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Effectiveness of Exterior-Beam Rotation-Prevention Systems for Bridge Deck Construction—Phase II

Prepared by
Li Hui
Faress Hraib
Miguel Vicente
Riyadh Hindi
Saint Louis University

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Effectiveness of Exterior Beam Rotation Prevention Systems for Bridge Deck Construction—Phase II

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16. Abstract

During bridge deck construction, construction loads are transferred through the overhang-formwork system into the girder system, resulting in significant torsional moments acting on the exterior girder. In some cases, these torsional moments can lead to excessive exterior-girder rotation, thin bridge decks, and cause potential safety and maintenance issues for the bridge. To resist these torsional moments, temporary rotation-prevention systems are commonly used during deck construction. The Illinois Department of Transportation (IDOT) recommended a new rotation-prevention system, which uses a transverse tie combined with a diagonal pipe to limit exterior-girder rotation. This system has been evaluated and validated to be effective, and efficient, based on the finite-element (FE) analysis in Phase I of this research project. In this study, the transverse tie–diagonal pipe system was evaluated through field instrumentation and FE analysis. Also, additional FE analysis was conducted to assess the effects of various parameters of this system, such as the dimensions of the members of this system, protective rubber at the end of pipes, and depth of connecting plates between the diaphragms and girders. Based on the results from this research, it is suggested that larger sections of ties and pipes should be used to ensure the stiffness of the temporary bracing system; and the protective rubber should be eliminated. Furthermore, a maximum ratio of unbraced length to girder depth (B/D ratio) of 4 is recommended when considering the effects of the diaphragm-to-girder connection.

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Members of the Technical Review Panel (TRP) were the following:

- Douglas Dirks, TRP Co-Chair, IDOT
- Mark Thomson, TRP Co-Chair, IDOT
- Dan Brydl, FHWA
- Joel Cumby, IDOT
- Josue Ortiz-Varela, IDOT
- Kevin Riechers, IDOT
- John Scheibal, IDOT
- Megan Swanson, IDOT

EXECUTIVE SUMMARY

The bridge deck is often designed to extend past the exterior girders to increase the width of the deck without increasing the required number of girders. During bridge deck construction, construction loads, mainly from the weight of the fresh concrete and the bridge deck finishing machine, are transferred through the formwork system to the exterior girder, resulting in significant torsional moments on that girder. In some cases, these torsional moments can lead to excessive exterior-girder rotation and cause potential safety and maintenance issues, such as deck-thickness loss or girder instability during construction. To resist these torsional moments and prevent excessive exterior-girder rotation, temporary rotation-prevention systems are commonly used during bridge deck construction. The Illinois Department of Transportation (IDOT) recommends using a transverse tie combined with a diagonal pipe as a temporary rotation-prevention system. This system has been evaluated and validated to be effective, and efficient, according to the finite-element (FE) analysis in Phase I of this research project. In this study, the transverse tie–diagonal pipe system was assessed through field instrumentation and FE analysis. Additional FE analysis was also conducted to evaluate the effects of different parameters of the temporary rotation-prevention system.

The objectives of the proposed research were to evaluate the effectiveness of the transverse tie–diagonal pipe system in limiting exterior-girder rotation. Also, current IDOT bracing policies and procedures were assessed; and some recommendations were proposed to minimize and prevent exterior-girder rotation during bridge deck construction. The specific objectives of the study included the following:

- Review the transverse tie–diagonal pipe rotation-prevention system proposed in Phase I of this study and determine its effectiveness.
- Validate the effectiveness of the specifications and details that came out of Phase I.
- Determine the ratio of unbraced length to girder depth (B/D) required for all various typical bridge beam depths to limit exterior-girder rotation.
- Develop comprehensive bracing requirements, including the B/D ratio, cross-frame/diaphragm spacing, and finishing-machine rail locations.

Four steel-girder bridges with different geometrical properties were selected and instrumented with multiple sensors to measure girder rotation, strain, and vertical deflection during deck construction. In all the bridges, the sensors, including tilt sensors and strain gages, were installed at various locations on the bridge, based on the results from preliminary FE analysis. The strain in the transverse ties and diagonal pipes for three of the bridges was measured to evaluate the effectiveness of the rotation-prevention system. Total stations were used to record changes in vertical deflections when the bridge deck finishing machine moved along the bridge. In this report, the field-monitoring results are reported in two forms: the maximum and the stable. The maximum value indicated the data from the sensors during deck construction when the bridge deck finishing machine moved over the sensor locations, while the stable value indicated the data after the completion of the concrete pouring of the bridge deck, and the superstructure is subjected to the fresh concrete weight only.

An analytical study was also conducted using the FE software SAP2000. FE models were established for all the instrumented bridges, based on the bridge plans provided by IDOT. In general, the bridge girders and diaphragms/cross-frames were modeled using thick-shell elements, while the members of the rotation-prevention systems were simulated using 3D frame elements. The finishing machines, as well as the fresh concrete, were simplified as loads acting on the bridge girders and formwork systems. The FE models were validated based on the field data and used further in the study to predict exterior-girder rotation in cases that were difficult to evaluate in the field.

Because different sizes of transverse ties and diagonal pipes, as well as protective end rubber, were observed in the construction field, additional FE analysis was performed to assess the effects of those parameters on exterior-girder rotation. Different sizes of transverse ties and diagonal pipes were evaluated in the FE models. The results indicated that the dimensions of the members of temporary rotation-prevention systems have very limited effect on exterior-girder rotation. However, based on the findings of this study and current practice, it is suggested to use a minimum #4 transverse tie bar and a minimum 1.5 inches round or square pipe strut, to ensure the stiffness of the system. Moreover, the maximum forces in the steel hanger was determined to be 3,000 lb, according to FE analysis and field monitoring. In addition, the use of the protective rubber at the end of the pipes significantly reduced the effectiveness of the rotation-prevention system, according to the FE analysis.

Moreover, the FE analysis results showed that the depth of the diaphragm-to-girder connection should be considered when determining the B/D ratio. When partial-depth connecting plates are used between the diaphragm and the girder, the diaphragms are less effective in resisting exterior girder rotations and should be discounted or ignored when determining the effective B/D ratio. However, full depth cross-frames and diaphragms with full-depth connecting plates can be included when determining the B/D ratio. A maximum B/D ratio of 4.0 is recommended in this report to limit exterior-girder rotation.

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CHAPTER 1: INTRODUCTION

1.1 OVERVIEW

The bridge deck is generally designed to extend past the exterior girders to increase the width of the deck without increasing the required number of girders. The construction of the bridge deck is often finished with overhang brackets generally spaced 3 to 6 ft apart along the exterior girder to support the construction loads coming from the fresh concrete, the bridge deck finishing machine, and other construction live loads (shown in Figure 1). In some cases, these loads on the overhang brackets can result in torsional moments acting on the exterior girders, causing excessive exterior-girder rotation. This rotation may cause bridge failure due to global and local instabilities during deck construction (Gupta et al., 2006; Lackey, 2017; Mohammadi et al., 2016; Shokouhian and Shi, 2015; Winkler et al., 2017). Furthermore, the rotation in the exterior girder may cause some potential maintenance issues, such as a thin deck with reduced concrete cover over reinforcement bars, and may lead to long-term corrosion and cracking in the deck (Ashiquzzaman et al., 2016; Clifton and Bayrak, 2008; Fasl, 2008).

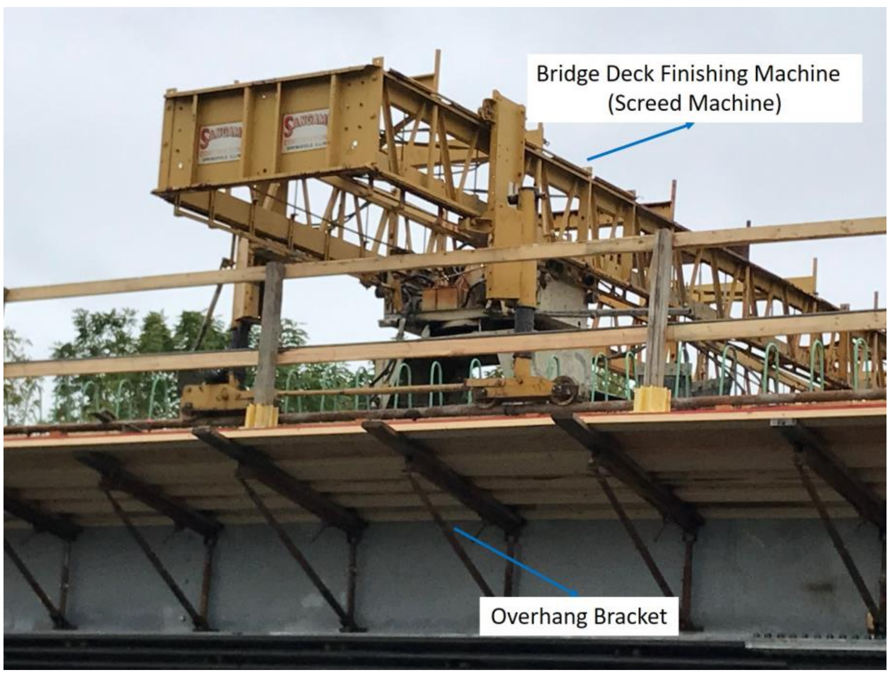


Figure 1. Construction of deck overhang.

The maximum rotation of the exterior girder can be expected to occur in the middle of two consecutive transverse cross-frame or diaphragm bracings during the concrete deck pouring process when the finishing machine is located midway between the two bracings. The torsional moments in the exterior girder are mainly caused by the loads from the fresh concrete and the bridge deck finishing machine, as illustrated in Figure 2. The concrete between the girders may also affect the total torsional moments, but this contribution is negligible since the hanger bolts do not transfer moment to the girder system.

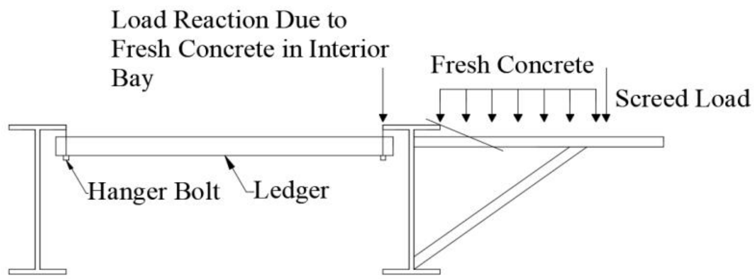
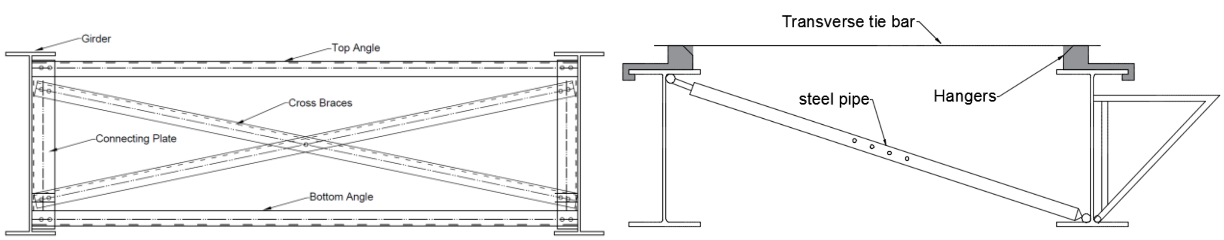


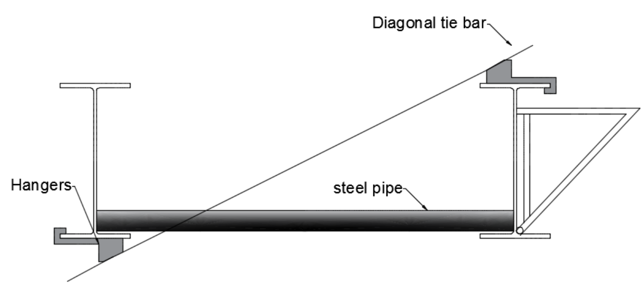
Figure 2. Loads on the exterior girder during deck construction.

To resist the torsional moments introduced by construction loads, temporary bracing systems are commonly applied during deck construction. In Phase I of the study, several temporary bracing systems were proposed and validated with a set of laboratory experiments (Figure 3). Compared with the traditional temporary bracing system using transverse ties and timber blocks, these new systems may reduce more than 70% of the rotation, according to the finite element (FE) analysis. However, an evaluation from the field bridge implementation is necessary to be conducted to ensure that the bracing system is appropriately applied and effective.



(a) Intermediate cross-frame

(b) Diagonal pipe–transverse tie system



(c) Transverse pipe–diagonal tie system

Figure 3. Recommended temporary bracing systems from Phase I (Ashiquzzaman et al., 2016).

The Illinois Department of Transportation (IDOT) funded this research investigation, entitled Effectiveness of Exterior-Beam Rotation-Prevention Systems for Bridge Deck Construction—Phase II, to identify the effectiveness of the improved temporary bracing system and to prevent, or minimize, exterior-girder rotations during bridge deck construction. In this report, an evaluation is conducted for the improved bracing system; and recommendations are presented for improving the bridge deck-overhang construction.

The remainder of this chapter provides a discussion on the scope of the research as well as a brief outline of this report.

1.2 PROJECT OBJECTIVES

The specific objectives of the study are as follows:

- Review the transverse tie–diagonal pipe bracing system developed in Phase I of the study and evaluate its effectiveness.
- Validate the effectiveness of the specifications and details that came out of Phase I.
- Determine the requirement of the ratio of unbraced length to girder depth (B/D) for all various typical bridge beam depth to limit exterior-girder rotation.
- Develop comprehensive bracing requirements including B/D ratio, cross-frame/diaphragm spacing, and finishing-machine rail locations.

1.3 SCOPE

The scope of this research includes field monitoring and FE analysis. Four bridges were monitored during deck construction as part of field testing. The bridges included steel wide-flange girder bridges and medium-depth steel plate girder bridges. The field data were used to assess the effectiveness of the proposed rotation-prevention system and validate the FE models. The validated FE models were used to conduct a further study to improve understanding of the behaviors of the temporary rotation-prevention system during deck construction. The effects of the dimensions of the rotation-prevention system, the end protective rubber, and the connecting plates between diaphragms/cross-frames and girders were investigated in this study. Also, a general B/D ratio for different beam sections was evaluated and validated. The results of this report can be used as a conservative method to prevent excessive exterior-girder rotation during deck construction.

1.4 CHAPTER ORGANIZATION

This report consists of the following six chapters:

Chapter 1: Introduction introduces the research project, including the research scope and objective.

Chapter 2: Background and Literature Review provides background information on the construction of the bridge deck overhang, including the formwork system, finishing machine, and load application. The background information and description of different types of rotation-prevention systems, as well as the concept of B/D ratio, are presented.

Chapter 3: Field Monitoring and FE Analysis presents the field-monitored data from four selected bridges. Also, FE analysis was performed using the commercial software SAP2000. FE models were validated using the field data.

Chapter 4: Finite-Element Analysis of Additional Bridges presents FE analysis of two bridges that were not field monitored during construction. FE models for these two bridges were developed to examine further the effectiveness of the pipe–tie system.

Chapter 5: Assessment of B/D Ratio to Limit Exterior-Girder Rotation discusses the effects of diaphragm connections on determining the B/D ratio. A general B/D ratio was evaluated and recommended based on the FE analysis results. This chapter also discusses the effects of different parameters on preventing exterior-girder rotation, including the size of pipes, ties, and the protective rubber.

Chapter 6: Conclusions and Recommendations presents a summary of significant findings from this research, along with recommendations for design and construction of the bridge deck in the future.

CHAPTER 2: BACKGROUND AND LITERATURE REVIEW

2.1 CONSTRUCTION OF DECK OVERHANG

2.1.1 Formwork System

During bridge deck construction, a formwork system is often used to shape the fresh concrete, as well as support the construction loads. Although several manufacturers, such as Dayton Superior and Meadow Burke, offer different types of formwork systems, the formwork system typically includes interior formwork supporting the fresh concrete between two girders, and the overhang formwork for the overhang portion of the bridge deck (Dayton Superior, 2017; Meadow Burke, 2018). The interior formwork mainly consists of plywood, ledge timbers, and hanger bolts, as shown in Figure 4. The fresh concrete of the bridge deck is placed directly on top of the plywood, which is supported by the ledge timbers. The hanger bolts connecting the ledge timbers with the bridge girders transfer the construction loads to the bridge girder system.

Figure 4. Interior formwork.

The overhang formwork system, shown in Figure 5, consists of plywood sheathing and timber joists supported on steel brackets usually placed 3 to 6 ft apart along the exterior girder (Clifton and Bayrak, 2008; Dayton Superior, 2017). The overhang brackets are typically connected to the top flange of the exterior girder using coil rods and react against the bottom of the girder web.

Figure 5. Overhang formwork.

2.1.2 Bridge Deck Finishing Machine

Phase I of this study showed that the weight of the bridge deck finishing machine is one of the primary contributors to excessive rotation of the exterior girder (Ashiquzzaman et al., 2016; Yang et al., 2010). The bridge deck finishing machine, which is a truss spanning the width of the bridge, strikes off the freshly poured concrete and smooths the concrete surface by moving a roller unit back and forth across the deck. Some finishing machines that are commonly used in the state of Illinois are the Bidwell 3600 and Bidwell 4800, as shown in Figure 6. The basic specifications of both models are shown in Table 1.

(a) Bidwell 3600

(b) Bidwell 4800

Figure 6. Bridge deck finishing machines.

Table 1. Specifications of Bridge Deck Finishing Machines

	Bidwell 3600	Bidwell 4800
Operating Weight (lb)	7,760	10,473
Standard Paving Width (ft)	32.5	32.5
Minimum Paving Width (ft)	8	12
Maximum Paving Width (ft)	86	116

Figure 7. Screed-machine rail located outside the exterior girder on the overhang.

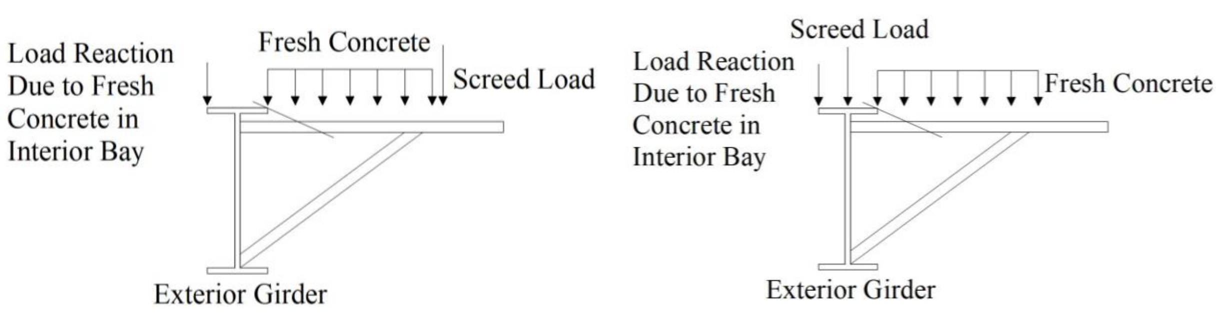
Figure 8. Screed-machine rail located at the center of exterior girder.

