pour (0 – 12 Hours)
2	Traffic-Induced vibration imparted after concrete setting time (6.7 - 12 Hours)

5.2.6 Monotonic Test Setup

The monotonic tests were 3-point bending ultimate strength tests for the specimens, where the specimens were loaded in the vicinity of the phaseline joint gradually until failure. To ensure that the whole region of the lap splice was subjected to positive bending moments during the monotonic test, the specimens were repositioned underneath the actuator as shown in Figures 5.6 and 5.7. The monotonic tests were run in a displacement control mode with a rate of 0.10 in/min. For the phased-constructed specimens (i.e., specimens 1 and 2), the monotonic tests were executed after the achievement of the second-phase deck to the minimum required compressive strength (i.e., 4000 psi). Ultimately, the monotonic test aimed to test the effect of applying different durations of simulated traffic-induced vibration on the overall flexural strength of the specimens.

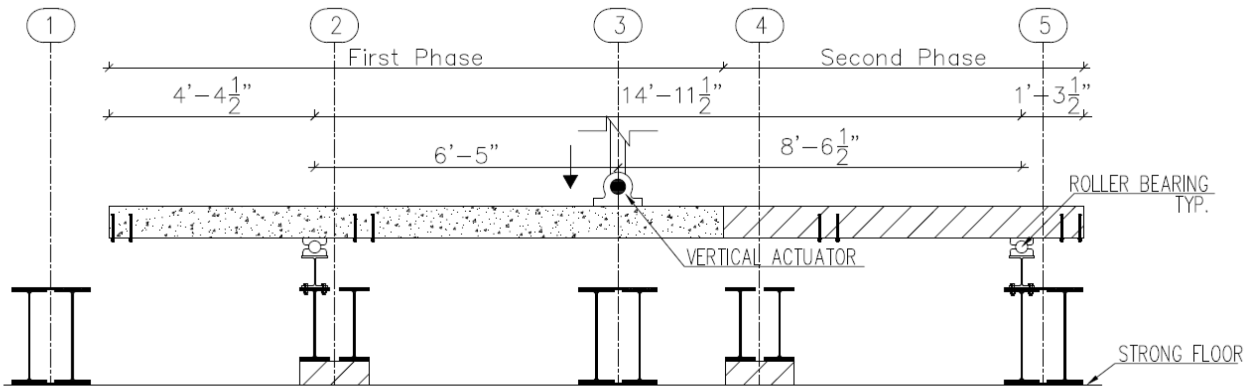


Figure 5. 6 Monotonic test setup

Figure 5. 7 Photograph of monotonic test

5.2.7 Instrumentation

Several 350-Ohm strain gauges from Micro Measurements and Texas Measuring Instruments Lab were instrumented to reinforcing bars along the lap splice, as shown in Figure 5.8. The strain gauges were primarily used to monitor the change in the average concrete-reinforcing bar bond stress along the lap splice during the monotonic ultimate strength tests. Furthermore, two LVDTs were instrumented to the bottom of the first- and second-phase decks in the vicinity of the phaseline joint, as shown in Figure 5.9. The LVDTs monitored the differential displacement between the first- and second-phase decks throughout the dynamic test. Moreover, the 110-kip MTS hydraulic actuator was equipped with a load cell and an LVDT to detect the displacements and forces imparted to the specimens by the actuator during the dynamic and monotonic tests.