To cover the largest amount of the bridge surface, the bridge deck finishing machine is often supported by screed rails located outside of the bridge deck overhang, as shown in Figure 7. The weight of the bridge deck finishing machine transfers to the exterior girder through the overhang bracket, which produces torsional moments acting on the exterior girder. These torsional moments may cause excessive rotation in the exterior girder, creating potential safety problems during the construction and maintenance issues during the service life of the bridge (Ashiquzzaman et al., 2016; Hraib et al., 2018; Hui et al., 2018). To minimize the torsional moments caused by the bridge deck finishing machine on shallow beam bridges, IDOT specifies that the screed rail should be installed at the center of the top flange on each exterior girder when the depth of the girder is less than 30 in. An example of the screed rail located on the top, center of an exterior girder is shown in Figure 8. In this case, the weight of the bridge deck finishing machine transfers directly to the exterior girder, minimizing additional torsional moments from the finishing machine on the exterior girder.

It is important to differentiate between the wet concrete weight and the finishing machine weight. The concrete weight is permanently added to a bridge. However, the finishing machine load is temporary and can vary depending on the bridge width, its operating angle, and the type and specifications of the machine. In skewed bridges, the finishing machine can be operated parallel or perpendicular to the skew. Therefore, the length of the machine is subjected to variation, and the weight of the machine varies as well.

2.1.3 Load Application

Although several types of construction loads are applied to the exterior girder through the overhang brackets during construction of the bridge deck, Phase I of this study demonstrated that the weight of the finishing machine and fresh concrete are the primary loads that cause rotation of the exterior girder. Figure 9a shows the simplified load application when the screed rail is located outside the overhang portion of the bridge deck. In this case, the torsional moments on the exterior girder are created by the weight of the fresh concrete and the finishing machine. Figure 9b shows the simplified load application when the screed rail is located at the center of the exterior girder. In this case, the torsional moment on the exterior girder are primarily from the weight of the concrete only.



(a) Bridge deck finishing machine outside the overhang (b) Bridge deck finishing machine on the top of exterior girder

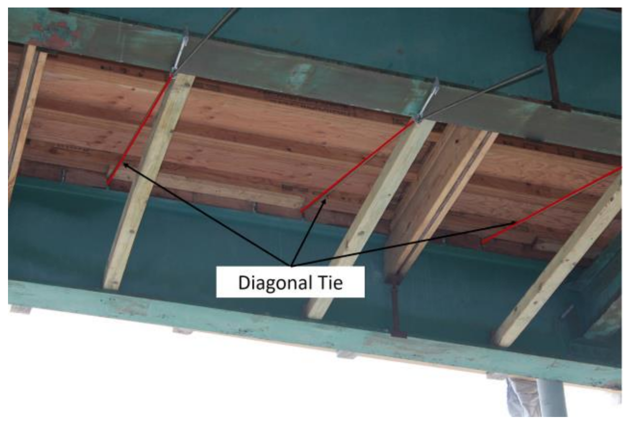
Figure 9. Construction loads when the screed rail is outside the overhang.

2.2 ROTATION-PREVENTION SYSTEM

2.2.1 Traditional Rotation-Prevention Systems

The traditional rotation-prevention systems widely used in the state of Illinois are a combination of transverse or diagonal ties with timber blocks, as shown in Figure 10. However, these combinations often lose their effectiveness when quality control is overlooked and the systems are improperly installed (Ashiquzzaman et al., 2016; Ashiquzzaman et al., 2017). Phase I of this study showed that the timber block is often not an effective temporary bracing due to the improper shimming. Due to the gap between the timber block and the girder, a part of the rotation in the exterior girder occurs before the timber block starts working. Additionally, inadequate tightening of the transverse ties and the improper angling of the diagonal ties can mitigate the effectiveness of those rotation-prevention systems.

(a) Transverse tie



(b) Diagonal tie

(c) Timber block

Figure 10. Traditional rotation-prevention systems.

2.2.2 Improved Rotation-Prevention System

In Phase I of this study, three types of improved rotation-prevention systems—the intermediate cross-frame, the transverse tie–diagonal pipe, and the transverse pipe–diagonal tie system—were developed and evaluated. The three systems were demonstrated to be effective and efficient alternatives to traditional rotation-prevention systems. However, for economic advantage and ease of installation, the transverse tie–diagonal pipe system (shown in Figure 11) was selected to be implemented in Illinois for the construction of the bridge decks.

Figure 11. Transverse tie–diagonal pipe system.

The basic concept of the transverse tie–diagonal pipe system is to provide additional restraints in the transverse direction for the exterior girder so as to prevent exterior-girder rotation. When the exterior girder attempts to rotate, the steel pipe reacts against the bottom of the girder's web, preventing the bottom of the girder from moving inward toward the interior girder; the transverse tie bars on the top of the girder reduce the outward transverse deflection of the top of the girder.

2.3 RATIO OF UNBRACED LENGTH TO GIRDER DEPTH (B/D RATIO)

Several studies have demonstrated that the B/D ratio is effective for evaluating exterior-girder rotation (Ashiquzzaman et al., 2016; Hraib et al., 2018; Hui et al., 2019). Increasing the number of temporary rotation-prevention systems, which reduces the unbraced length (correspondingly decreasing the B/D ratio), results in reduced girder rotation during the bridge deck construction. Because the girder depth is directly related to the capacity of a girder to resist torsional moment, the rotation of the exterior girder can be decreased when using a deeper girder section. However, this is not a very good design strategy because the total cost of a bridge increases once a deeper girder section is selected. The most effective way to reduce the B/D ratio is to reduce the unbraced length. The unbraced length and girder depth for calculating the B/D ratio are illustrated in Figure 12.

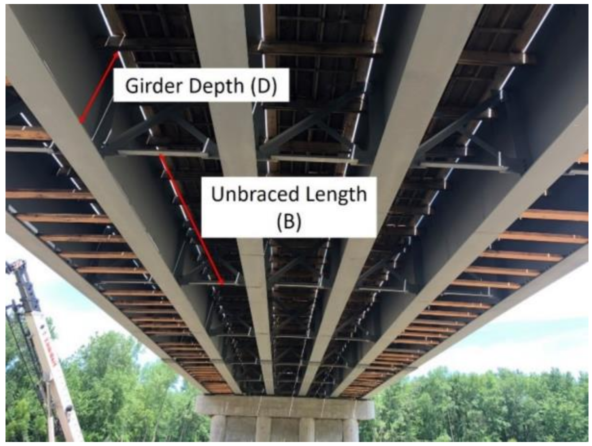


Figure 12. Unbraced length (B) and girder depth (D).

(a) Cross-frame with full-depth connecting plate

(b) Diaphragm with angle steel connection

(c) Diaphragm with partial-depth connecting plate

(d) Diaphragm with full-depth connecting plate

Figure 13. Different types of permanent bracing systems and connections.

The types of bracing systems and the connections between these systems and the girders may also influence the B/D ratio (Hraib et al., 2018, 2019). Four types of permanent bracing systems widely used in bridge design are shown in Figure 13. The cross-frame (shown in Figure 13a) typically braces the entire depth of the girder section and provides adequate stiffness to prevent exterior-girder rotation. However, when diaphragms are used, the effectiveness of the bracing system at resisting transverse rotation strongly depends on the relative depth between the girder and the depth of the connecting plates. When the diaphragms are connected to the girders using steel angles (Figure 13b) or partial-depth connecting plates (Figure 13c), distortion of the girder web may occur, resulting in different rotation values between the web and flanges of a girder. In this case, the top flanges may still suffer a transverse rotation greater than the allowable limit. However, when using a full-depth connecting plate between the diaphragms and girders (shown in Figure 13d), the diaphragms provide resistance to the transverse rotation of the girders because the connection plates are welded to the web and flanges. Therefore, it is necessary to take the types of bracing systems and connections between the bracing system and the girder into consideration when determining the B/D ratio.

CHAPTER 3: FIELD MONITORING AND FE ANALYSIS

3.1 GENERAL INFORMATION ABOUT THE BRIDGES

Four steel-girder bridges were selected and instrumented with multiple types of sensors to monitor their behavior during deck construction. The diagonal pipe–horizontal tie system, recommended in Phase I of this research project, was implemented in three of the selected bridges. For the other bridge, Lawrence Bridge, no temporary rotation-prevention system was applied due to its small B/D ratio. Some geometric properties and construction details of the bridges are shown in Table 2.

Table 2. Properties of Field-Monitored Bridges

Bridge	Lawrence Bridge	Ford Bridge	Madison Bridge	McDonough Bridge
Contract No.	74180	66C84	76G56	68215
Structure No.	051-0064	027-0104	060-0349	055-0097
Beam Type	66″ PL Gdr	W27	40″ PL Gdr	W36
Skew	None	14^{o}	None	15^{o}
No. of Spans	3	3	3	3
Span Length	162′ + 195′ + 162′	57′-5″ + 74′-8″ + 57′-5″	77′ + 96′ + 77′	65′ + 105′ + 65′
Overhang Width	3′-8″	2′-11″	3′-5.5″	2′-7″
Girder Spacing	7′-2″	6′-8″	7′-3″	6′
Diaphragm Type	Cross-frame	C12x25	C15x40	C15x40
Maximum Diaphragm Spacing	19′-9″	17′-4″	23′-8″	25′
Finishing Machine Type	Finishing machine	Finishing machine	Finishing machine	Finishing machine
Finishing Machine Location	On overhang	On exterior girder	On overhang	On overhang
Temporary Bracing Type	None	Diagonal pipe–horizontal tie	Diagonal pipe–horizontal tie	Diagonal pipe–horizontal tie
Diagonal Pipe Section	N/A	HSS2.5X0.25	HSS2.5X2.5X0.25	HSS1.66X0.14
Horizontal Tie	N/A	#5 bar	#5 bar	#4 bar
Temporary-Bracing-System Spacing in the Field	N/A	8 ft	8 ft	8 ft
B/D Ratio	3.95	3.56	2.4	2.67

3.2 SENSORS AND GAGES USED IN FIELD MONITORING

3.2.1 Tilt Sensor

Tilt sensors with high accuracy (model CXTLA02 manufactured by MEMSIC Incorporation) were used to measure rotation of bridge girders during deck construction. The angular range of the tilt sensor is ±20°, and the resolution is 0.03° rms. These sensors were mounted onto open aluminum boxes that could be easily attached to the steel girder, as shown in Figure 14. As the sensor measures the rotation in both the transverse and longitudinal directions, caution must be used during installation to maintain the correct orientation. The transverse direction of the sensor recorded the rotation along the width of the girder, while the longitudinal direction of the sensor measured the rotation along the length of the bridge. The tilt sensors were placed in the same pattern and orientation for all monitored bridges to ensure consistency in data collection.

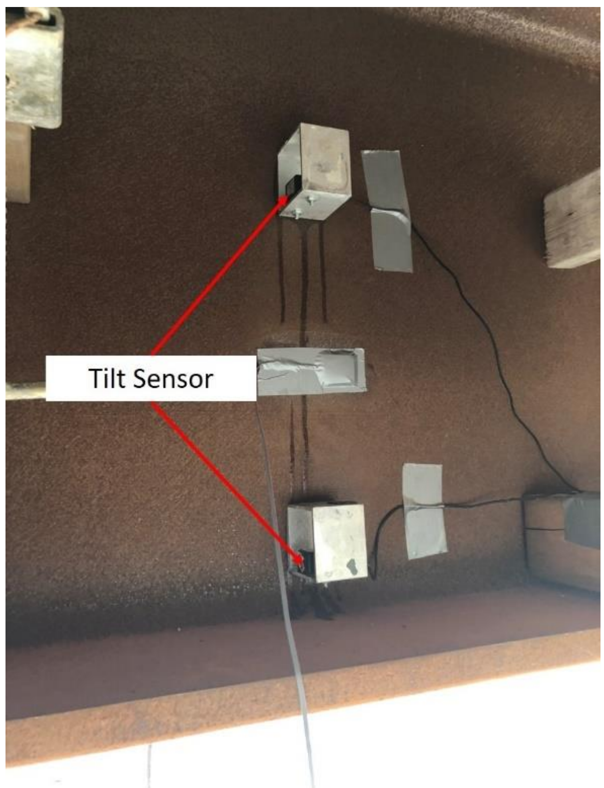


Figure 14. Tilt sensors with open aluminum boxes.

The rotation values recorded by the tilt sensors can be either positive or negative. The sign convention in this report considers inward rotation as negative (shown in Figure 15) and outward rotation as positive (shown in Figure 16). Along the span of each monitored bridge, three sections were chosen for installing the tilt sensors at both the center and the bottom of the web.

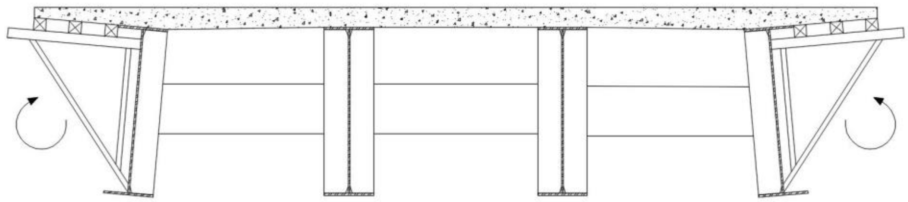


Figure 15. Inward rotation of the exterior girder.

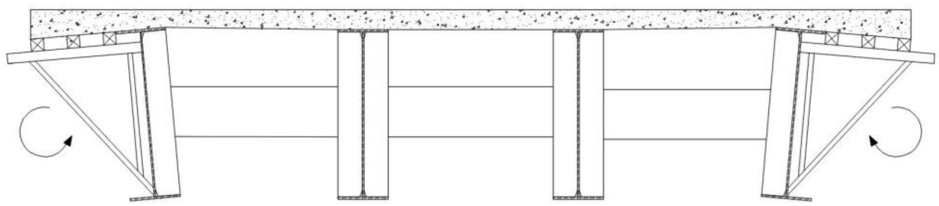


Figure 16. Outward rotation of the exterior girder.

The rotation values of the exterior girders were recorded by tilt sensors in a time-series format with a sample rate of 30 seconds. The simple moving average (SMA) method was used to reduce fluctuation and to smooth the recorded field data. The time-series and the SMA data are shown in Figure 17. The moving average method utilizes Equation (1),

$$SMA = \frac{1}{n}\sum_{i=0}^{n-1} P_i \tag{1}$$

where P_i is the data at time *i,* and *n* is the number of data.

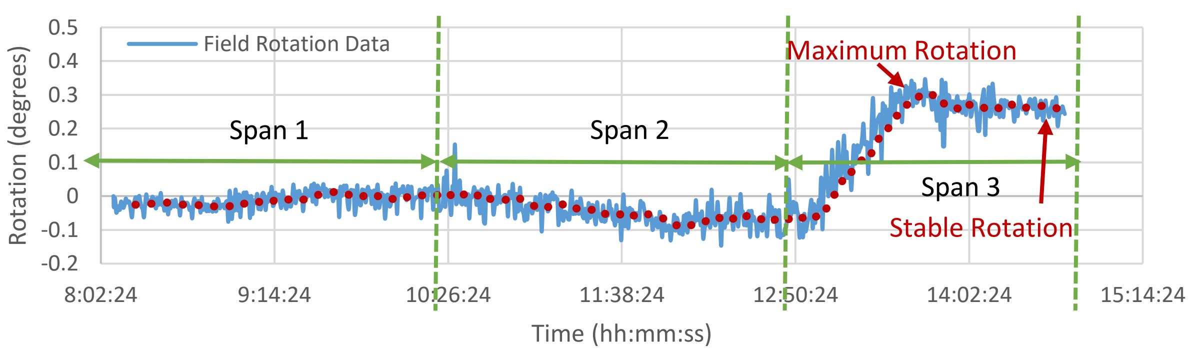
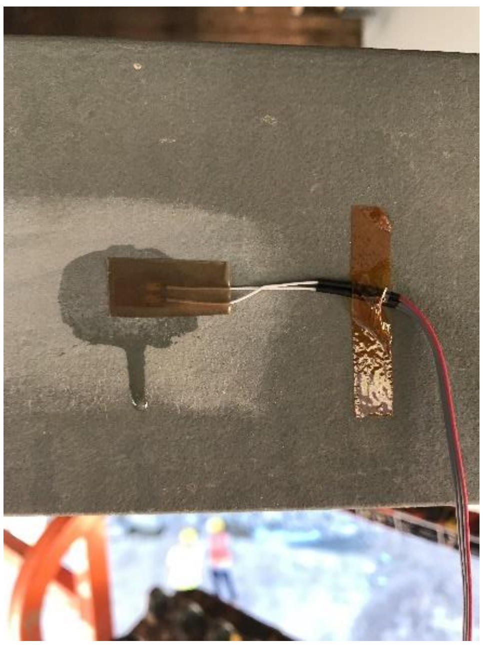


Figure 17. Time-series of rotation data and simple moving average.

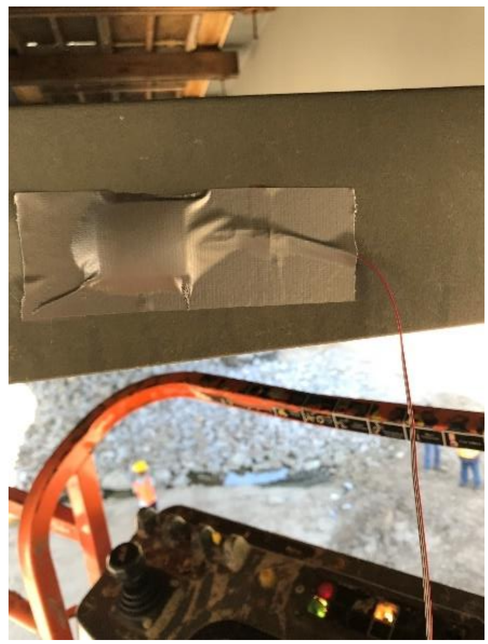
An example of the maximum and stable rotations during a pouring process of the concrete deck slab are illustrated in Figure 17. The maximum rotation occurs when the finishing machine is located at a particular section and the fresh concrete is poured up to that section. The stable rotation is recorded after the finishing machine has passed, and the finishing machine load is removed or no longer influencing rotation, and the bridge girders are subjected to only the fresh concrete weight. Therefore, the exterior girders remain at the stable rotation value after the hardening of the concrete deck.

3.2.2 Strain Gages

Foil strain gages (CEA-06-125UN-350 manufactured by Micro-Measurements) were used to measure strain values during deck construction. Performance of the strain gages is sensitive to moisture and can easily be affected during construction procedures. Therefore, self-fusing tape and coating compounds were used to protect the strain gages from potential external distress. The installation of the strain gages and the protective coating are shown in Figure 18. The strain gages were also placed on the temporary bracing system (transverse tie–diagonal pipe system), diaphragms, and girders. Figure 19 illustrates the installation process of straing gages on diaphragms.



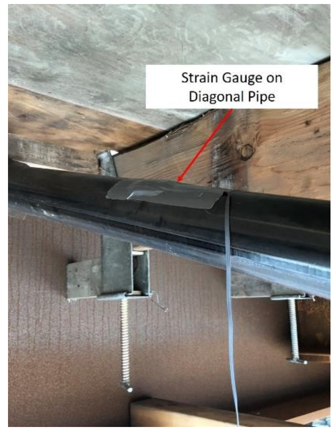
(a) Strain gage



(b) Protective coating

Figure 18. Strain-gage installation and protection.

Similar to the rotation values recorded using the tilt sensors, the strain had two significant values: the maximum and stable values. The maximum strain was recorded when the finishing machine was located at the strain gage location, and the stable strain was measured when only the fresh concrete load was applied to the bridge. These two values can be clearly identified from the time-history graph of the strain gage shown in Figure 20.



(a) Strain gage on transverse tie (b) Strain gage on diagonal pipe

(c) Strain gage on diaphragm

Figure 19. Locations of strain gages.

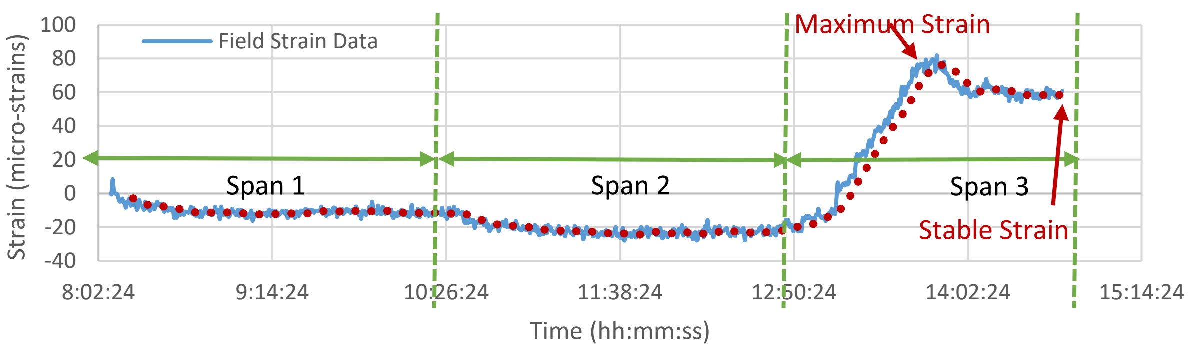


Figure 20. Time-series of strain data and simple moving average.

3.2.3 Total Station

Three total stations were utilized for each bridge to measure vertical deflections of exterior girders, as shown in Figure 21. Two prisms (reflectors) were employed to record deflections of the overhang formwork and the girder at each instrumented section. Additionally, a reference point was also established using a single prism located at a steady point (shown in Figure 21c). The main purpose for using the total station readings was to measure deflections of the exterior girders during the deck-slab construction and to validate the FE models.

(a) Total station

(b) Location of prisms

(c) Reference point

Figure 21. Deflection measurement.

Once the finishing machine started to move along the bridge, deflections were measured every 15 minutes. As each total station was assigned to measure just one specific section, all three total stations were able to record the readings simultaneously. After the deck pour was completed, at least two extra readings were obtained to ensure that deflection data were stable. Figure 22 shows an example of how deflection data would appear at the location of a prism.

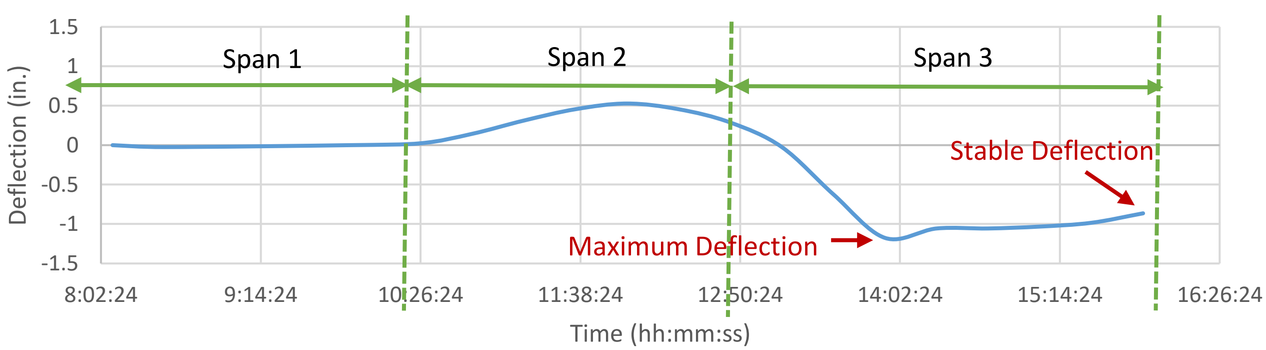


Figure 22. Field data for deflection measurement for one prism.

3.3 FIELD MONITORING OF BRIDGES

3.3.1 Lawrence Bridge

The Lawrence Bridge consists of six steel-plate girders with a web depth of 66 in. The permanent lateral bracing systems of the bridge consist of several cross-frames spaced along its three spans. During deck construction, the rails for the finishing machine were placed on the edge of the overhangs of the bridge, which had a width of 44 in. The full depth of the girders was laterally braced using cross-frames with a maximum spacing of 21′-9″, leading to a B/D ratio of 3.95. Therefore, based on Phase I of this research and IDOT recommendations, rotation-prevention systems were not required for this bridge. The bridge was monitored in this research project to verify the B/D ratio concluded in Phase I of this research. The pouring of the concrete deck was divided into two stages. In the first stage, the pouring of the concrete deck started at both abutments and ended at splice 1 and splice 4, in the side spans; and the center portion of the mid-span was poured with concrete (shown in Figure 23). The remaining sections of the concrete deck were poured in the second stage of the deck construction at a later date.

3.3.1.1 Instrumentation Plan

The instrumentation plan and locations of the sensors for Lawrence Bridge are shown in Figures 23 to 25. Three sections—S1, S2, and S3—were chosen and instrumented with tilt sensors, strain gages, and prisms. An initial FE analysis was conducted to select the locations of these sections based on predicted maximum rotation during deck construction.

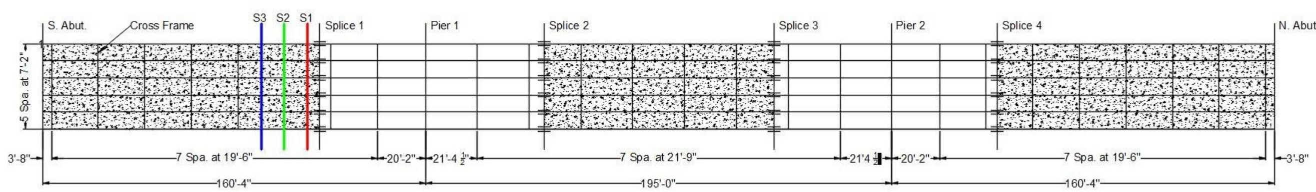


Figure 23. Instrumentation plan for Lawrence Bridge.

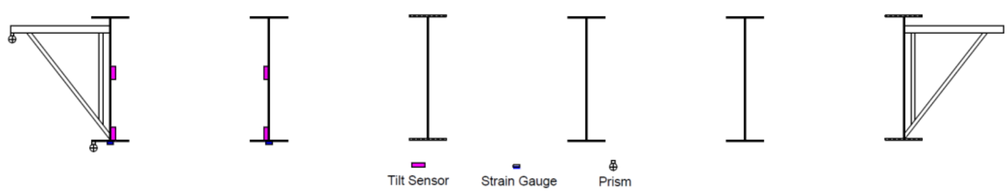


Figure 24. Locations of sensors at S1 and S3 of Lawrence Bridge.

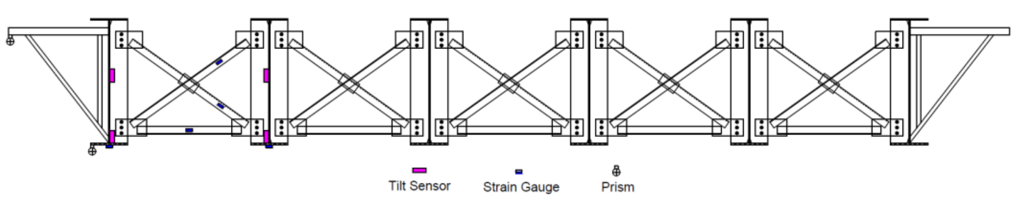
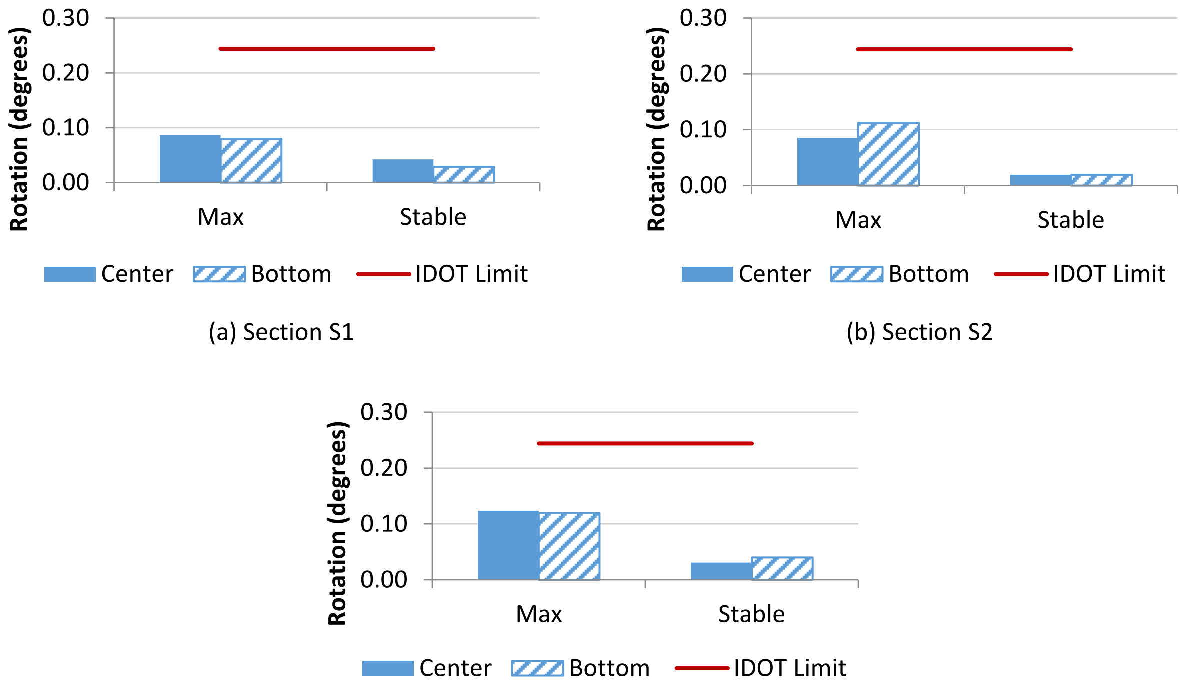


Figure 25. Locations of sensors at S2 of Lawrence Bridge.

3.3.1.2 Rotation Field Data

The maximum and stable rotation values of the exterior girder at each instrumented section of Lawrence Bridge are illustrated in Figure 26. The results indicated that the maximum rotation (0.12° at both the center and bottom of the web) of the exterior girder during deck construction occurred at S3, located approximately at the center between two cross-frames. At the center of the web, sections S1 and S2 had the same rotation value of 0.09°. However, the maximum rotation at the bottom of the web (0.08° at S1 and 0.12° at S2) had a difference of 20%, compared with the values at the center of the web, due to distortion of the girder. The stable rotation ranged from 0.02° to 0.04°, and the values at the center of the web were close to the data recorded at the bottom of each section.



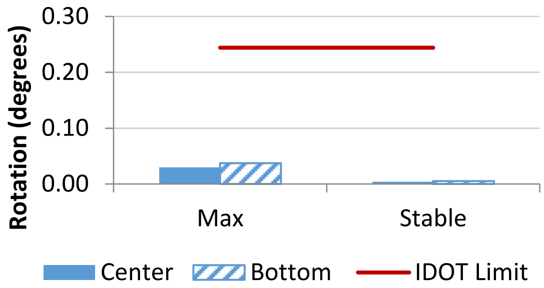
(a) Section S1

(b) Section S2

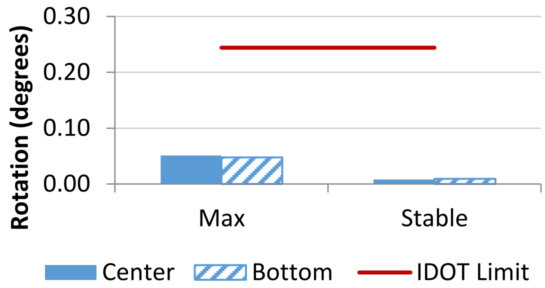
(c) Section S3

Figure 26. Maximum and stable rotations of exterior girder for Lawrence Bridge.

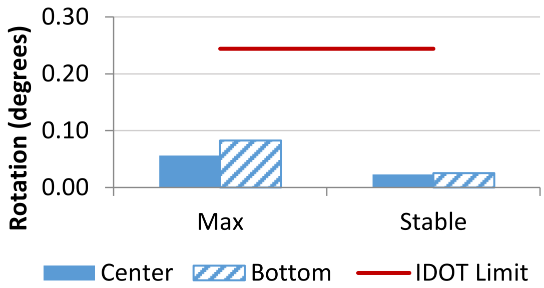
As shown in Figure 27, the interior girder had significantly lower rotation values, compared to the exterior girder. The maximum rotation value for the interior girder was measured as 0.08° at S3. However, the stable rotation at the interior girder showed very small values, ranging between 0.01° and 0.02°.



(a) Section S1



(b) Section S2



(c) Section S3

Figure 27. Maximum and stable rotations of interior girder for Lawrence Bridge.

Although temporary bracing systems were not used during deck construction of Lawrence Bridge, the maximum rotation (0.12°) was 50% below the IDOT limit (0.24°). In this bridge, cross-frames were fully braced along the depth of the girders; and the maximum spacing of these cross-frames (B) was greater than four times the girder's depth (D). As such, this bridge validates use of that criterion, recommended in Phase I of this project, in limiting the rotation of the exterior girders.

3.3.1.3 Strain Field Data

Because Lawrence Bridge did not utilize any temporary bracing system, strain gages were used to measure the strains in the cross-frame, as well as the bottom flanges of the exterior and interior girders at S2. The maximum and stable strain values recorded in the diagonal and horizontal members of the cross-frame are illustrated in Figure 28. The results showed that the horizontal cross-frame member was in compression, while one of the diagonal chords was in tension and the other had almost zero strain. This pattern aligns with the behavior of X-shaped cross-frames with a bottom chord when the girder is subjected to torsional moments. The stable strain followed the same trend but with values lower than the maximum by 50%.

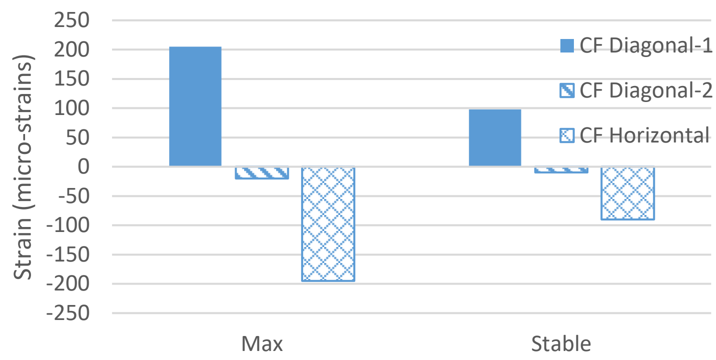


Figure 28. Field measurement of strain values in the cross-frame for Lawrence Bridge.

The maximum and stable strain values in the bottom flange of the girders are shown in Figure 29. The results indicated that after the finishing machine was removed from the bridge, the strain value dropped by 83% in the exterior girder and 68% in the interior girder. When the finishing machine was moving along the bridge, the exterior girder experienced a larger strain value, 148 micro-strains, compared to only 62 micro-strains in the interior girder. The difference in strain values is due to the effect of the finishing machine, which transferred the weight directly onto the exterior girder during deck construction, resulting in a larger load on the exterior girder. However, the stable strain showed similar values for both girders (25 and 20 micro-strains in the exterior and interior girders, respectively) because only the weight of the fresh concrete was transferred to the girder systems.

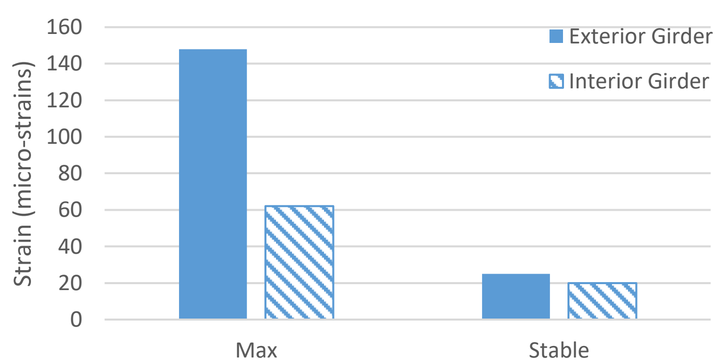
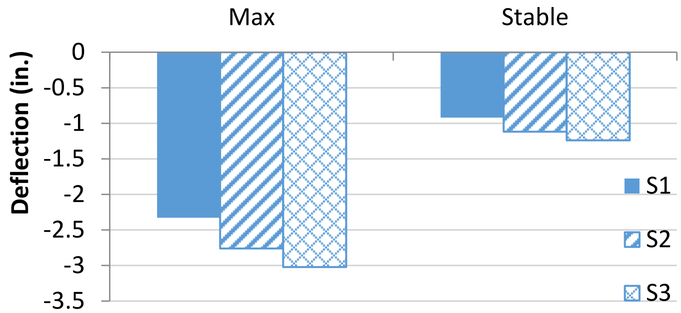


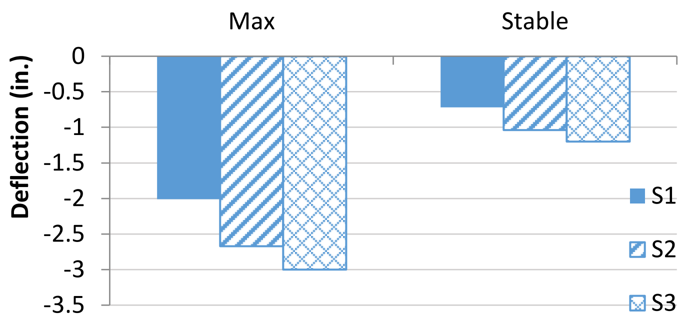
Figure 29. Field measurement of strain on the bottom flange of girders for Lawrence Bridge.

3.3.1.4 Deflection Field Data

Vertical deflections at sections S1, S2, and S3 were measured at the exterior girder and the steel brackets during deck construction. The maximum vertical-deflection values of the steel brackets were 2.33, 2.76, and 3.02 in. at S1, S2, and S3, respectively, as shown in Figure 30. After the finishing machine was removed from the bridge, the stable vertical deflection decreased by 60%, compared to the maximum values, resulting in 0.92, 1.12, and 1.24 in. at S1, S2, and S3, respectively. Vertical deflection of the exterior girder experienced values very similar to those of the steel brackets, with only a 13%, 3%, and 1% decrease in values at S1, S2, and S3, respectively.



(a) Field measurement of deflections of steel brackets



(b) Field measurement of deflections of exterior girder

Figure 30. Field measurement of deflections for Lawrence Bridge.

3.3.2 Ford Bridge

Ford Bridge utilized six W27 girders over three spans and spaced at 80 in. The bridge has a 14° skew and an overhang width of 35 in. Channel sections (C12x25) with a maximum spacing of 17′-7½″ were employed as the permanent lateral bracing system (diaphragms), as shown in Figure 31. Because IDOT requires finishing machine rails to be placed over the centerline of the exterior girders for W30 sections and below in order to limit exterior-girder rotation, the rails of the finishing machine were installed right above the exterior girder. The rotation-prevention system (transverse tie–diagonal pipe system) maintained a consistent spacing of 8 ft along the bridge. The pouring of the concrete deck started from the south abutment and ended at the north abutment.

3.3.2.1 Instrumentation Plan

An initial FE analysis of Ford Bridge indicated that the maximum rotation occurs approximately at the selected sections, shown in Figure 31. Three sections—S1, S2, and S3—were chosen to examine the rotation of both exterior girders in the bridge. The locations of all sensors instrumented at each section are illustrated in Figures 32 to 34.