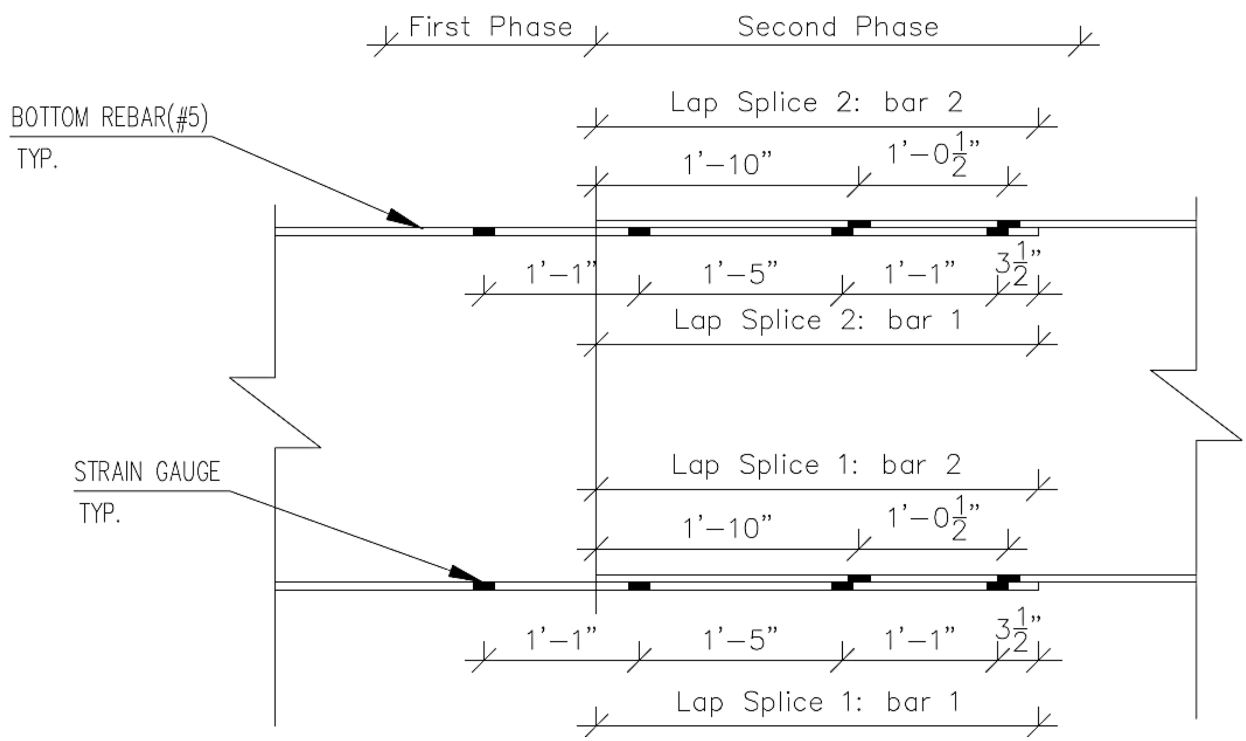


Figure 5. 8 Plan view of strain gauge instrumentation

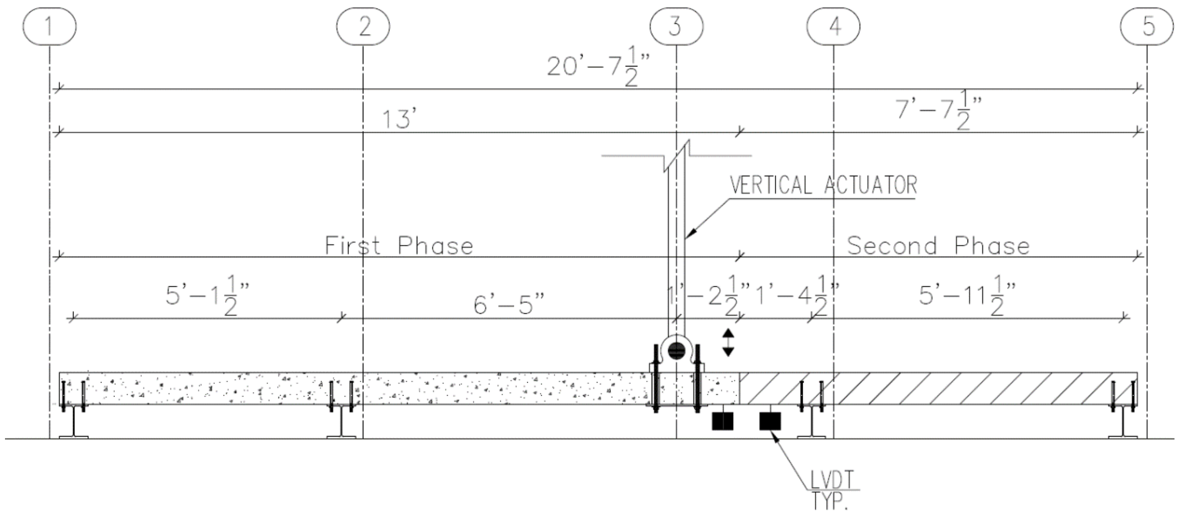


Figure 5. 9 Elevation view of LVDT instrumentation

5.2.8 Material Testing

Concrete Setting Time

The initial and final setting times for the concrete mix 47 BD were determined in accordance with ASTM C 403 – Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance. The initial and final setting times determined were 5.2 and 6.7 hours, respectively. The setting time refers to the time elapsed since the mixing of the concrete constituents and until the concrete achieves a certain penetration resistance value. The penetration resistance values at the initial and final setting times were 500 and 4000 psi, respectively. Figure 5.10 shows the setting time test results.

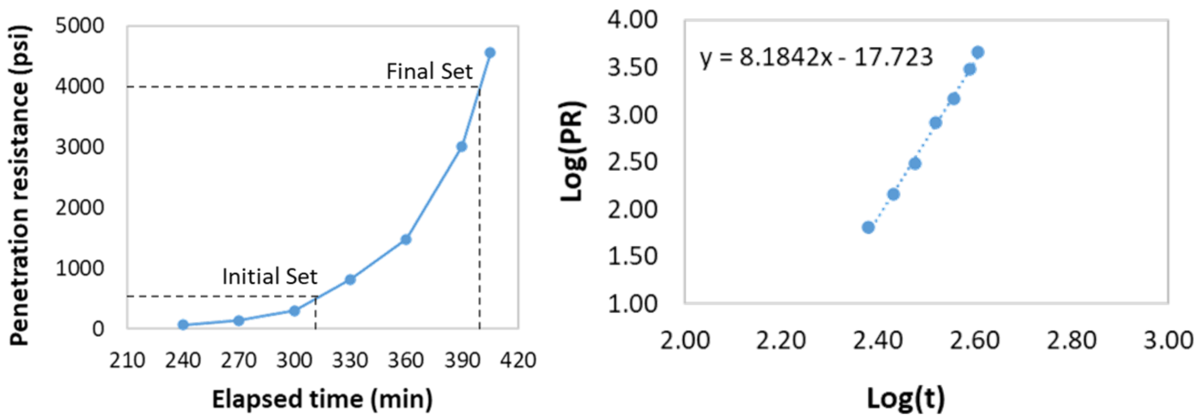


Figure 5. 10 Penetration resistance versus elapsed time and log-log plot with linear regression.

Concrete Compressive Strength

4-in by 8-in concrete cylinders were cast for all the concrete pours used in casting the specimens in accordance with ASTM C31 – Standard Practice for Making and Curing Concrete Test Specimens in the Field. The compressive strengths of all the concrete cylinders were determined in accordance with ASTM C39 – Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens. The concrete cylinders were grinded before conducting the compressive strength tests. Table 5.3 lists the compressive strengths for the concrete cylinders of specimens 0, 1, and 2, respectively. The tables show the compressive strengths of the specimens on the dynamic and monotonic test days. Interpolation and extrapolation techniques were used for estimating the compressive strengths when no cylinder compressive strength tests were conducted on any of the specimen's test days.

Table 5. 3 Average compressive strength of specimens

Specimen	Phase	Age (days)	Test day	Average Compressive Strength (psi)
0	-	3	-	3439
	-	7	-	4258
	-	28	-	5412
	-	30	Monotonic	5467
1	1	3	-	3438
		7	-	4258
		21	Dynamic	4860
		28	Monotonic	5412
	2	3	-	4575
		14	Monotonic	5714
		28	-	7164
2	1	3	-	4321
		17	Dynamic	5887
		28	Monotonic	7117
	2	3	-	4690
		11	Monotonic	5262
		28	-	6478

5.3 RESULTS AND DISCUSSION

5.3.1 Dynamic Tests

Differential Displacements

The displacements recorded by the LVDTs were processed using signal processing techniques to extract the differential displacements between the first- and second-phase decks during the dynamic tests of specimens 1 and 2. The test data were divided into 60-minute-long data segments; the differential displacements were calculated as the subtraction of the second-phase deck displacements from the first-phase deck displacements. Figure 5.11 shows the 95th percentiles of the peak differential displacements of specimens 1 and 2 over the dynamic test duration; the amplitudes of the simulated traffic-induced vibration displacement protocol applied during the curing of the second phase and data collection are overlaid on the plots of Figure 5.11. The casting of the second-phase deck had begun and finished during the first hour of the dynamic the test for both specimens 1 and 2; the first hour of the dynamic test corresponds to the datapoint at hour 1 in Figure 5.11.

Figure 5.11 shows that the differential displacements of specimens 1 drastically decreased from 0.0603 in to 0.0109 in over the first 7 hours of the second-phase deck curing (i.e., before the final setting). However, the rate of decrease of the differential displacements over the curing hours 1-7 (i.e., before the final setting of the second-phase deck) was steeper than the rate of decrease over the curing hours 7-12 (i.e., after the final setting of the second-phase deck) for both specimens 1 and 2. The trend of results indicates that most of the deck stiffening occurred over the critical time window of curing between the concrete pour and the final setting of the concrete (i.e., 1-7 hours), which aligns with the trend of the results seen in the field monitoring. Therefore, the closure of the first-phase deck to traffic over the critical time window of curing will allow the second-phase deck to greatly stiffen without the presence of any source of traffic-induced vibration, which may alleviate some early-age deterioration of phased-construction bridges.