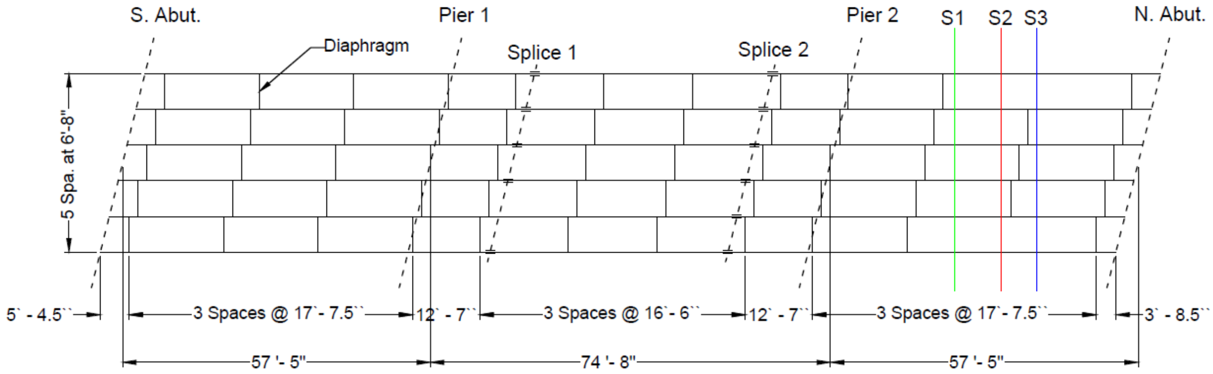


Figure 31. Instrumentation plan for Ford Bridge.

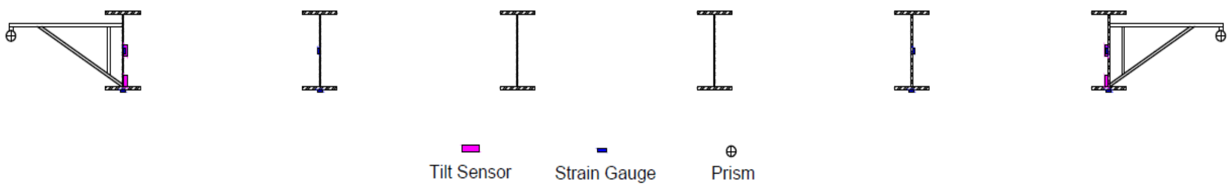


Figure 32. Locations of sensors at S1 of Ford Bridge.

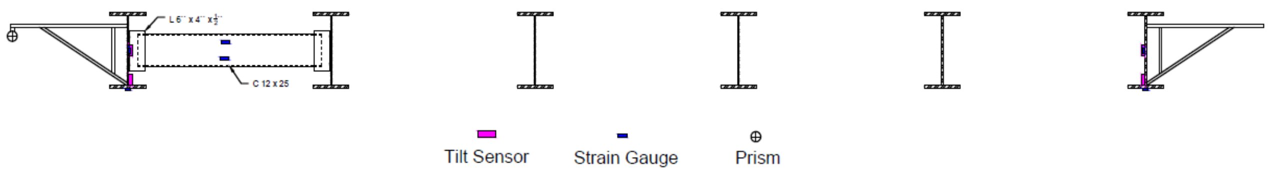


Figure 33. Locations of sensors at S2 of Ford Bridge.

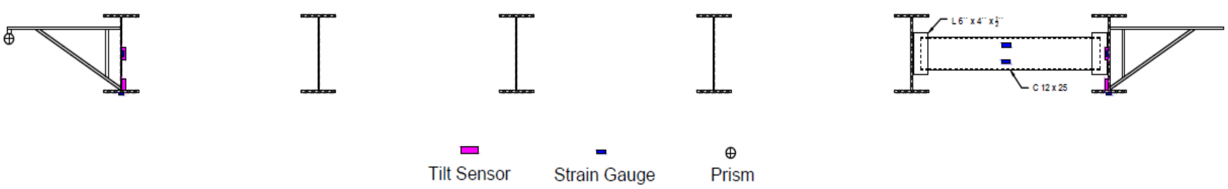
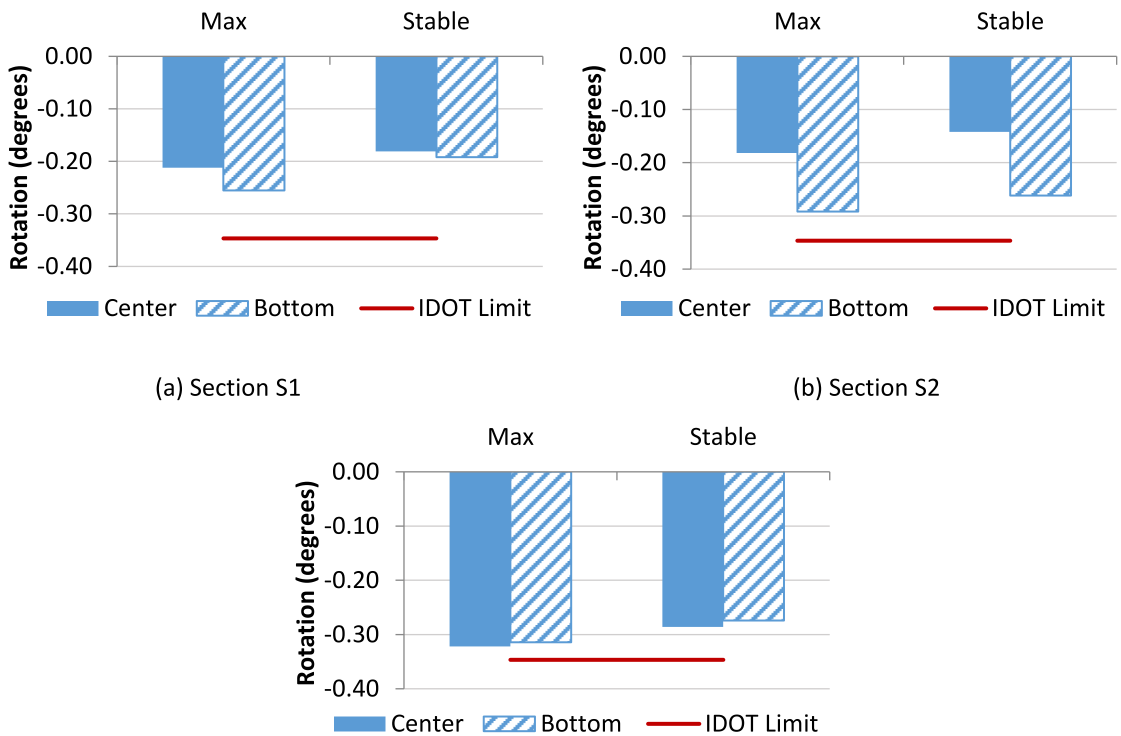


Figure 34. Locations of sensors at S3 of Ford Bridge.

3.3.2.2 Rotation Field Data

The maximum and stable rotations of the exterior girder on the acute-angle side (G1) are shown in Figure 35. Negative rotation values, indicating an inward rotation, were measured for all three sections during deck construction. Section S3 experienced the greatest inward rotation, 0.32° at the center of the web and 0.31° at the bottom of web, because this section was located between diaphragms with the largest spacing. At the bottom of web, sections S1 and S2 had similar rotation values, compared to those at the center of web. However, the bottom of the web suffered larger rotation values, 0.26° and 0.29° for S1 and S2, respectively, due to distortion of the girder’s web. The stable rotation values ranged from 0.18° to 0.28°, with an average 16% decrease in rotation, compared to maximum rotation data.



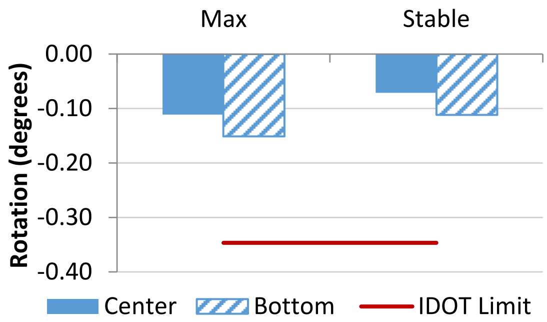
(a) Section S1

(b) Section S2

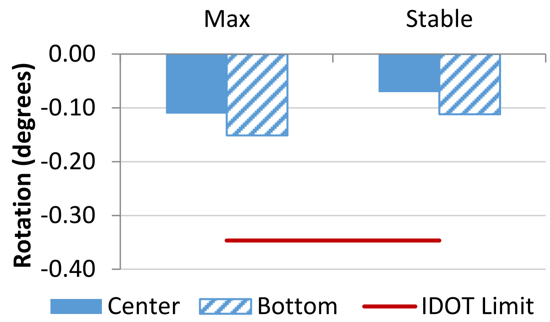
(c) Section 3

Figure 35. Maximum and stable rotations of exterior girder on G1 for Ford Bridge.

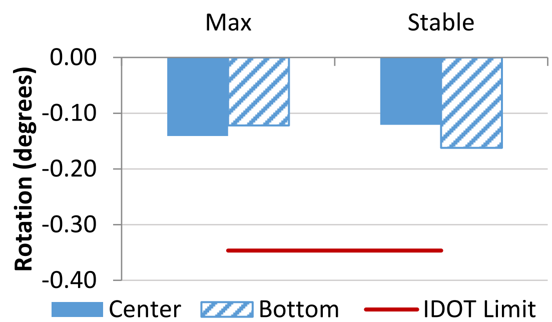
The exterior girder on the obtuse-angle side (G6) also experienced inward rotation, but it was relatively smaller than those recorded in G1, as shown in Figure 36. The maximum rotation value in G6 occurred at S2, with a value of 0.19°. The rotation values decreased by approximately 48%, 29%, and 53% for S1, S2, and S3, respectively, in contrast to the rotation values in G1. Therefore, the skew of the bridge has significant influence: it can redistribute the rotation on the exterior girder and force the exterior girder on the acute side of the bridge to experience a larger rotation.



(a) Section S1



(b) Section S2



(c) Section S3

Figure 36. Maximum and stable rotations of the exterior girder on G6 for Ford Bridge.

The rotation limit for Ford Bridge is 0.35°, based on IDOT requirements for deck construction. The results showed that the maximum rotation was measured as 0.32° in the inward direction, about 8% below the limit. A possible reason for the inward rotation is that the fresh concrete on the interior formwork resulted in torsional moments in the exterior girder, contrary to those caused by the overhang concrete. In this case, because the weight of the finishing machine was not applied on the cantilever, the torsional moments caused by the loads on the internal formwork can be greater than those caused by the forces on the overhang formwork. Additionally, an alignment error could have caused the finishing machine rails to shift during installation, causing unexpected torsional moments to act on the exterior girder. Therefore, caution is required when installing finishing machine rails above the centerline of the exterior girder.

3.3.2.3 Strain Field Data

The strain gages were installed on the transverse ties and diagonal pipes of the temporary rotation-bracing system in both exterior panels of the bridge. Unlike that of other monitored bridges, the strain in Ford Bridge showed very low values, with a maximum of 5 micro-strains, as shown in Figure 37. These low strain values indicated that the temporary rotation-prevention system did not perform adequately since the diagonal pipe–horizontal tie system was not designed to resist inward rotation of exterior beams, and the torsional moments created by the construction loads were completely resisted by the girder system.

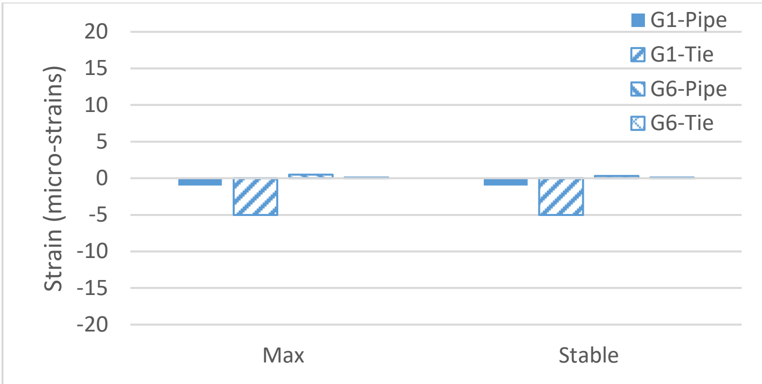


Figure 37. Field measurement of strain in ties and pipes for Ford Bridge.

3.3.2.4 Deflection Field Data

Vertical deflections at sections S1, S2, and S3 were measured for the exterior girder (G6) during deck construction. Prisms were not installed on the steel brackets due to inaccessibility of the overhang formwork. The maximum vertical deflection of the exterior girder was 0.62 in. at S1 (shown in Figure 38). For all three sections, the stable vertical deflections were recorded as less than 0.1 in.

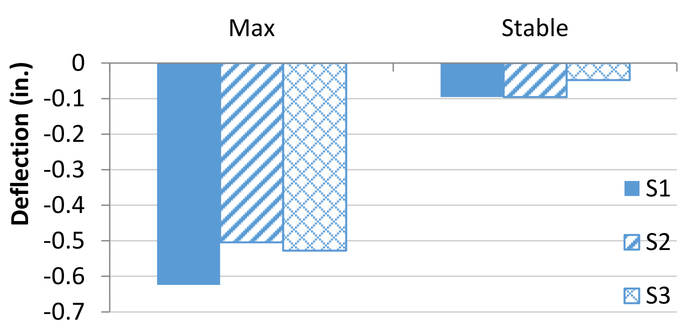


Figure 38. Field measurement of deflection of exterior girder for Ford Bridge.

3.3.3 Madison Bridge

The Madison Bridge is a continuous bridge with three spans, located on Route 159 in the state of Illinois. The bridge consists of six 40-in. web plate girders spaced at 87 in. The bridge has three spans: 75′-4″, 96′-0″, and 75′-4″. The lateral bracing system consists of C15x40 diaphragms connected to the girders using steel L-angles and spaced along the span of the bridge. Maximum diaphragm spacing is 23′-8″. Overhang width is 41.5 in. A temporary rotation-prevention system in the form of diagonal pipes and transverse ties was placed along the three spans of the bridge, with 8-foot spacing, based on IDOT recommendations. The concrete deck slab was poured, starting from the west abutment and ending at the east abutment, covering the entire bridge.

3.3.3.1 Instrumentation Plan

Similar to the procedures with the other instrumented bridges, a preliminary FE analysis of the bridge was conducted to predict the location of the maximum rotation in the exterior girder. Thus, three sections—S1, S2, and S3, as shown in Figure 39—were chosen to be instrumented and investigated. Tilt sensors, strain gages, and prisms were installed at each of the three sections; the locations are shown in Figures 40 to 42.

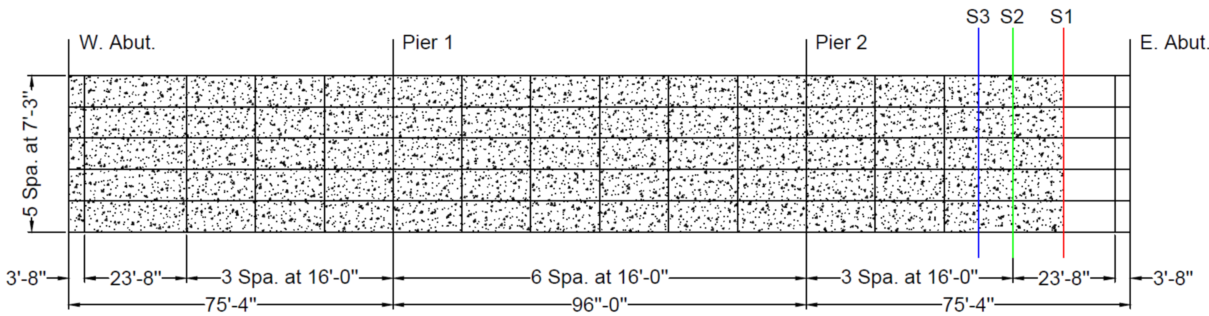


Figure 39. Instrumentation plan for Madison Bridge.

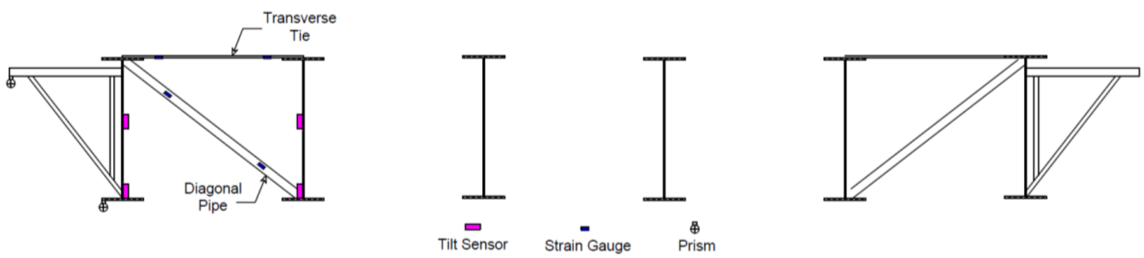


Figure 40. Locations of sensors at S1 of Madison Bridge.

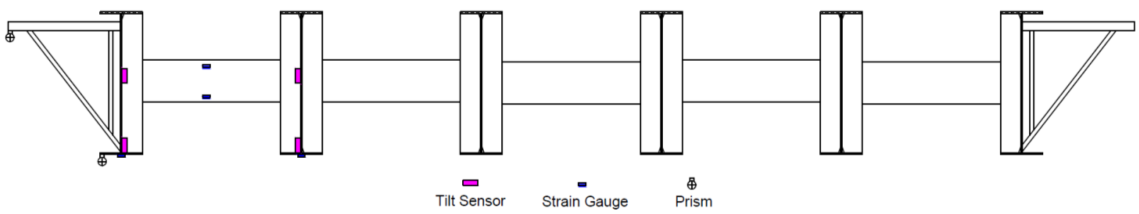


Figure 41. Locations of sensors at S2 of Madison Bridge.

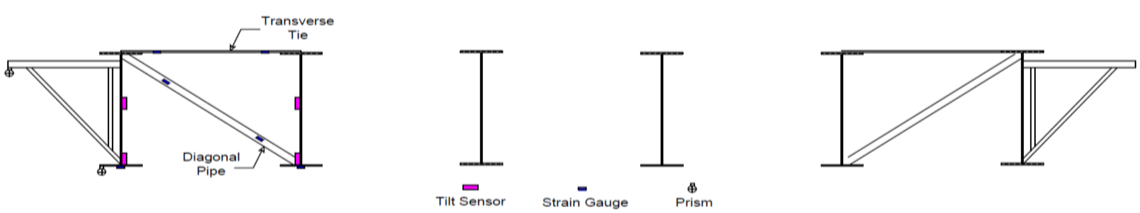
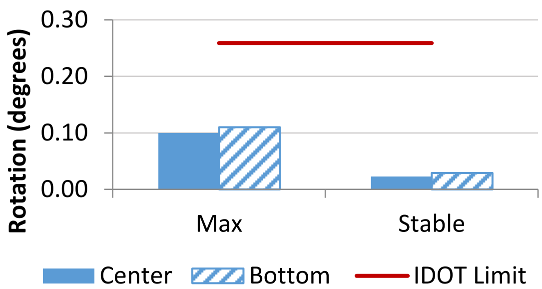


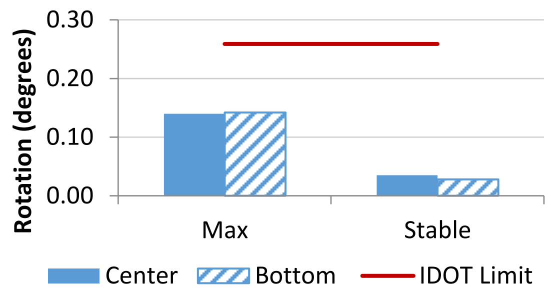
Figure 42. Locations of sensors at S3 of Madison Bridge.