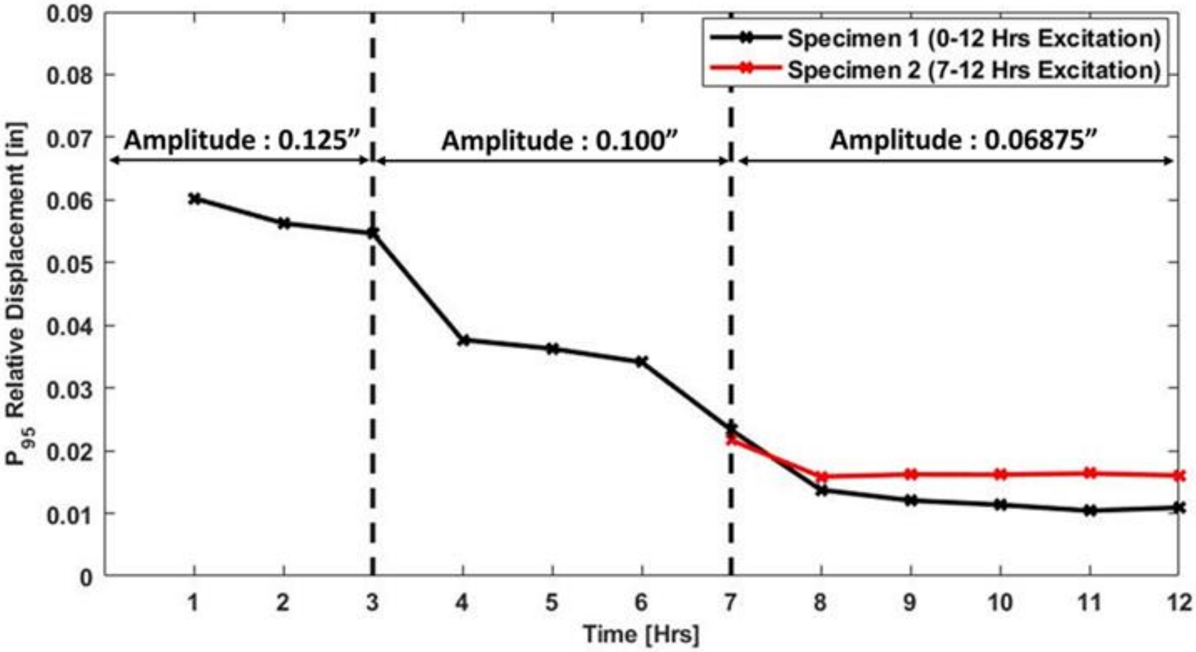
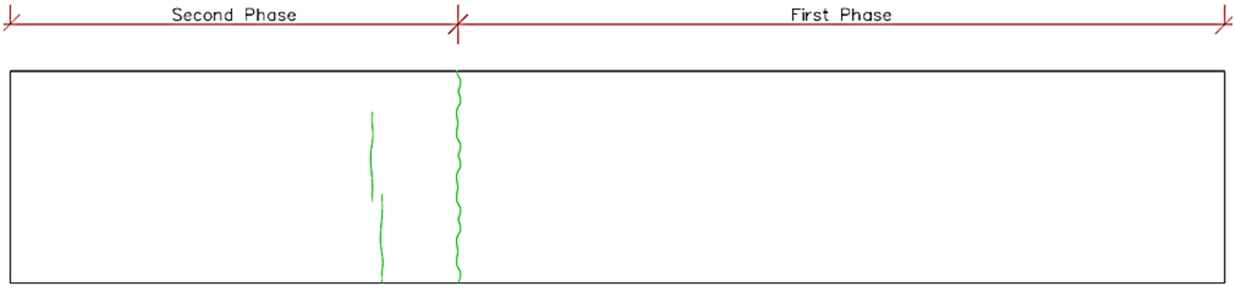


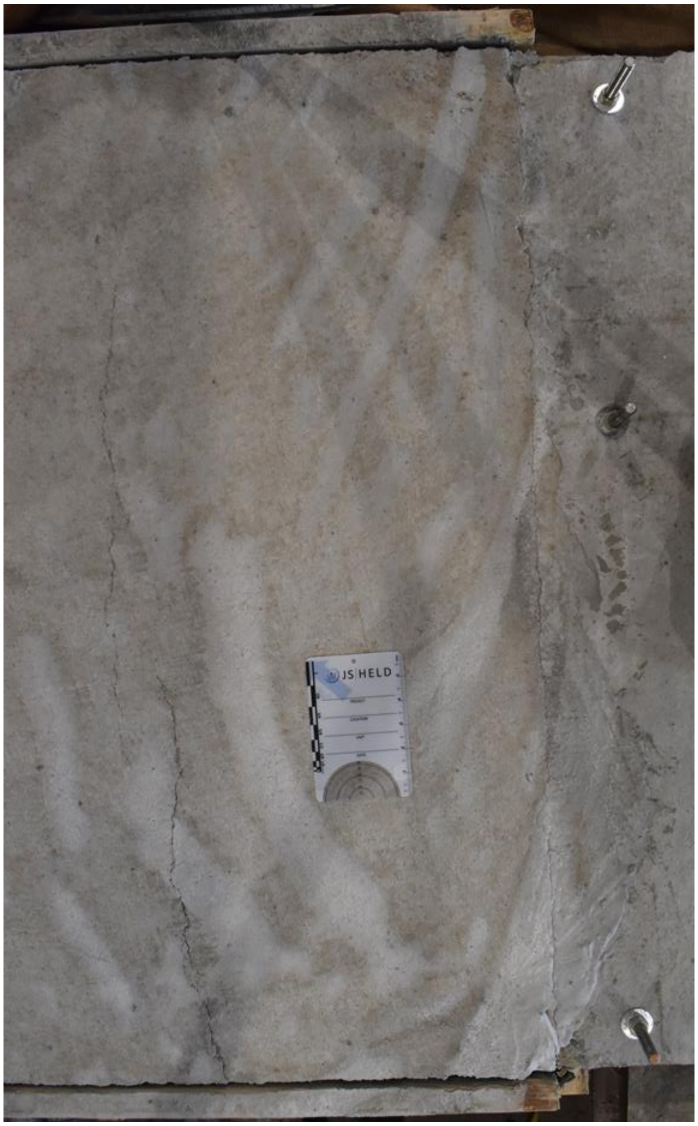
Figure 5. 11 Relative displacements between the first and second phases over the dynamic test duration.

Crack Patterns

Specimens 1 and 2 were inspected for cracks after 3 days from the dynamic tests; both specimens developed flexural cracks due to the negative moments at the top of the second-phase decks in the vicinity of the closest W8X24 beam to the phaseline, as shown in Figures 5.12 and 5.13, respectively. However, the cracks of specimen 1 were 1/16-in thick, while the cracks of specimen 2 were hairline thick. Moreover, unlike specimen 2, specimen 1 exhibited a 1/16-in thick crack along the phaseline. The comparison between the cracking patterns of both specimens showed that delaying the impartment of the simulated traffic-induced vibration until the final setting of the second-phase deck reduced the cracking and width of the cracks developed after the dynamic test. The development of early-age cracks at the top of the deck is very critical to the durability of the bridge decks on the long run; since bridge decks operate in very harsh environments (i.e., subjected to snow, deicing compounds/salts, and dynamic traffic loads), the cracks at the top of the deck would eventually lead to the rusting of the reinforcing bars, deterioration of the concrete-rebar bond, and loss of the deck integrity. Hence, early-age deck cracks need to be addressed as soon as possible to prolong the life span of the phased-construction bridge-decks.