3.3.3.2 Rotation Field Data

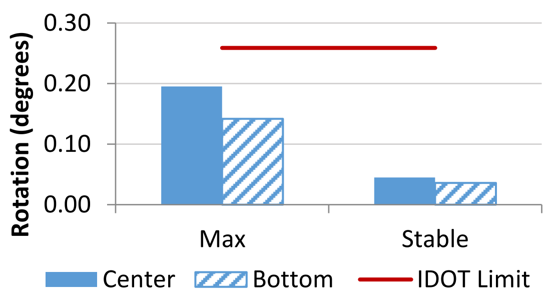
The maximum and stable rotation values of the exterior girders at the three sections—S1, S2, and S3—are shown in Figure 43. The rotation at the center of the web showed a maximum value of 0.1°, 0.14°, and 0.20° at sections S1, S2, and S3, respectively. Rotation at S1 showed the lowest value because a temporary rotation-prevention system was placed at that section. After the finishing machine was removed from the bridge, rotation values showed a significant decrease. The stable rotation values dropped to approximately 25% of the maximum values. The center and bottom of the girder's web did not show much difference because the diaphragms were fully braced to the girder using connecting plates, which had the same depth as the web and were welded to both the web and flanges of the girders.



(a) Section S1



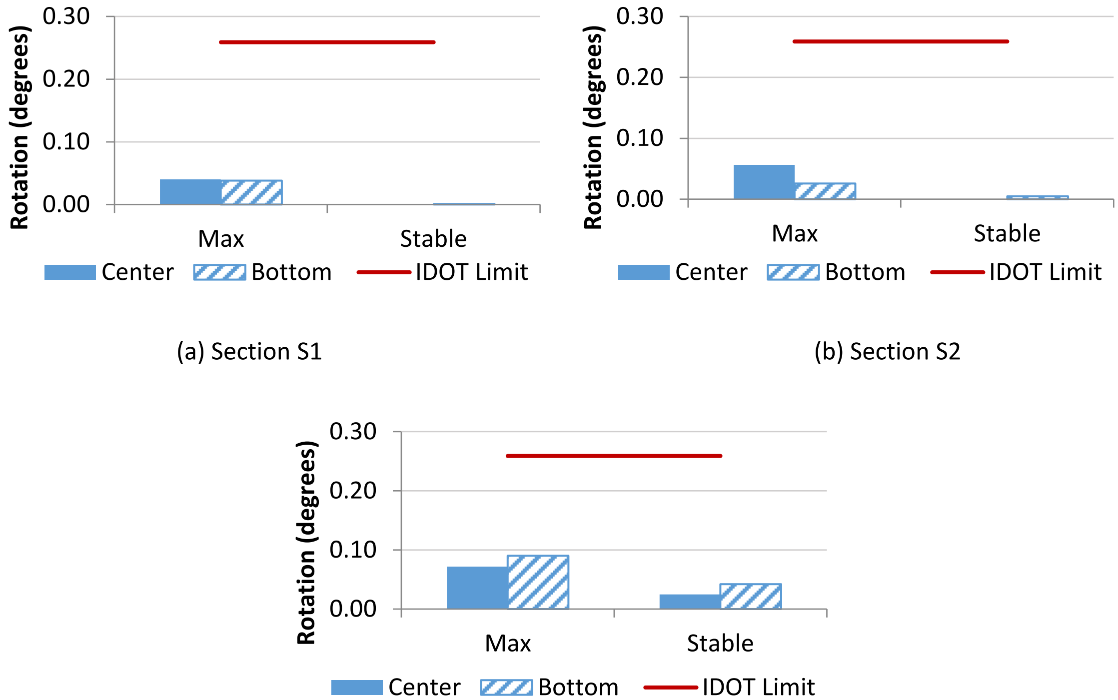
(b) Section S2



(c) Section S3

Figure 43. Maximum and stable rotations of exterior girder for Madison Bridge.

The rotation values of the first interior girder were significantly lower than the corresponding values recorded in the exterior girder. As shown in Figure 44, maximum rotation, 0.09°, was observed at S3. The stable rotation values at the first interior girder were close to zero. The low rotation in the first interior girder was expected, as the effect of the torsional moment faded toward the centerline of the bridge. This behavior can be attributed to the diaphragms' distributing the moments among all the girders.



(a) Section S1

(b) Section S2

(c) Section S3

Figure 44. Maximum and stable rotations of exterior girder for Madison Bridge.

The rotation limit for Madison Bridge is approximately 0.26°. The results showed that the maximum rotation was 25% lower than the IDOT limit when the temporary rotation-prevention system was utilized and spaced at 8 ft along the bridge. This finding also proves the effectiveness of the transverse tie–diagonal pipe system for limiting exterior-girder rotation.

3.3.3.3 Strain Field Data

Strain gages were installed on the transverse ties and diagonal pipes at section S1 to evaluate the effectiveness of the rotation-prevention system during deck construction. Figure 45 shows the maximum and stable strain data in the rotation-prevention system. As expected, the transverse tie experienced tensile stress while the diagonal pipe was under compression. The average maximum strain at the transverse tie and the diagonal pipe was 295 micro-strains (tension) and 88 micro-strains (compression), respectively. The average stable strain was very low, compared with the maximum, 35 micro-strains in the transverse tie and 16 micro-strains in the diagonal pipe. These stable strain values were significantly lower, as the finishing machine was no longer on the bridge, which meant the concentrated torsion moment created by the finishing machine was no longer affecting rotation of the exterior girder.

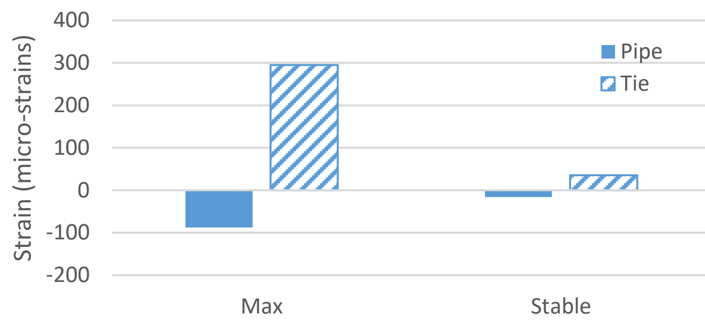
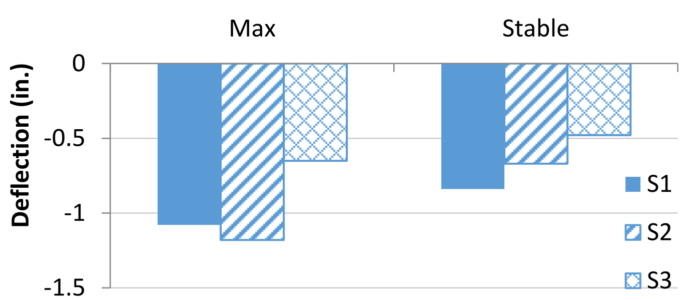


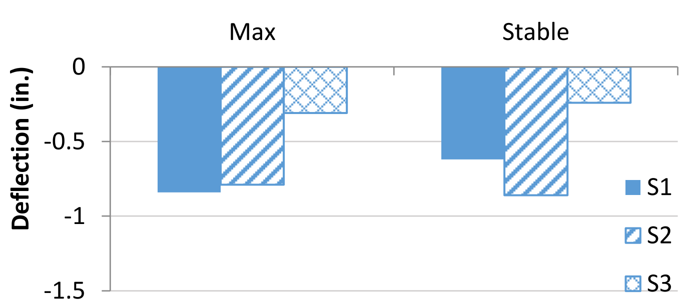
Figure 45. Field measurement of strain on transverse ties and diagonal pipes for Madison Bridge.

3.3.3.4 Deflection Field Data

Vertical deflection of the exterior girder was recorded at the three sections—S1, S2, and S3. The maximum and the stable vertical deflections at the bracket and the exterior girder at each section are shown in Figure 46. The maximum deflections of the steel bracket were 1.08, 1.18, and 0.65 in. at sections S1, S2, and S3, respectively. The corresponding stable deflections of the steel bracket were 0.84, 0.67, and 0.48 in. The exterior girder showed a smaller deflection, compared with the steel bracket, because the rotation of the exterior girder led to extra deflection at the edge of the overhang.



(a) Field measurement of deflection in steel bracket



(b) Field measurement of deflection in exterior girder

Figure 46. Field measurement of deflection for Madison Bridge.

3.3.4 McDonough Bridge

The McDonough Bridge is a three-span bridge consisting of six W36 girders spaced at 6 ft. The bridge has a 15° skew and three spans of 63′-3.25″, 105′-0″, and 63′-3.25″. The lateral bracing system consists of C15x40 diaphragms connected to the girders using L6″ x 4″ x 1/2″ angle steel along the span of the bridge. Maximum spacing of the diaphragms is 25 ft. Overhang width is 31 in. During the concrete pouring of the deck slab, the finishing machine moved along the span of the bridge on rails located at the edge of the overhangs. A temporary rotation-prevention system, in the form of diagonal pipes and transverse ties was placed along the length of the bridge, spaced at 8 ft, based on IDOT recommendations. The pouring of the concrete deck slab started from the east abutment and ended at the west abutment.

3.3.4.1 Instrumentation Plan

Three sections—S1, S2, and S3—were chosen to be instrumented with multiple sensors to measure rotation, strain, and deflection, as shown in Figure 47. Selection of the sections was based on the preliminary FE analysis of the bridge. Figures 48 and 49 show locations of the sensors in each section.

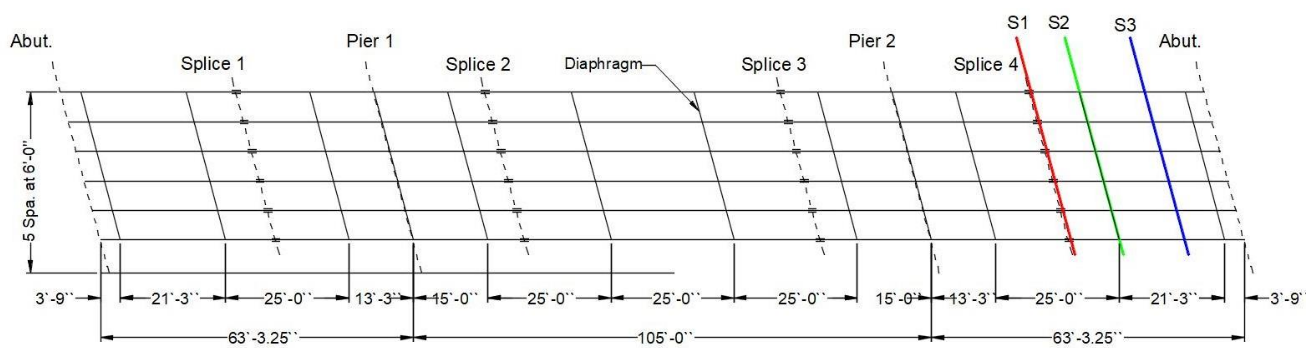


Figure 47. Instrumentation plan for McDonough Bridge.

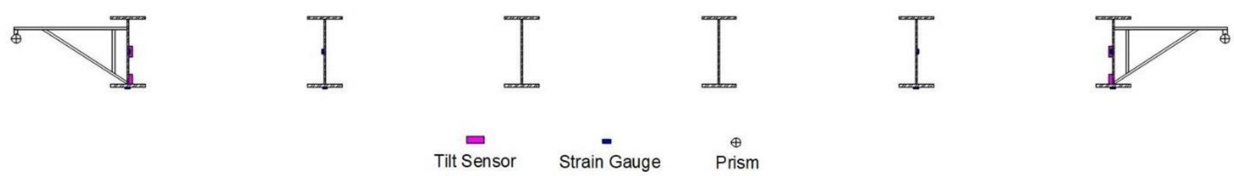


Figure 48. Locations of sensors at S1 and S3 for McDonough Bridge.

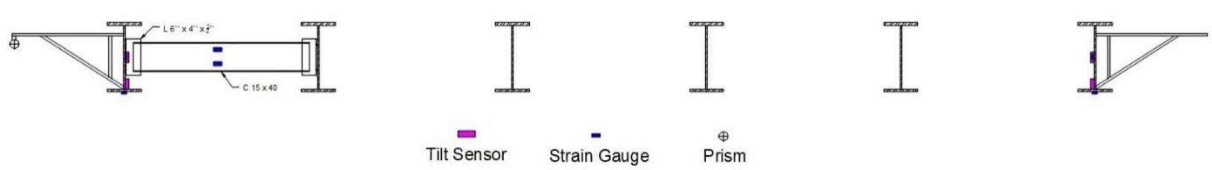


Figure 49. Locations of sensors at S2 for McDonough Bridge.

3.3.4.2 Rotation Field Data

The maximum and stable rotations of the exterior girder on the acute-angle side (G1) are shown in Figure 50. Because the sensors were installed a couple of days before the pouring of the concrete deck, some of the tilt sensors were damaged by harsh weather conditions before deck construction. Maximum rotation, 0.33°, occurred at S1. At section S2, maximum rotation occurred when the finishing machine was loaded at that section, with a rotation value of 0.19°. The rotation dramatically decreased when the finishing machine was removed from the bridge, resulting in a 26% and 42% decrease in the maximum rotation values for S1 and S2.

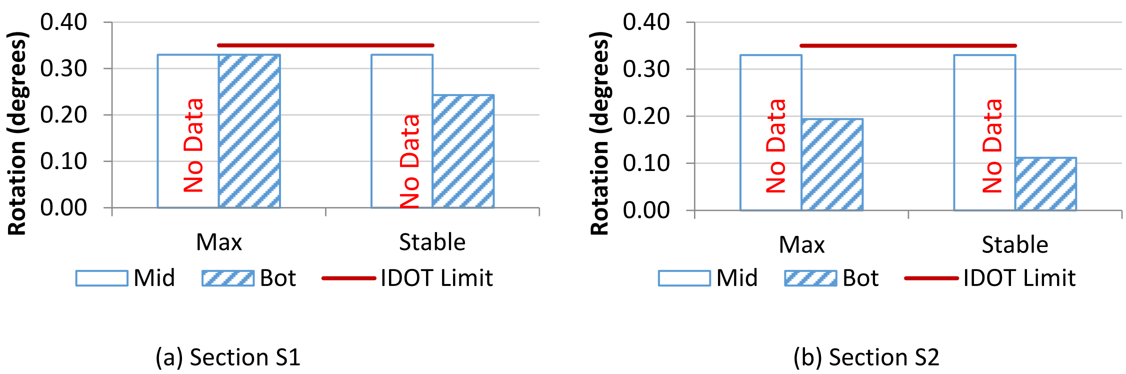
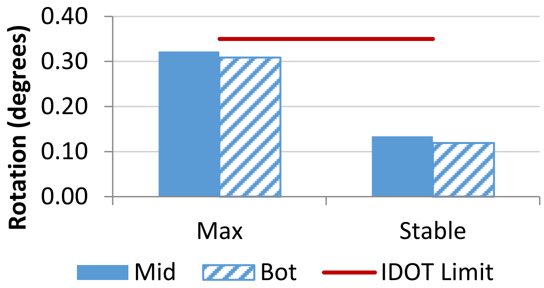
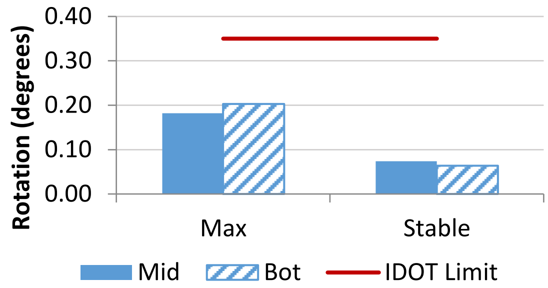


Figure 50. Maximum and stable rotations of exterior girder on G1 for McDonough Bridge.

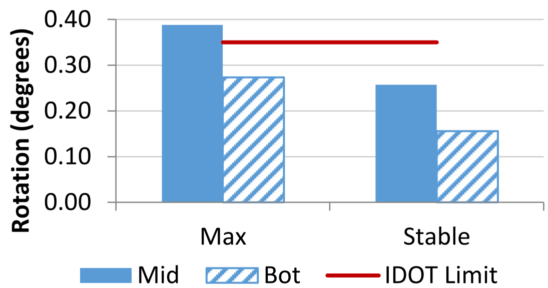
Rotation data of the exterior girder on the obtuse-angle side (G6) are shown in Figure 51. The maximum rotation value, 0.39°, of the bridge occurred at S3. For S1 and S2, 0.32° and 0.20° were observed as the maximum rotations. The stable rotation was significantly lower than the maximum values; a more than 50% decrease in the maximum rotation values was measured. Rotation at the center and bottom of the web showed a difference of more than 30% in section S3, indicating web distortion due to the deck construction. The distortion at this section was mainly due to the use of the partial-depth connecting plate, which braced only part of the web along the depth of the girder section.



(a) Section S1



(b) Section S2



(c) Section S3

Figure 51. Maximum and stable rotations of exterior girder on G6 for McDonough Bridge.

Figure 52. Protective rubber at the ends of the pipes used in McDonough Bridge.

Based on IDOT requirements for deck construction, the rotation limit for McDonough Bridge is 0.35°. The field data indicated that maximum rotation was 0.39°, about 10% above the limit value. A possible reason for these high rotation values is the use of protective rubber at the ends of the diagonal pipes (shown in Figure 52), leading to reduced effectiveness of the rotation-prevention systems.

3.3.4.3 Strain Field Data

Strain gages were installed on the transverse ties and diagonal pipes of the temporary rotation-prevention systems in both exterior panels of the bridge. The temporary rotation-prevention systems on both sides of the bridge behaved similarly, with almost the exact strain values in ties and pipes, as shown in Figure 53. When the finishing machine was on top of the section, maximum strain occurred, showing values of 80 micro-strains in both pipes and 260 micro-strains in both ties. After the pouring of the concrete deck was finished and the finishing machine passed over the abutment, very small strain values were observed (25 micro-strains for pipes and 8 micro-strains for ties).

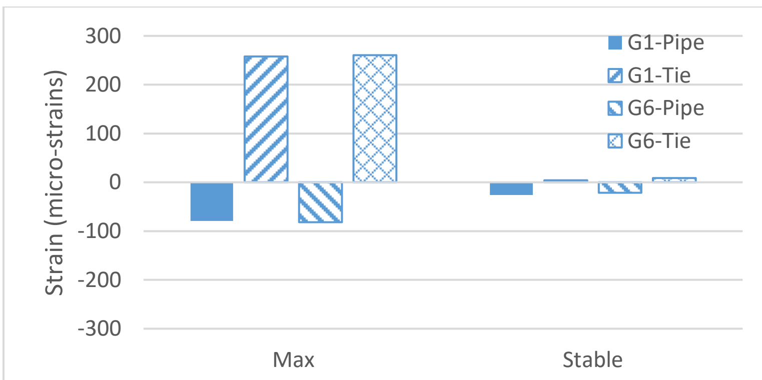
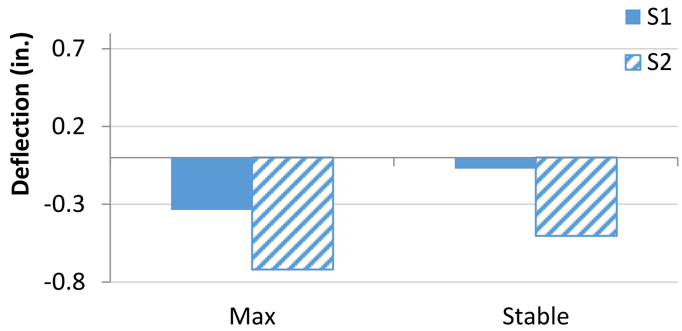


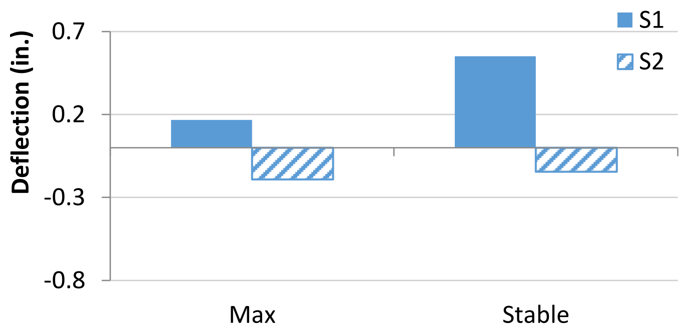
Figure 53. Field measurement of strain in ties and pipes for McDonough Bridge.