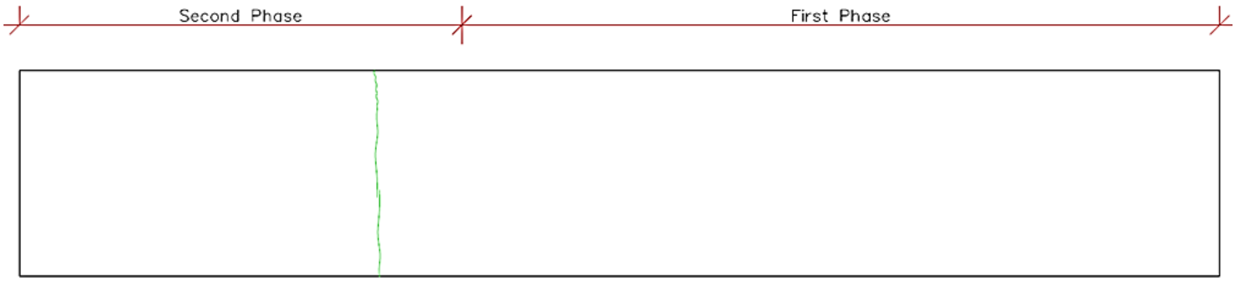


(a)

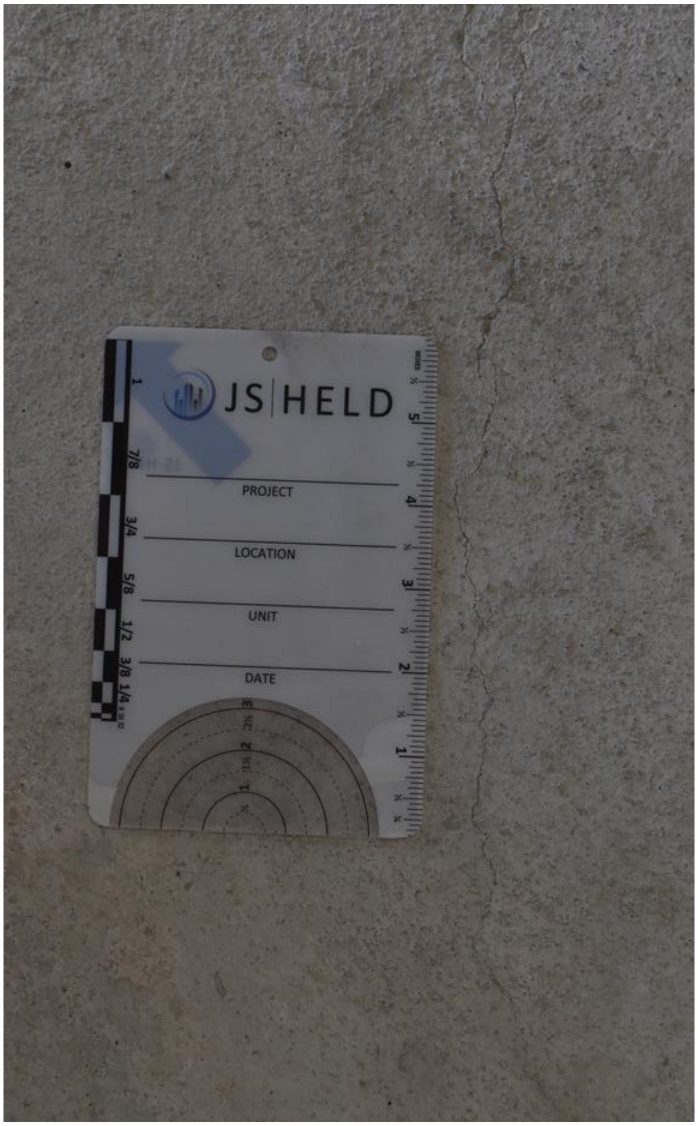


(b)

Figure 5. 12 (a) Crack map and (b) photo of cracks on the surface of Specimen 1 that developed within 3 days after the dynamic test (crack width = 1/16”).



(a)



(b)

Figure 5. 13 (a) Crack map and (b) photo of cracks on the surface of Specimen 2 that developed within 3 days after the dynamic test (crack width: hairline).

5.3.2 Monotonic Tests

Moment-Deflection

Figure 5.14 shows the maximum moment-displacement plots of the monotonic tests for specimens 0, 1, and 2. The maximum moment of the specimen occurred at the point of load application. The compressive strengths (i.e., f_c') of specimen 0 and, the phases of specimens 1 and 2 are provided in the figure for comparison. All the specimens failed in flexure-compression after substantial yielding of the bottom reinforcing bars. Furthermore, the peak maximum moments of all the specimens exceeded the design moment strength (i.e., ϕM_n= 35.48 kip-ft) by at least 33%. Specimens 0 and 1 had very similar response as they had similar compressive strength, while specimen 2 had substantially higher maximum moments which can be attributed to the higher compressive strength of the first-phase deck. The results show that despite the impartment of traffic-induced vibration during the curing of the second-phase deck, all specimens were able to develop peak moments that are higher than the required design strength.

Crack Patterns

Crack maps were developed for specimens 0,1, and 2 after the destructive monotonic tests, as shown in Figures 5.15 – 5.17, respectively. All specimens had the same cracking pattern in the vicinity of the actuator (i.e., point of load application), where concrete crushing occurred at the top of the deck and flexural cracks at the bottom. Moreover, nearly all the specimens exhibited the same cracking pattern in the first phase. However, specimens 0 and 2 developed hairline flexural cracks in phase 2, while specimen 1 had none. The development of cracks in phase 2 was indicative of the robustness of the structural integrity of specimens 0 and 2 across the phaseline. Therefore, it can be deduced that delaying the application of the traffic-induced vibration until the final setting of the second-phase deck resulted in specimen 2 (phased-constructed) exhibiting similar behavior as specimen 0 (non-phased constructed).

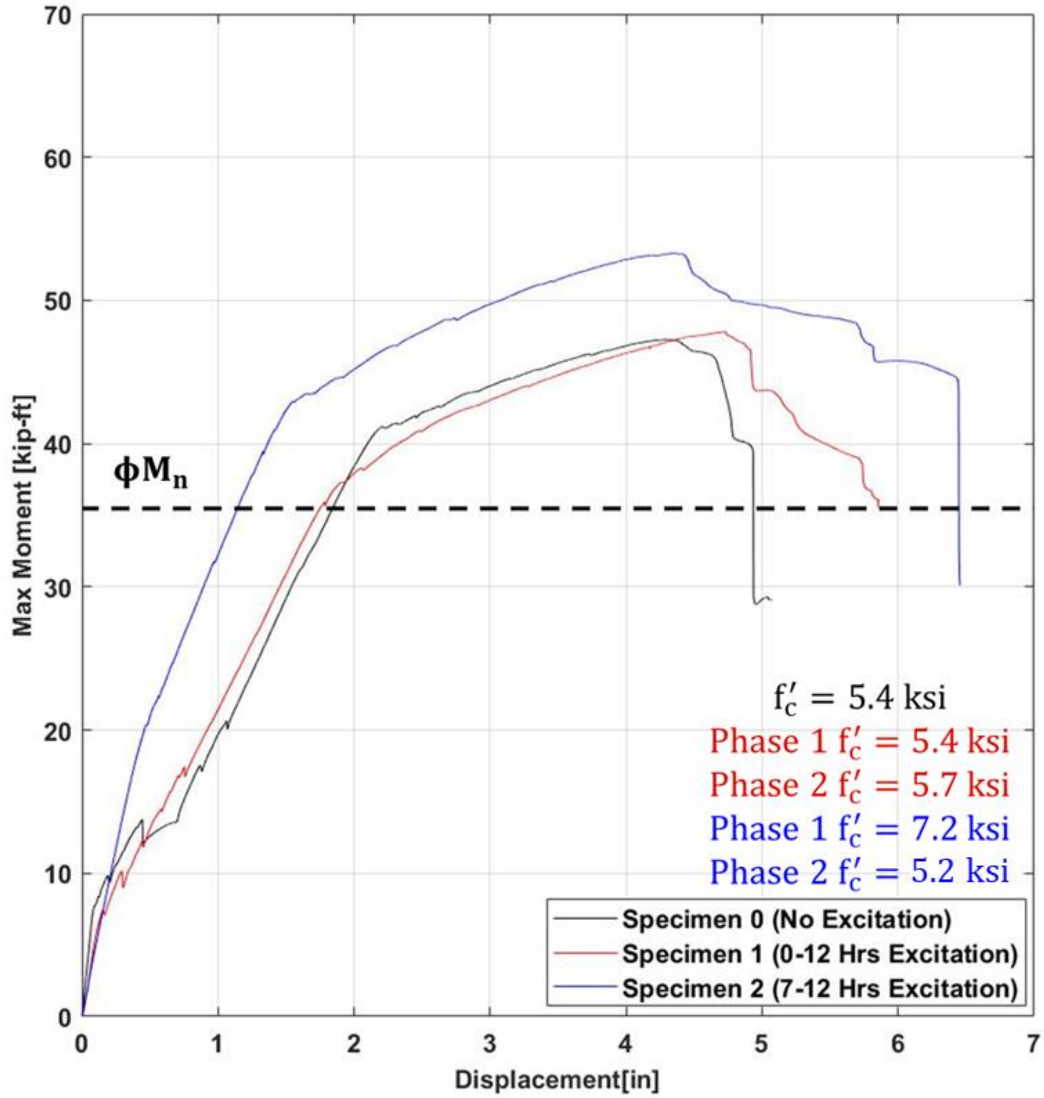


Figure 5. 14 Maximum moment as a function of actuator displacement for monotonic tests.

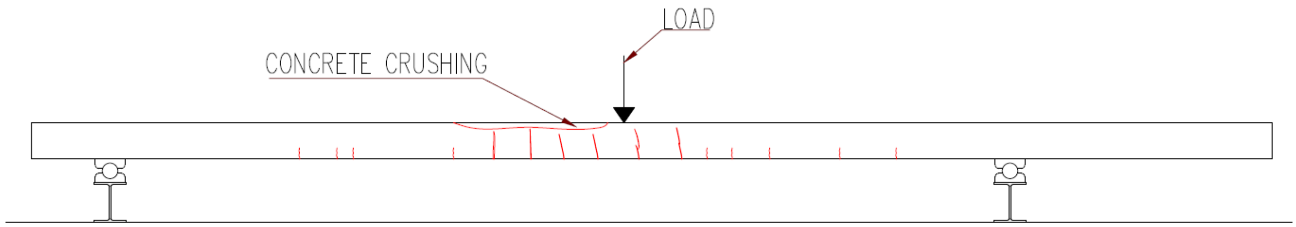


Figure 5. 15 Crack map of Specimen 0 after the monotonic test.

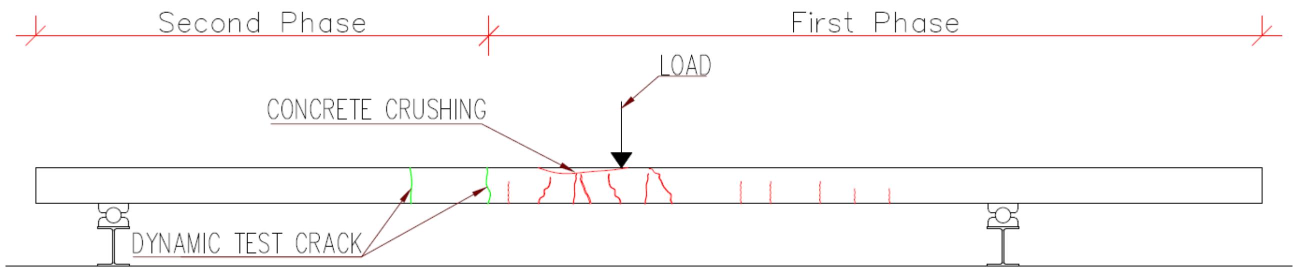


Figure 5. 16 Crack map of Specimen 1 after the monotonic test.

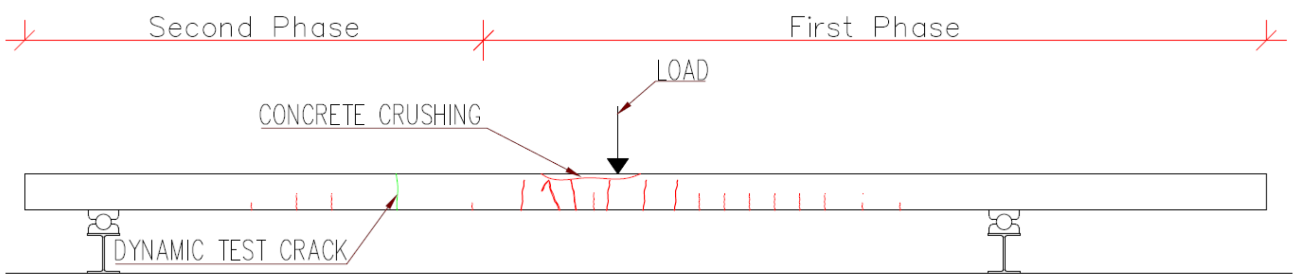


Figure 5. 17 Crack map of Specimen 2 after the monotonic test.

Strain Patterns

For every specimen, the strain data collected during the monotonic test for the two instrumented lap splices were processed and averaged. Figures 5.18 – 5.20 show the processed strain data plotted versus the actuator displacement for specimens 0, 1, and 2, respectively. In Figures 1.17, 1.18, and 1.19, bar 1 refers to the first-phase reinforcing bar extended into the second-phase deck, while bar 2 refers to the second-phase reinforcing bar. The results showed that all the specimens developed reinforcing-bar yielding in the first-phase deck. However, only specimen 0 was able to develop reinforcing-bar yielding in the second-phase deck in the vicinity of the phaseline joint. Unfortunately, the strain results are not conclusive and robust conclusions on the effect of phased construction on the concrete-rebar bond strength in the lap splice region cannot be drawn.