3.3.2.4 Deflection Field Data

Prisms were installed at sections S1 and S2 on one of the exterior girders (G6) to measure vertical deflection during deck construction. The field measurement of the deflections is shown in Figure 54. The exterior girder had maximum vertical-deflection values of 0.33 in. in the downward direction and 0.55 in. in the upward direction. Section S2 experienced upward deflection because the length of the middle span was 1.67 times larger than the side span, which resulted in a large upward deflection in the side span when the middle span was fully loaded with the fresh concrete weight. Vertical deflection of the steel bracket showed a significant difference compared to the girder's deflection. This is because the vertical deflection of the steel bracket was created not only by the construction loads on the bridge but also the rotation of the exterior girder. Outward rotation of the exterior girder resulted in a large downward deflection at the end of the cantilever overhang bracket, leading to different vertical deflections between the girder and the steel bracket. Maximum vertical deflection of the steel bracket was 0.72 inches, which occurred at S1. The stable vertical deflection followed the same trend, with a maximum downward deflection of 0.50 in. and an upward deflection of 0.55 in.



(a) Field measurement of deflection in the steel bracket



(b) Field measurement of deflection in the exterior girder

Figure 54. Field measurements of deflection for McDonough Bridge.

3.4 FINITE-ELEMENT ANALYSIS

3.4.1 Finite-Element Modeling

SAP2000, an FE analysis software package, was used to simulate numerically the four instrumented bridges. The FE models were then validated using the field data. Also, by using the same modeling technique, the research was extended to evaluate performance of the temporary bracing systems in different circumstances. All girders and diaphragm sections were modeled in SAP2000 using thick-shell elements. Components of the temporary rotation-prevention system (diagonal pipe and horizontal tie) were represented using frame elements. To simulate behavior of the temporary rotation-prevention systems, the diagonal pipes were modeled as compression-only members and the horizontal ties as tension-only members. The steel bracket and finishing machine rails were modeled using frame elements. Shell elements were also implemented to represent the connecting plates between the girders and diaphragms/cross-frames. Rigid links were utilized to simulate bolted and welded connections. The transverse tie is usually subjected to vertical sagging; this is typically caused by the weight of the tie itself, interference with the deck-slab reinforcement, and inadequate tightening of the tie. Sagging of the tie may lead to a less effective temporary rotation-prevention system and cause excessive exterior-girder rotation. Therefore, nonlinear links with gaps were used to connect the girders to the tie to mimic a similar behavior of a real construction site. Figure 55 shows a full-scale FE model for Madison Bridge. The rest of the bridges were implemented with the same modeling technique to ensure consistency in the simulation.

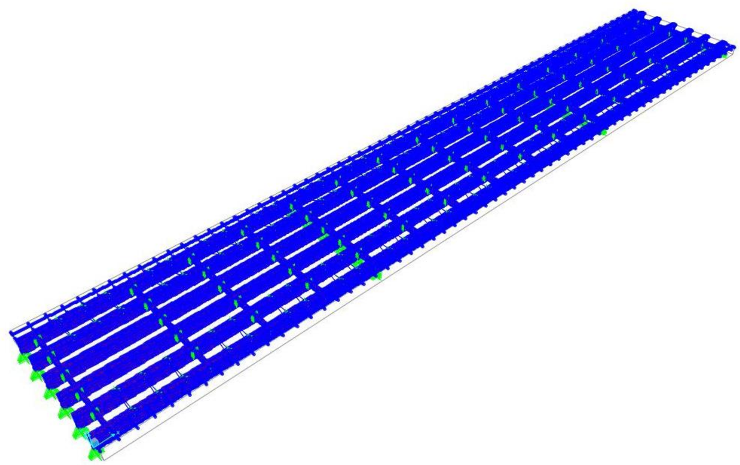


Figure 55. FE model of Madison Bridge.

3.4.2 Finite-Element Models' Loading Concept

The deck-slab formwork is supported by hangers connected to the girders of the bridge. The fresh concrete weight was distributed among the girders using the concept of the tributary area. Therefore, concentrated loads were used at the edges of the flanges of the girders to distribute the weight of the concrete between the girders. The fresh concrete weight on the overhang formwork was simulated as concentrated loads on the steel brackets. Concentrated loads were also used to represent the wheels of the finishing machine. The finishing machine wheel loads vary from bridge to bridge due to different deck-slab widths and types of finishing machines used. Figure 56 illustrates the loading concept used in all FE models developed in this project. Other loads during construction were neglected due to their limited impact on the rotation of the exterior girders.

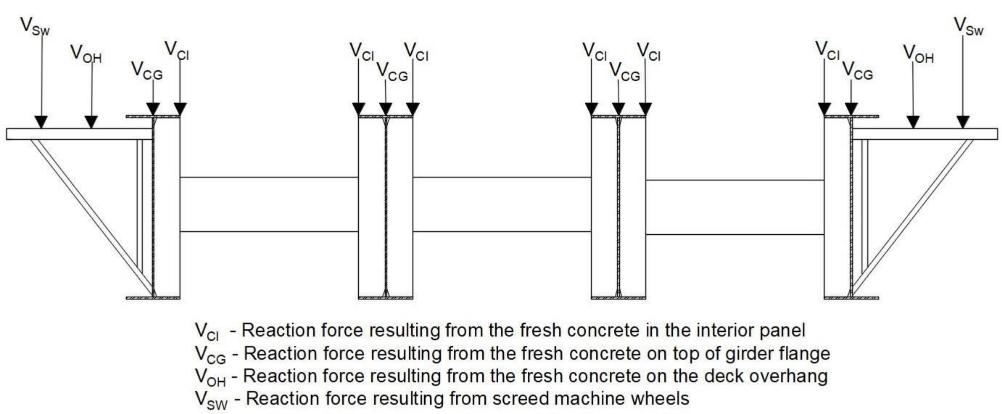


Figure 56. Loading concept for FE Analysis.

3.4.3 Materials' Properties

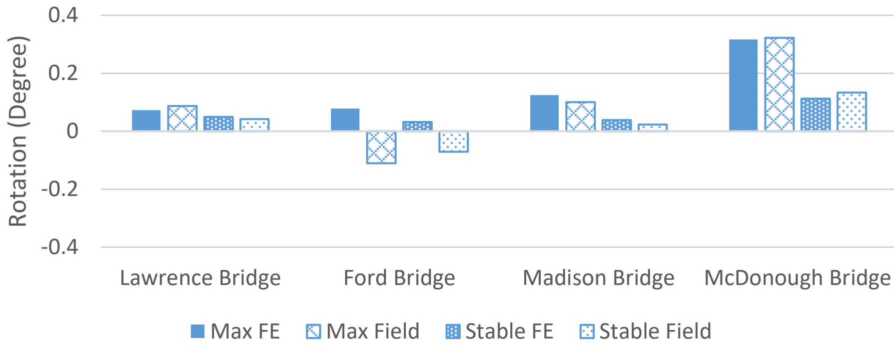
In all FE models, the modulus of elasticity (E) and Poisson's (υ) ratio for steel were taken as 29,000 ksi and 0.3, respectively. The fresh concrete was represented using the tributary-area concept that considered the unit weight of reinforced concrete equal to 150 lb/ft^3.

3.4.4 Comparison of the Rotation Field Data and FE Results

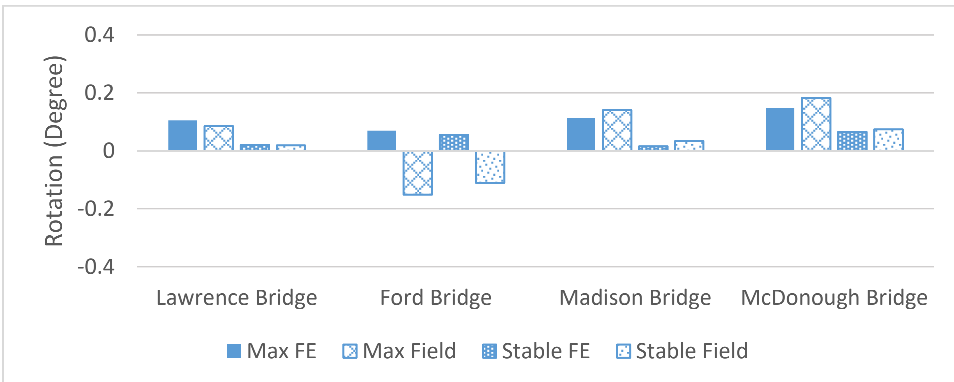
A detailed comparison of the rotation field data and FE results at the center of the web for sections S1, S2, and S3 for all monitored bridges is shown in Figure 57. At section S1, the FE results of Lawrence, Madison, and McDonough Bridges were remarkably similar to the field data. The difference in the maximum rotation between the field and the FE results was 16%, 20%, and 2% in Lawrence, Madison, and McDonough Bridges, respectively. Similarly, as shown in Figure 57a, the FE and field stable rotation values were very close to each other. In all the monitored bridges, section S2 was at the location of a permanent lateral bracing system (cross-frame or diaphragm). The FE results in Lawrence, Madison, and McDonough Bridges were similar to the field data shown in Figure 57b. It should also be noted that the rotation values at section S2, in both the field and FE, were lower than those recorded at S1 because the permanent diaphragms there resisted rotation of the exterior girders. Section S3 showed very similar behavior to section S1 in all bridges. The FE and field rotation values in all bridges were also comparable, with very small differences, except in Ford Bridge, as shown in Figure 57c.

The FE results of Ford Bridge, however, were not analogous to the field data. The inward rotation recorded in the exterior girders was not predicted in the FE model. There are several possible reasons as to why this discrepancy occurred. For instance, if the finishing machine rails were not installed exactly at the centerline of the exterior girder and shifted toward the center of the bridge, an inward torsional moment may occur, causing inward rotation. Also, because the formwork hangers can transfer some moments to the edges of the girders, an inward rotation could have developed due to the unbalanced forces acting on the exterior girder.

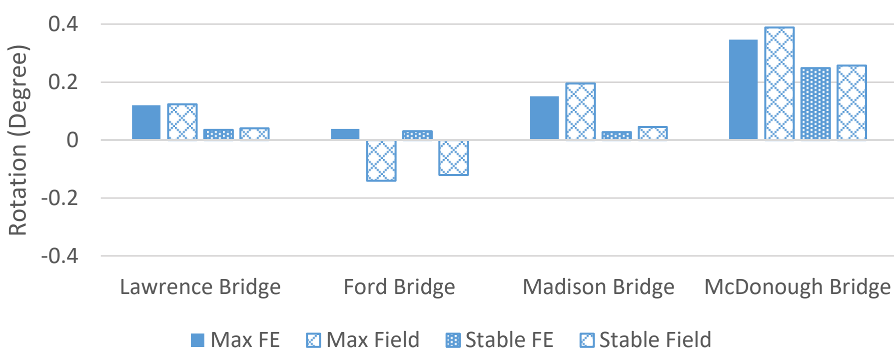
Figure 58 compares the field-recorded rotations with the FE rotations at sections S1, S2, and S3 at the bottom of the web of the exterior girder for all four bridges. The difference between the FE and field maximum rotation results at section S1 was 9%, 1%, and 8% in Lawrence, Madison, and McDonough Bridges, respectively. The corresponding differences at section S2 were 1%, 4%, and 2%. However, in general, the rotation values were lower at S2 than at other sections due to the higher torsional stiffness provided by the diaphragm at this section. The FE and field maximum rotations at section S3 were also very close to each other. Finally, in all three sections in Lawrence, Madison, and McDonough Bridges, the stable rotation values were low but still similar in the field and FE analysis.



(a) Section S1

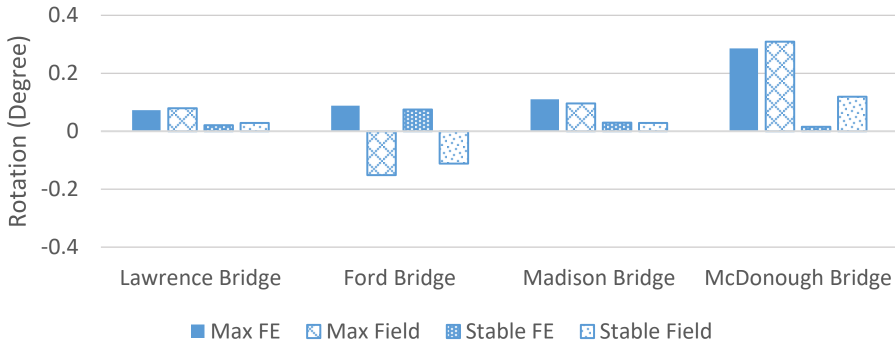


(b) Section S2

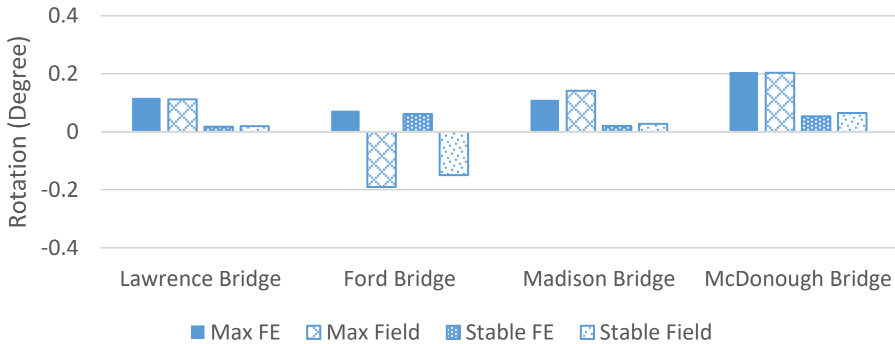


(c) Section S3

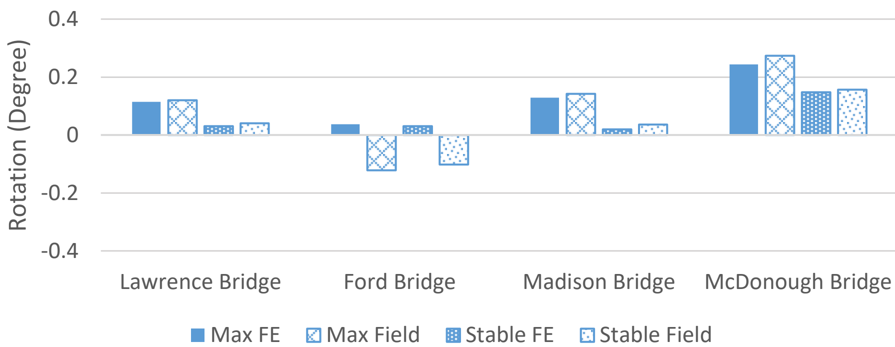
Figure 57. Rotation field data vs. FE results at the center of the web of exterior girders.



(a) Section S1



(b) Section S2



(c) Section S3

Figure 58. Rotation field data vs. FE results at the bottom of the web of exterior girders.

The results from the field monitoring and the FE analysis also indicated that rotation values at both the top and bottom of the girder webs were similar in Lawrence and Madison Bridges, with maximum differences of 11% and 14%, respectively. For Ford and McDonough Bridges, rotation values at the center of the webs were significantly larger than those at the bottom of the webs, resulting in distortion of the girder's web. The maximum changes between the center and the bottom of the webs were 36% and 30% in Ford and McDonough Bridges, respectively. Lawrence and Madison Bridges did not experience significant distortion, due to the use of full-depth connecting plates, which distributed the reaction forces along the entire depth of the web. However, because partial-depth connecting plates were used in Ford and McDonough Bridges, the reaction forces were distributed only along the depth of the connecting plates, leading to large torsional forces at the center of the girder web. Therefore, larger rotation values occurred at the center of the web; and differences were observed between the center and the bottom of the girder web.

3.4.5 Comparison of Strain Field Data and FE Results

The FE and field strain values in the transverse ties and diagonal pipes used in all four bridges are shown in Figure 59 and Figure 60. Lawrence Bridge had no strain values, as no temporary rotation-prevention system was used in this bridge. Additionally, due to inward rotation of the exterior girders in Ford Bridge, the field strain values in the transverse ties and the diagonal pipes were almost zero, bearing very little similarity to the strain values from the FE model. However, the FE maximum and stable strain values in the transverse ties utilized in Madison and McDonough Bridges were very close to the field strains. The maximum recorded difference between the FE and field strains in the transverse ties was less than 10%, while the maximum recorded difference in the strain values of the diagonal pipes was approximately 30%. However, in both Madison and McDonough Bridges, the transverse ties' strain values were positive (tension) and the diagonal pipes' strain values negative (compression). This behavior was accurately predicted in the FE models of these two bridges. Therefore, it proves that the FE modeling technique used for the temporary rotation-prevention system is valid and can accurately predict the resistance of the system to exterior-girder rotation.

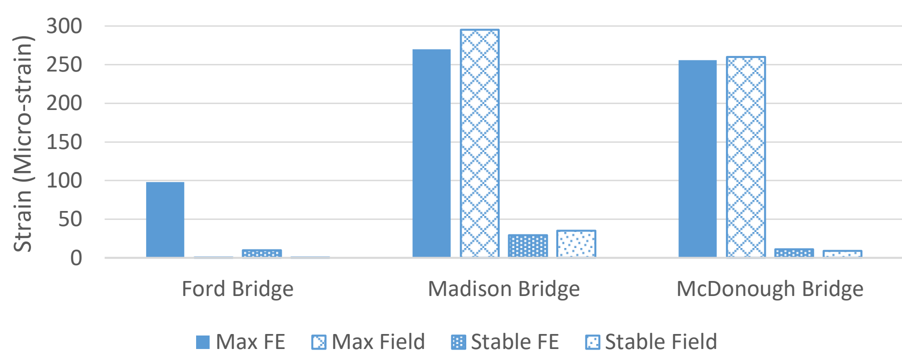


Figure 59. Strain field data vs. FE results in transverse ties.