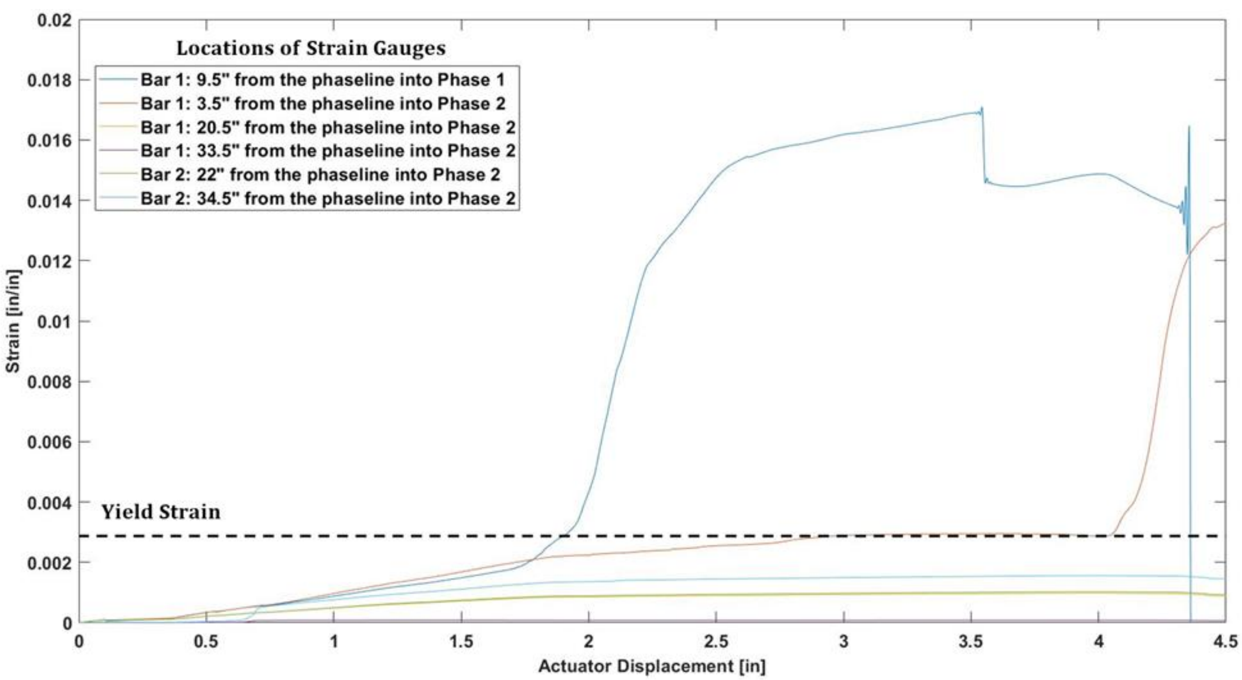


Figure 5. 18 Strain as a function of actuator displacement for Specimen 0.

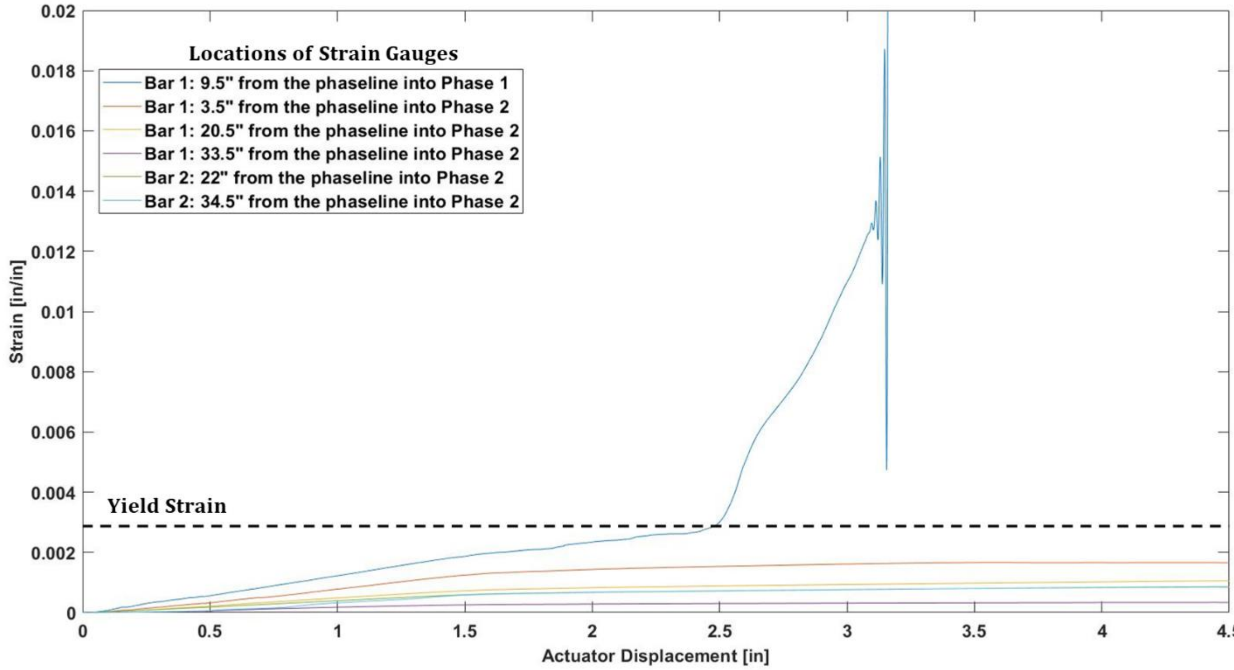


Figure 5. 19 Strain as a function of actuator displacement for Specimen 1.

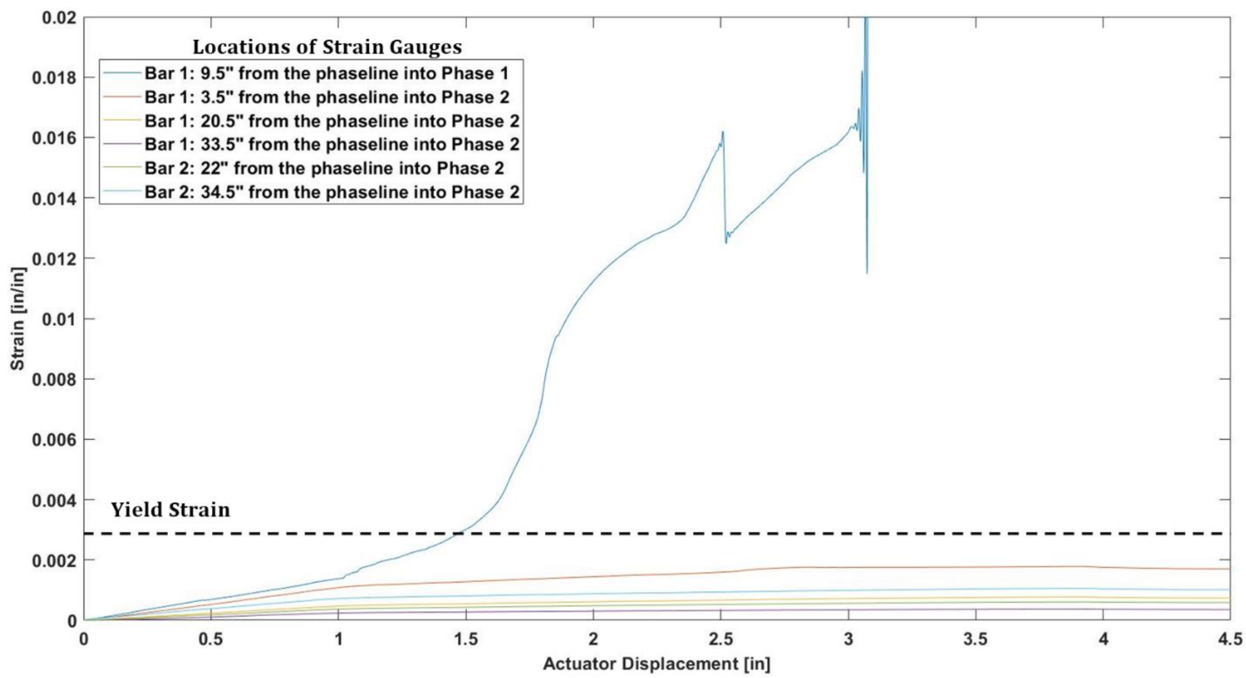


Figure 5. 20 Strain as a function of actuator displacement for Specimen 2.

Specimen Sectioning

After the monotonic tests, the second-phase deck of every specimen was cut at three locations for a comprehensive investigation of the effect of phased construction on the concrete-rebar interface in the second-phase deck. Figure 5.21 shows the cutting locations in the second-phase deck, where cutting occurred in the vicinity of the phaseline, dynamic cracks, and the end of the lap splice. After sectioning the specimens as needed, cross-sectional photos at either side of the cut sections were taken for every specimen, as shown in Table 5.4. The photos showed there were no voids nor cracks in the vicinity of the reinforcing bars that could be directly attributed to the impartment of simulated traffic-induced vibration during the early-age curing of the second-phase decks. While a few photos of the phased-constructed specimens (i.e., specimens 1 and 2) showed some small voids in the vicinity of the reinforcing bars, it could not be related to phased construction, as similar voids were seen around some of the reinforcing bars of the non-phased-constructed specimen (i.e., specimen 0).

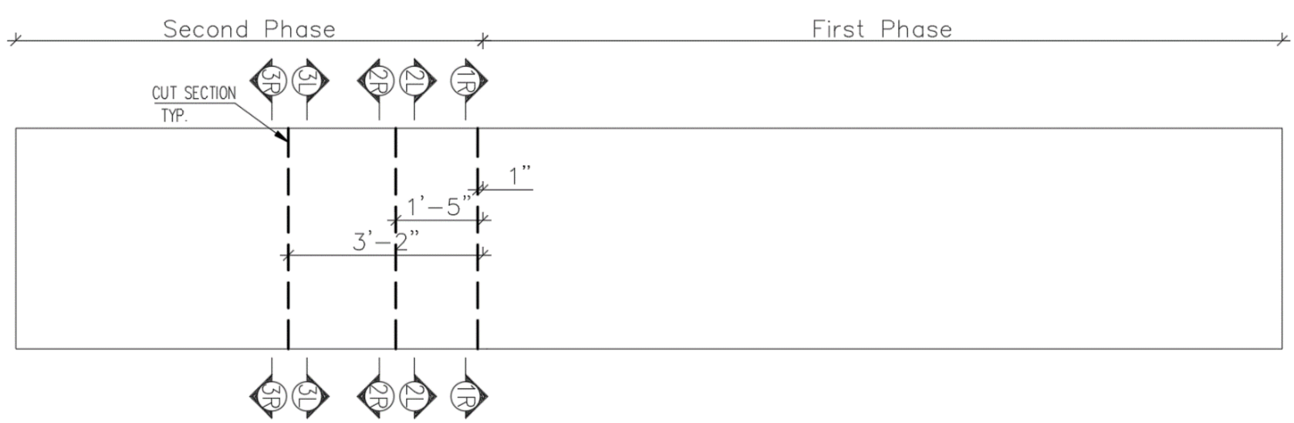
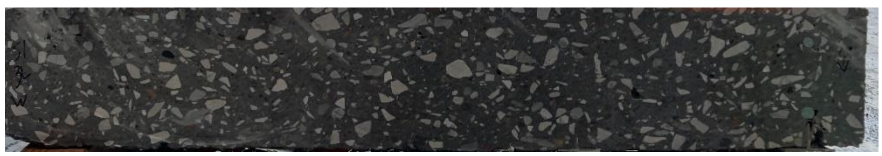
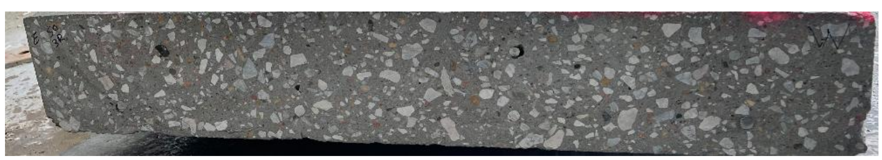
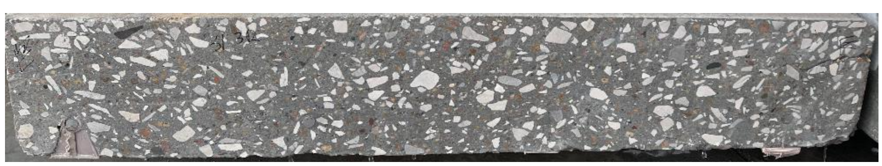


Figure 5. 21 Locations of cutting planes for Specimens 0, 1, and 2.

Table 5. 4 Photos of cut cross-sections of Specimens 0, 1, and 2.

Cross-section	Specimen	Photo
1-R	0	
	1	
	2	
2-L	0	
	1	
	2	
2-R	0	

	1	
	2	
3-L	0	
	1	
	2	
3-R	0	
	1	
	2	

CHAPTER 6 – CONCLUSIONS AND RECOMMENDATIONS

6.1 SUMMARY

Due to the current state of deteriorating infrastructure in the region and country, the number of bridges in the state and in the country in need of replacement is expected to increase. However, the complete closure of a traffic route to allow for the construction of a new bridge is often not feasible - particularly in rural Nebraska, in which truck traffic is limited to few routes and is critical to the economic vitality of the state. To address this need and reduce detours, phased construction has become a very prevalent practice for bridge replacement, which allows the bridge to remain partially open to traffic throughout construction. While phased construction can be interpreted as a very broad term, herein it is defined as the situation where one segment of the bridge is constructed adjacent to an existing segment. Typically, the number of traffic lanes is reduced to allow for partial demolition of the bridge. Then, a new segment of the bridge is constructed – termed the first phase. Once traffic is re-routed to the new segment, the remaining bridge is demolished and replaced – the new construction termed the second phase. In most situations, rebar extends from the first phase deck and is spliced to the second phase deck reinforcement prior to pouring of the deck.