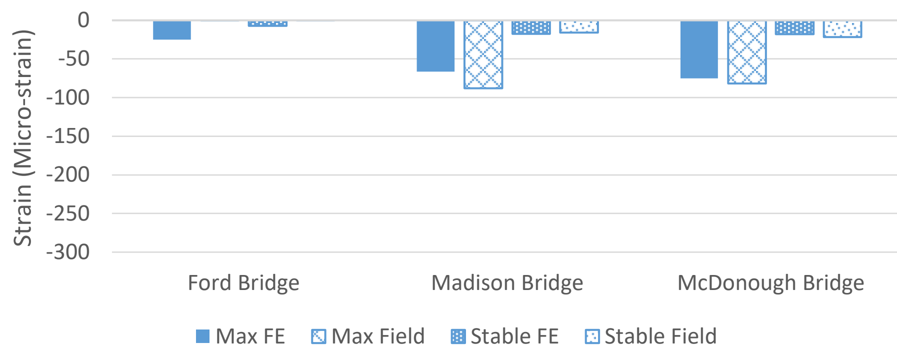
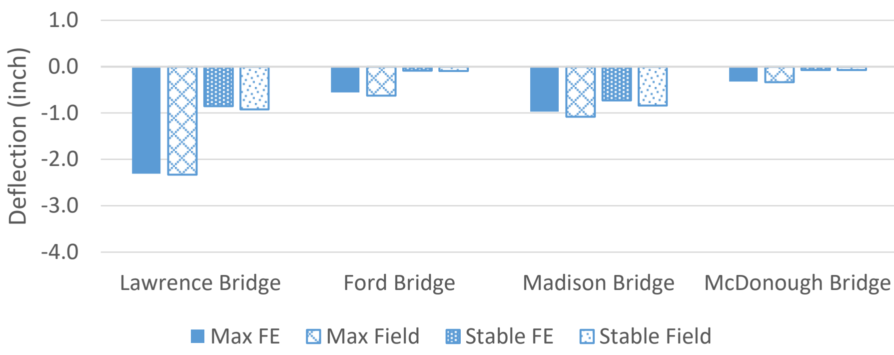


Figure 60. Strain field data vs. FE results in diagonal pipes.

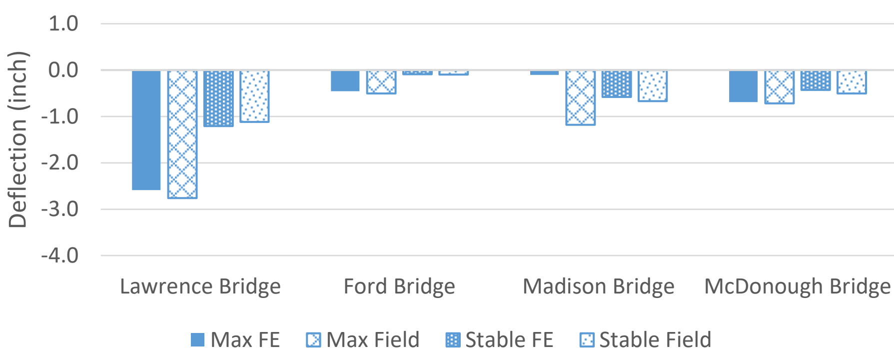
3.4.6 Comparison of Deflection Field Data and FE Results

Figure 61 compares the bracket FE deflection values with the corresponding field values at sections S1, S2, and S3 in all monitored bridges. The results showed that the maximum difference between the FE and field deflection results in Lawrence, Ford, Madison, and McDonough Bridges was less than 7%, 10%, 12%, and 5%, respectively. The difference between the deflections in the stable cases was even smaller because the source of the load was only the fresh weight of the concrete. Similarly, Figure 62 compares exterior-girder FE deflections with the field values. As illustrated, the results are very similar to those recorded at the bracket locations, with the differences being within acceptable limits. However, due to inward rotation of the exterior girder in Ford Bridge, the upward deflection in the exterior girder in the field was not predicted in the FE analysis of that bridge.

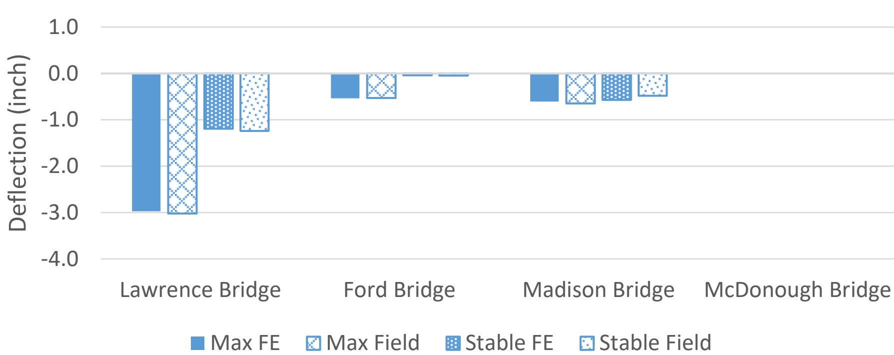
In conclusion, the FE modeling technique used to mimic behavior of the field-monitored bridges and the temporary rotation-prevention systems can predict rotations, strains, and deflections within an acceptable error. These errors are due to the imperfect nature of the construction process, which cannot be fully simulated. However, this FE model can be used to expand this research to find the effects of different parameters on the behavior of the girders during construction, as explained in Chapter 4 of this report.



(a) Section S1

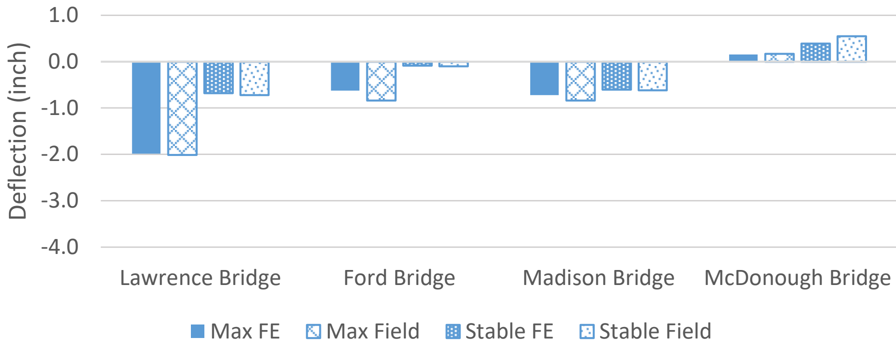


(b) Section S2

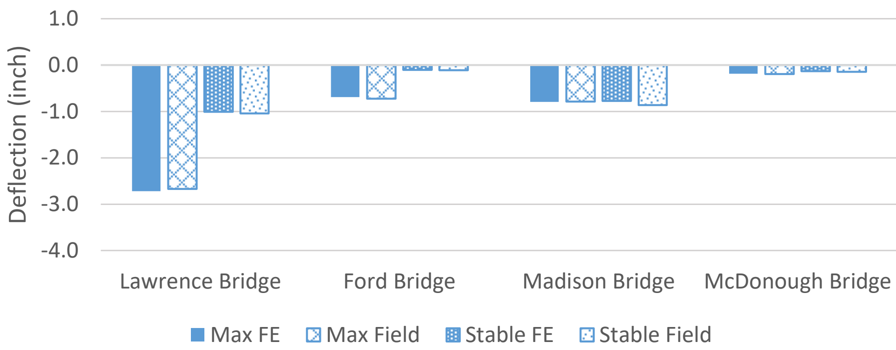


(c) Section S3

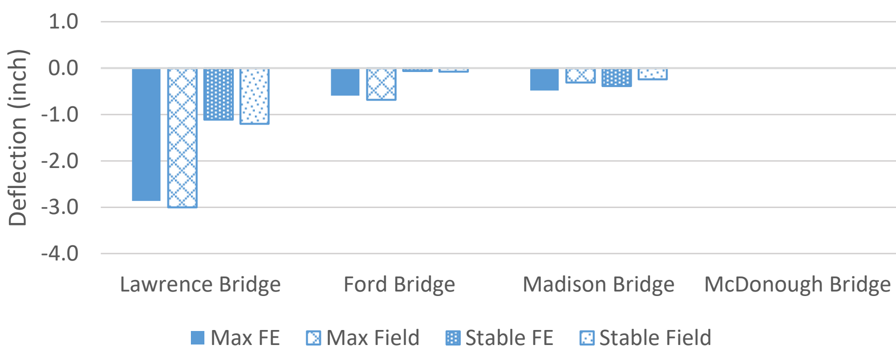
Figure 61. Deflection field data vs. FE results for overhang bracket.



(a) Section S1



(b) Section S2



(c) Section S3

Figure 62. Deflection field data vs. FE results for exterior girder.

CHAPTER 4: FINITE-ELEMENT ANALYSIS OF ADDITIONAL BRIDGES

4.1 OVERVIEW

Due to the time limitation of the research project, one bridge with a small skew in LaSalle County (LaSalle Bridge) and one highly skewed bridge in Madison County (Madison-II Bridge) were not field-monitored during construction. However, FE models for these two bridges with and without the temporary bracing system (pipe–tie system) were developed to examine further the effectiveness of the system. This section gives the bridges' descriptions and the results from FE analysis. Table 3 shows the basic information for these two bridges.

Table 3. Properties of LaSalle Bridge and Madison-II Bridge

	LaSalle Bridge	Madison-II Bridge
Contract No.	66C58	76E68
Structure No.	050-0258	060-0002
Beam Type	W33x141	W36x182 & W33x130
Skew	11°	50°51′00″
No. of Spans	3	3
Span Length	60′+80′+60′	68′-11″+87′-8″+69′-3/4″
Overhang Width	2′-9.5″	3′-1.5″
Girder Spacing	5′-11″	6′-7″
Permanent Diaphragm Type	C12x25	W12x40 & W16x36
Largest Diaphragm Spacing	20′	25′
B/D Ratio	7.27	9.09
Finishing Machine Location	On overhang	On overhang

4.2 FINITE-ELEMENT ANALYSIS OF LASALLE BRIDGE

4.2.1 Bridge Description

The continuous, three-span, steel-girder bridge consists of six W33x141 girders spaced at 71 in. with an overhang width of 33.5 in. This bridge has a small skew of 11°, and the span lengths are 60 ft, 80 ft, and 60 ft, respectively. The diaphragm sections are C12x25, connected to the girders using L6x4x1/2 angles. Maximum diaphragm spacing is 20 ft, as shown in Figure 63. The finishing machine weight was computed based on the Bidwell 4800, and the weight of each wheel was taken as 1,200 lb.

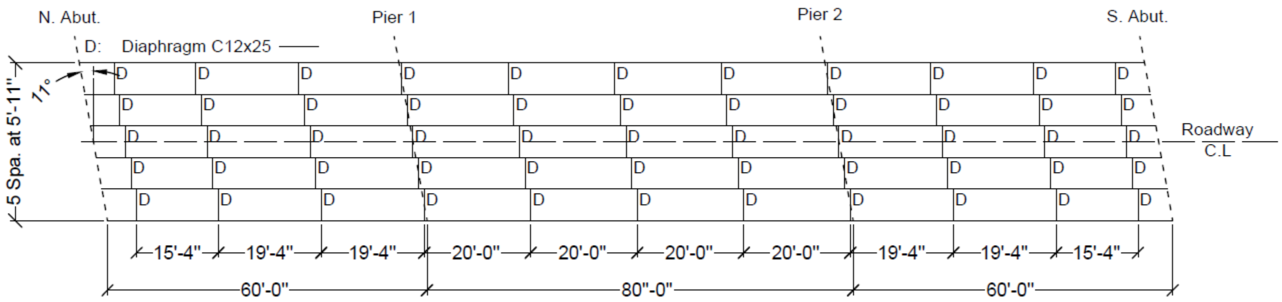


Figure 63. Plan of LaSalle Bridge.

4.2.2 Rotation Results

The maximum rotation was recorded in the exterior girder at a section approximately 112 ft from the north abutment. At the center of the web of the exterior girder, the FE results showed that before adding any temporary bracing system, the maximum rotation was 1.01°, which is approximately 2.4 times the IDOT rotation limit for this bridge. However, as shown in Figure 64, adding one pipe–tie system at the mid-distance between each two diaphragms resulted in a maximum rotation of 0.29°, which is 9% below the limit. Moreover, because the diaphragms in this bridge are not connected to the full depth of the girders' web, IDOT recommends utilizing the pipe–tie system along the exterior bays, with a spacing of 8 ft. In this case, the maximum rotation was 0.25°, 23% below the IDOT limit. Therefore, it can be concluded that using the pipe–tie system is effective in reducing the maximum rotation in the exterior girders.

Figure 64. Maximum rotation for LaSalle Bridge.

4.2.3 Strain (Stress) Results

The strain/stress results are shown in Figure 65. When the temporary bracing system was spaced every 8 ft, the maximum strain in the transverse tie was recorded as 251 micro-strains (tension); and the corresponding force was 2,238 lb. The maximum strain in the pipe was 57 micro-strains (compression), and the corresponding force was 2,792 lb. These values were recorded when the finishing machine was approximately at the location of the temporary bracing system.

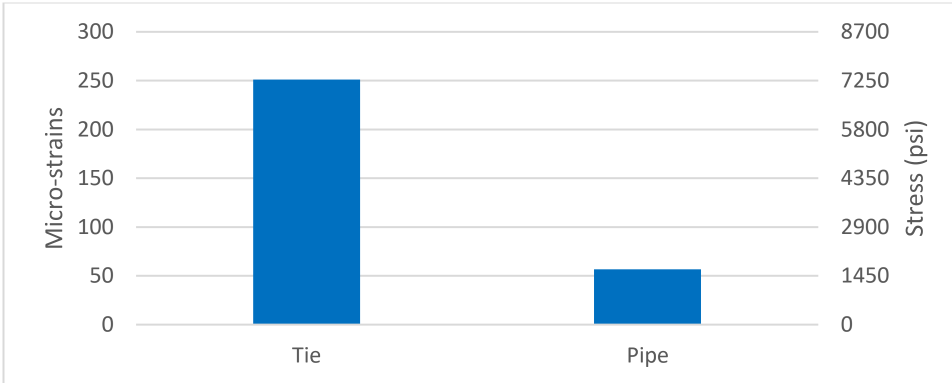


Figure 65. Maximum strain (stress) for LaSalle Bridge.

4.3 FINITE-ELEMENT ANALYSIS OF MADISON-II BRIDGE

In skewed bridges, previous research showed that differential deflection of the girders can lead to large rotations in both exterior and interior girders during deck-slab construction. These rotations create a permanent global twist in the superstructure toward the acute angles of the skewed bridge (Hraib, 2019). Therefore, for bridges with high skew angles, it is more accurate to compare the deflection of the overhang relative to the global twist of the structure (the transverse slope at a particular section). Deflection of the overhang will be referred to as the *relative deflection.* It is computed with respect to the deflection and slope of the whole superstructure at a particular section that is perpendicular to the centerline of the bridge. Relative deflection results will be compared to the IDOT overhang deflection limit of 3/16 in. It should be noted that by utilizing appropriate haunches along the girders, the effect of the global twist of the girders on the final thickness of the deck slab becomes minimal. However, temporary rotation of the girders due to the finishing machine has the most impact on the final thickness of the deck slab. In this section, the effectiveness of the pipe–tie system is examined in a bridge with a skew angle of 51°.

4.3.1 Bridge Description

The continuous, three-span steel-girder bridge consists of seven steel girders, six of which are W36x182 spaced at 79 in. The seventh W33x130 exterior girder is 48 in. from the adjacent W36x182 girder, as shown in Figure 66. The bridge is heavily skewed, with an angle of approximately 51°. The diaphragms in the exterior bay, between G1 and G2, are W12x40 beams connected to the girders using L angles. However, the diaphragms at all other bays are W16x36 beams connected to the girders in a similar manner. Maximum diaphragm spacing is 25 ft, and overhang width is 37 in.

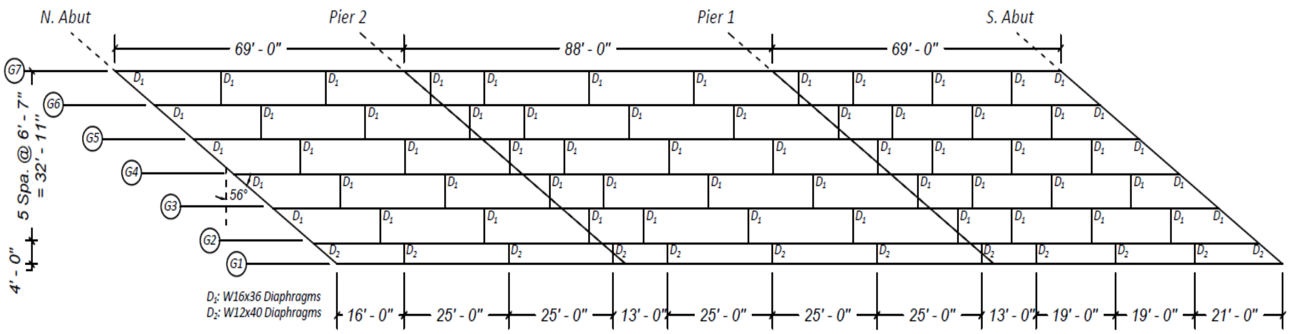


Figure 66. Plan of Madison-II Bridge (Hraib, 2019).

FE models of this bridge were developed to examine the behavior of the bridge before and after adding the pipe–tie system. Finishing machine loading was computed based on GOMACO C-450 finishing machine weight that accounted for the machine length when it moved parallel with the skew. The weight of each wheel was taken as 1,210 lb. The pipe–tie system was spaced at 11 ft in both exterior bays. This spacing was calculated based on a B/D ratio of 4 for the smallest depth of the girders (33 in.) of this bridge. Moreover, due to the type of diaphragm connections, which does not utilize the full depth of the girder web, the system was spaced regardless of the diaphragm spacing along the bridge.

4.3.2 Rotation Results

Although the rotation of the exterior girders in heavily skewed bridges is not the best indicator for the final thickness of the deck slab (Hraib, 2019), maximum rotation values in the exterior girders were compared before and after adding the temporary bracing system.

The maximum rotation of Madison-II Bridge is shown in Figure 67. The results indicated that before adding the pipe–tie system, maximum rotation in the exterior girder G1 (W33x130 section) was 3.2°. However, the corresponding value for exterior girder G7 (W36x182 section) was recorded as 1.06°. It was noticed that the maximum rotation in G1 was more than three times as large, compared with the maximum rotation in G7. This behavior is mainly caused by differential deflection of the girders during and after pouring of the concrete deck. Due to the relatively smaller section of W33x130 compared with the other girders, deflection in G1 is higher than that in the rest of girders. Moreover, adding the pipe–tie system led to a reduction of only 20% in maximum rotation of G1. The low reduction in rotation is because the rotation of G1 is mainly caused by differential deflection caused by the different moment of inertia between G1 and G2, rather than the torsional moment acting on the overhang. However, the system was able to reduce rotation by 74% in G7 because differential deflection is not the primary reason for exterior-girder rotation.

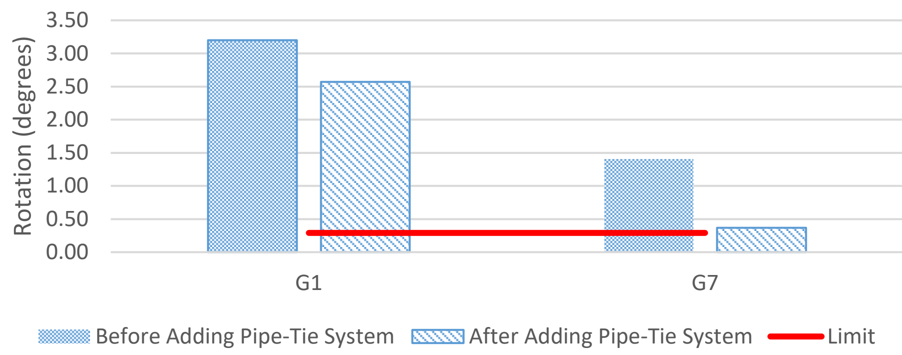


Figure 67. Maximum rotation for Madison-II Bridge.

4.3.3 Overhang Deflection Results

To evaluate accurately the effectiveness of the pipe–tie system in this bridge, ten sections perpendicular to the bridge centerline were selected to examine the deflections at the top of the girders and the overhangs. The selected sections are shown in Figure 68. As the finishing machine moved parallel to the skew, sections 3 to 8 were examined with two different loading cases. Each case assumed that the finishing machine wheels passed over the overhang at the examined section.