Second-phase decks cure under the effect of traffic-induced vibration transmitted from the adjacent first-phase deck through reinforcement, formwork, or cross-diaphragms, which raises concerns about the structural integrity and durability of those decks. Traffic-induced vibration causes the reinforcing bars extended from the first phase and embedded into the second phase to have differential movements during the second phase curing, which can potentially lead to an accelerated degradation of the concrete-reinforcement bond in the vicinity of the construction joint (i.e., phase line). Therefore, the primary goal of this study is to generate a fundamental understanding of the transmission of traffic-induced vibration, the extent of degradation on phased construction bridge decks, and the impact of potential mitigation measures. To this end, this project was largely conducted through three approaches: 1) a survey of state DOTs, in which current practices surrounding phased construction and observations of premature deck deterioration were gathered and synthesized; 2) field monitoring of phased construction bridges, in which the response of phased construction bridges was measured before, during, and after second phase bridge deck pours for two case study structures in Nebraska; and 3) full-scale experimentation, in which 3 full-scale strip bridge specimens were cast in a controlled laboratory environment under varying durations of traffic-induced vibration.

6.2 CONCLUSIONS AND RECOMMENDATIONS

While little conclusive evidence regarding correlations of premature deterioration was captured through the survey of state DOTs, premature deterioration of second-phase decks was largely observed by those state DOTs that utilize phased construction, but rarely or never with a closure pour. In addition, the survey concluded that practices in Nebraska are fairly typical of many state DOTs, which makes the further conclusions and recommendations of this project broadly applicable beyond state borders. Field monitoring of two phased-construction bridges identified that rebar may displace up to 0.1 inch within the curing second-phase decks due to traffic-induced vibration, which is well above the thresholds seen in previous small-scale experimentation to result in reduced bond strength. Furthermore, field monitoring was able to identify a critical window of curing for the second phase. The results revealed that the natural frequencies of the first and second phases converged after 6-7 hours from the pour completion of the second-phase deck. As a result, the effect of delaying traffic on the durability of the phased-construction bridges was tested experimentally. The experimental study results showed there was no reduction in moment-carrying capacity of the specimens as a result of traffic-induced vibration; however, it was observed that applying traffic-induced vibration after the final setting of concrete reduced the width and extensiveness of cracking in the second phase. Therefore, this project has identified traffic-induced vibration is a distinct source of premature deterioration and cracking in phased-construction bridges. Based on the outcomes of this study, it is highly recommended to fully close the first phase to traffic until the second phase reaches the final setting (e.g., 6-7 hours for Nebraska mix design 47 BD). Hence, the potential early-age deck deterioration and strength reduction associated with traffic-induced vibration can be avoided.

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APPENDIX A

Survey of Phased Construction Practices for Concrete Bridge Decks

In this survey, the term "Phased Construction" refers to the placement and/or curing of bridge deck concrete in one or more lanes while the remaining lanes are open to traffic.

What state DOT do you represent? ____

1. How often do you use phased construction?
   a. Not at all (0%)
   b. Rarely (0-25%)
   c. Sometimes (25-50%)
   d. Often (75-100%)
   e. Always (100%)

2. How often does phased construction include a third pour between the two phases of the bridge (closure pour)?
   a. Not at all (0%)
   b. Rarely (0-25%)
   c. Sometimes (25-50%)
   d. Often (75-100%)
   e. Always (100%)

3. a) What types of traffic restrictions are imposed during concrete curing, if any? Select as many restrictions as applicable.
   a. Speed limit for vehicles
   b. Load limit for trucks
   c. Close the lane nearest to the phase line
   d. None
   e. Other (please specify): ____

   b) How long are any traffic restrictions in place?
   a. 12 Hours or less
   b. 1 Day
   c. 2 Days
   d. 3-5 Days
   e. Other (please specify): ____

4. a) What types of construction operation restrictions (set by the contractor or DOT) are imposed during concrete curing, if any? Select as many restrictions as applicable.
   a. Limit on the use of heavy equipment on the new deck for a period of time following curing
   b. Limit on the use of heavy equipment near the phase line
   c. Restrictions on the use of vibration for concrete consolidation during placement
   d. None

e. Other (please specify): ______________________________

b) How long are these restrictions in place?
a. 1 Day
b. 2 Days
c. 3-5 Days
d. 6-10 Days
e. Other (please specify): ______________________________

5. a) Do the decks of phased construction bridges show more, less, or similar cracking compared to the decks of non-phased bridges?
   a. More cracking
   b. Less cracking
   c. Similar

   b) Which deck of a phased construction bridge exhibits the most cracking?
   a. Phase 1 deck (first pour)
   b. Phase 2 deck (second pour)
   c. Closure pour (between first and second pour)

   c) If known, how long after the bridge pour are the cracks initially observed?
   a. 2 Weeks
   b. 1-3 months
   c. 3-6 months
   d. 6-12 months
   d. Other (please specify): ______________________________
   e. Unsure

   d) How is the deck surface of the joint at the phase line treated?
   a. Sealed
   b. Left exposed
   c. Overlay (type): ________________________

6. Have you noticed any correlation between the bridge geometry, structural system (i.e., steel-girders, prestressed concrete girders, etc.), or rebar splicing method and the deck or closure pour degradation?
   a. Yes (please elaborate below)
   b. No

   Comments: ______________________________________

   ____

   ____

7. a) What requirements do you have for bridge deck curing? Select as many requirements as applicable.
    a. Using liquid curing compound
    b. Using burlap and soaker hoses (wet curing)
    c. None
    d. Other (please specify): ______________________________

   b) How long is the bridge deck curing process?
    a. 1-3 Days
    b. 3-7 Days
    c. 7-10 Days
    d. 10-14 Days
    e. Other (please specify): ______________________________

8. What is the standard rebar splice between the two phases of a phased (staged) construction bridge?
    a. Mechanical (using couplers)
    b. Lap splice (please specify the overlap length)
    c. Welded splice
    d. No splice
    e. Other (please specify): ______________________________

9. When are transverse diaphragms between the two phases connected?
    a. Prior to concrete placement
    b. After concrete placement
    c. Prior to concrete placement, but the fasteners are not fully tightened

10. Do you have any recommendations for the design or construction of bridge decks that could enhance their durability especially when phased construction is conducted?

11. Would you be willing to provide curing records, details of the standard concrete mix used in bridge decks in your state, and/or crack observations for your bridges so that a statistical analysis can be conducted by our research team? If so, please provide the contact information for the responsible person.

12. Do you have any other feedback for the research team?