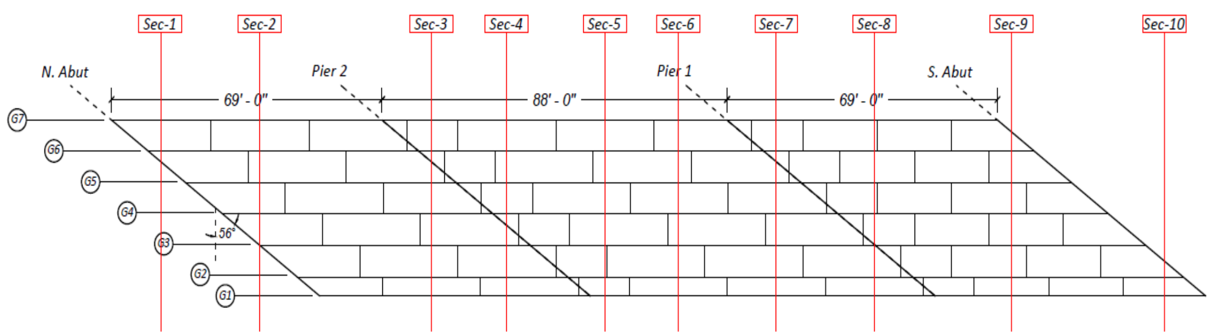


Figure 68. Selected sections for evaluating overhang deflection (Hraib, 2019).

Relative deflection of the overhang was computed at each section. For each loading case, the finishing machine passed over one of the overhangs at the examined section; and relative deflection values were compared before and after adding the pipe–tie system to the bridge. Deflections at Sect-1, Sect-5, and Sect-6 are plotted in Figure 69. The results showed that after the temporary bracing system was added to the bridge, the overhang slope (adjacent to G7) changed significantly to align with the overall slope of the deck slab at all sections. However, that was not the case for the overhang adjacent to G1, in which the slope of the overhang was drastically different than the transverse slope of the deck slab even after the temporary bracing system was added.

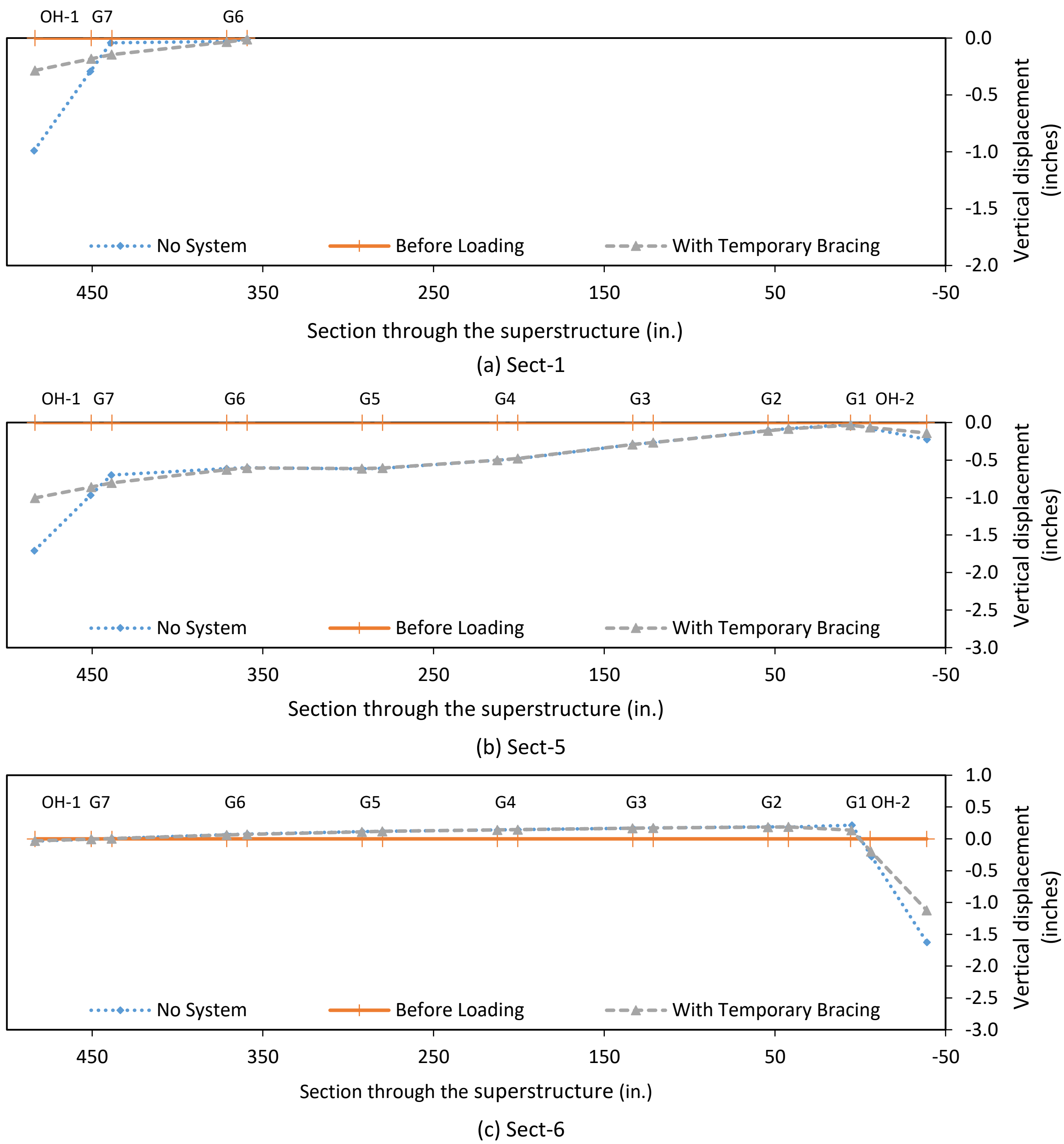


Figure 69. Top flanges and overhang deflections in Madison-II bridge (Hraib, 2019).

However, to better understand the effectiveness of the pipe–tie system on deflection of the overhang during deck construction, relative deflections in both overhangs are summarized in Table 4. The results showed that the relative deflections of the overhang adjacent to G7 (OH-1) were always higher than the IDOT limit. However, after adding the temporary bracing system, deflection values dropped significantly, to below the limit. In contrast, before adding the system, relative deflections of the overhang adjacent to G1 (OH-2) were also higher than the IDOT limit. However, adding the pipe–tie system did not reduce relative deflection below the IDOT limit.

Table 4. Relative Deflection in Overhang Before and After Adding the System

Section	Loading Case	OH-1 Deflection (in.)		OH-2 Deflection (in.)	
		Before	After	Before	After
Sect-1	Case-4	-0.942	-0.066	N/A	N/A
Sect-2	Case-10	-0.948	0.082	N/A	N/A
Sect-3	Case-21	-0.314	-0.076	-0.569	-0.329
	Case-7	-0.003	0.003	-1.787	-0.983
Sect-4	Case-25	-0.868	-0.054	-0.453	-0.296
	Case-12	-0.001	0.008	-1.134	-0.775
Sect-5	Case-32	-0.949	-0.081	-0.268	-0.160
	Case-19	-0.001	0.000	-0.678	-0.392
Sect-6	Case-37	-0.647	-0.061	-0.573	-0.382
	Case-23	0.003	0.003	-1.907	-1.206
Sect-7	Case-43	-0.314	-0.075	-0.507	-0.305
	Case-29	-0.004	0.003	-1.731	-0.963
Sect-8	Case-49	-0.459	-0.079	-0.551	-0.368
	Case-35	0.000	0.004	-1.617	-1.082
Sect-9	Case-44	N/A	N/A	-1.087	-0.874
Sect-10	Case-54	N/A	N/A	-1.581	-0.894

Therefore, it can be concluded that the pipe–tie system can reduce relative deflection of the overhangs in heavily skewed bridges. However, the system cannot reduce relative deflections in the overhang, or excessive girder rotation caused by differential deflection of the girders, regardless of whether the source of the differential deflection is a high skew angle or the section properties of a different girder.

CHAPTER 5: ASSESSMENT OF B/D RATIO TO LIMIT EXTERIOR-GIRDER ROTATION

5.1 EFFECT OF THE SIZE OF TRANSVERSE TIE AND DIAGONAL PIPE

To ensure that the performance of the temporary rotation-prevention system is satisfactory, IDOT recommended the use of #5 rebars for the transverse ties and 2.5-in.-diameter pipes for the diagonal pipes. However, different sizes of transverse ties and diagonal pipes were observed at construction sites during field monitoring of the four bridges. A summary of the sizes of ties and pipes used in the instrumented bridges is shown in Table 5. Lawrence Bridge did not implement any temporary rotation-prevention system during deck construction because a deep steel-girder section was used. Madison and Ford Bridges used #5 rebars for transverse ties, combined with 2.5-in.-diameter pipes as the rotation-prevention system. However, McDonough Bridge applied #4 rebars as transverse ties; and the diameter of the diagonal pipes was 1.5 in. Therefore, it is necessary to evaluate the effect of the size of ties and pipes on the rotation of the exterior girder.

Table 5. Size of Transverse Ties and Diagonal Pipes

Bridge Name	Size of Transverse Tie	Diameter of Diagonal Pipe
Lawrence Bridge	*No temporary bracing system*	
McDonough Bridge	#4	1.5″ SCH 40 round pipe
Madison Bridge	#5	HSS 2.5X2.5X1/4 square pipe
Ford Bridge	#5	2.5″ SCH40 round pipe

FE models were developed in SAP2000 to study the effect of different sizes of transverse ties and diagonal pipes. Figure 70 shows an example of the FE model. To consider the most critical case during bridge deck construction, the spacing between the permanent diaphragms was considered as 25 ft, the maximum allowable diaphragm spacing specified by the IDOT *Bridge Manual* (2012). The depth of girders is 30 in., resulting in an original B/D ratio of 10. The beam spacing was considered as 72 in., and the temporary bracing systems are placed every 8 ft according to IDOT recommendation, which reduces the B/D ratio to 3.2. Details of the combination of pipe sections and sizes of ties used in the FE analysis are listed in Table 6.

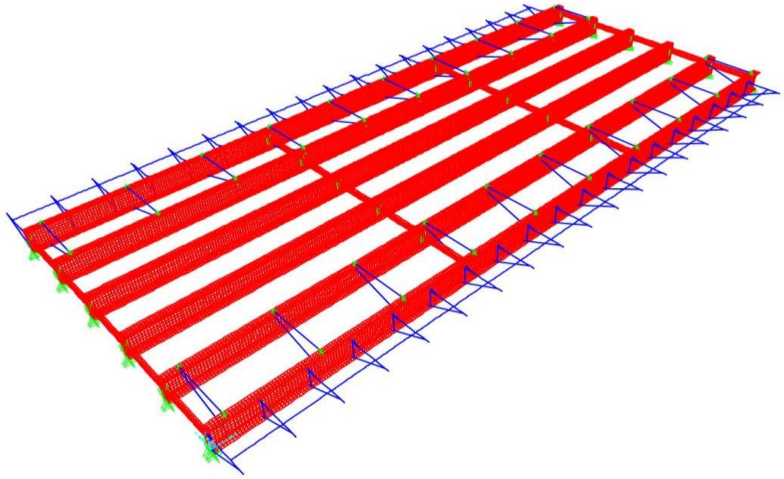


Figure 70. FE model for evaluating size of ties and pipes.

Table 6. Combinations of the Sections of Pipe and Tie

Case	Pipe Section	Size of Tie
Case-I	1.5″ SCH 40 round pipe	#4
Case-II	1.5″ SCH 40 round pipe	#5
Case-III	2.5″ SCH 40 round pipe	#4
Case-IV	2.5″ SCH 40 round pipe	#5
Case-V	HSS 2.5X2.5X1/4 square pipe	#4
Case-VI	HSS 2.5X2.5X1/4 square pipe	#5

Comparison of the maximum rotation values of the bridge models with different sizes of ties and pipes is shown in Figure 71. The results indicated that maximum rotation values were reduced approximately 3% when using #5 rebar instead of #4 rebar for transverse ties. Also, around 3% difference in rotation values was observed when a larger round-pipe section was implemented during deck construction. The square pipe showed very similar performance when compared with the round pipe of similar size; less than 1% difference was found, based on the results of FE analysis of the two pipe sections. Thus, the size of transverse ties and diagonal pipes has minimal effect on the prevention of exterior-girder rotation.

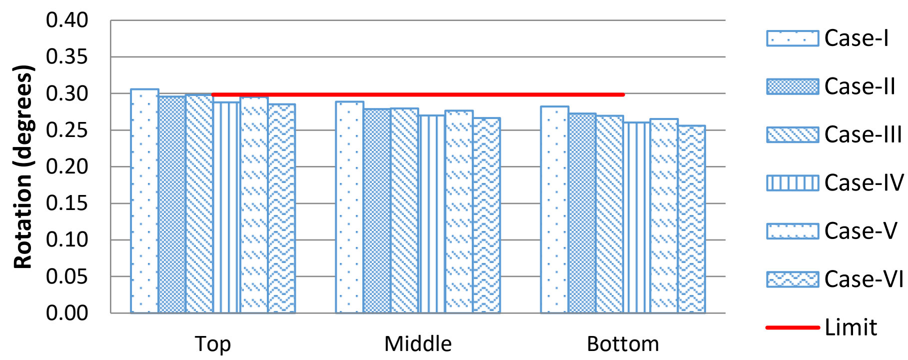


Figure 71. Comparison of maximum rotation values.

The maximum forces that occurred in the transverse ties are illustrated in Figure 72. Values of the maximum forces were very similar in all six cases examined, with less than 4% difference. Moreover, the sections of diagonal pipes showed a limited effect on the forces in the ties. The maximum forces in the transverse ties were approximately 2,215 lb when #4 bars were used and 2,150 lb when #5 bars were utilized. The yield forces for #4 and #5 rebars, shown in Table 7, are five and eight times larger than the maximum forces observed in #4 and #5 transverse ties, respectively.

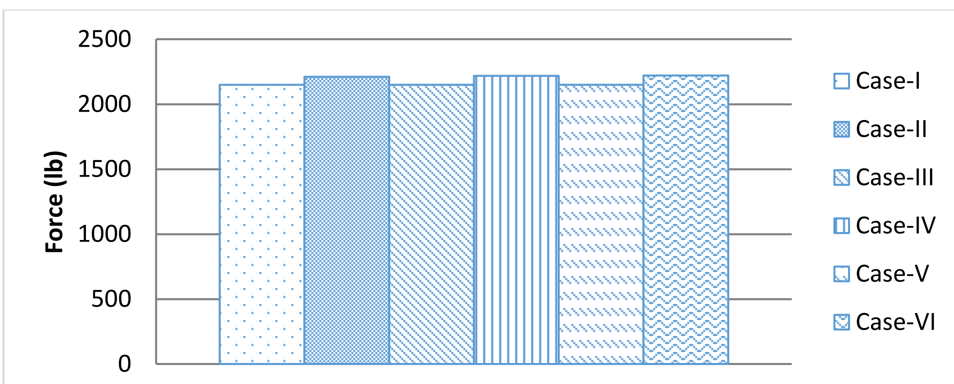


Figure 72. Comparison of maximum forces in transverse ties.

Figure 73 shows the comparison of maximum axial forces in the diagonal pipes for all six cases. The size of transverse ties and diagonal pipes does not have a significant impact on the forces, with less than 2% of a maximum difference. Because the diagonal pipes experience only compression forces during deck construction, the buckling load for each pipe section, as shown in Table 7, was computed to ensure the effectiveness and the capacity of the bracing systems. The results indicated that the maximum forces in diagonal pipes were only 18%, 6%, and 3.5% of the buckling load for 1.5" schedule-40 round pipe, 2.5" schedule-40 round pipe, and HSS 2.5X2.5X1/4 square pipe, respectively.

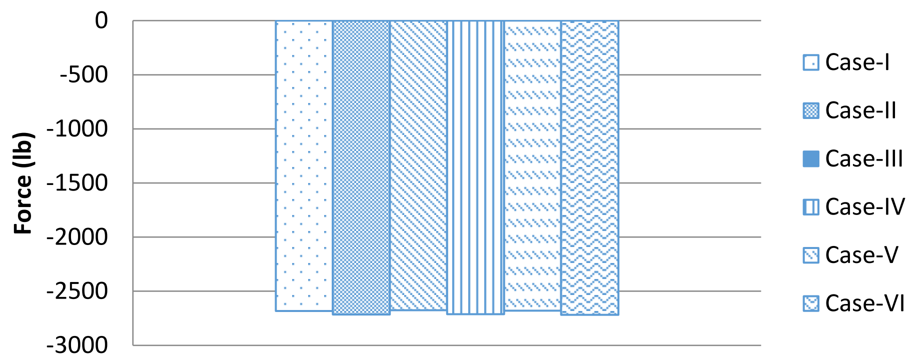


Figure 73. Comparison of maximum forces in diagonal pipes.

Table 7. Yield and Buckling Forces for Transverse Ties and Diagonal Pipes

Structural Component	Yield Force (lb)	Buckling Force (lb)
#4 rebar	12,000	—
#5 rebar	18,600	—
1.5″ SCH40 round pipe	—	14,579
2.5″ SCH40 round pipe	—	44,551
HSS 2.5X2.5X1/4 square pipe	—	76,682

The results from FE analysis indicated that the size of ties and pipes has minimal effects on the effectiveness of the transverse tie–diagonal pipe system. However, to ensure the system has sufficient stiffness, it is suggested to use transverse ties with a section no less than #4 rebar, while the diameter of pipes should not be less than 1.5 in. The results indicate that the IDOT recommendation, which requires use of 2.5-in. round pipes and #5 rebars, is adequate for preventing exterior-girder rotation.

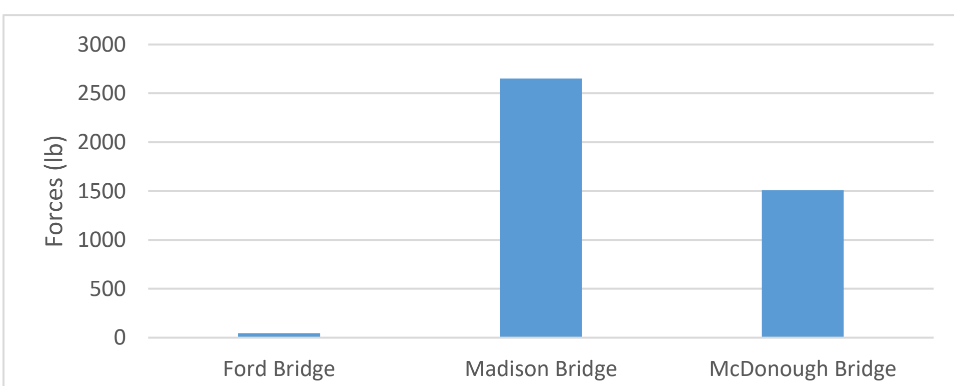


Figure 74. Maximum forces in transverse ties for monitored bridges.

Because the selection of steel hangers is generally made based on the working loads of transverse ties, it is necessary to evaluate the maximum force in steel hangers during bridge deck construction. The FE analysis shows that the largest force in the transverse tie is 2,700 lb. In addition to the FE analysis, the maximum forces in transverse ties for the monitored bridges are shown in Figure 74, which indicates that maximum forces for the subject structures were all under 3,000 lb. Therefore, it is recommended that the load capacity of steel hangers should be no less than 3,000 lb. However, a safety factor must be added to this load capacity to ensure the safety of the bridge and the effectiveness of the pipe–tie system.

5.2 EFFECT OF PROTECTIVE RUBBER AT THE END OF PIPE STRUTS

During deck construction, the diagonal pipes act as compression members, reacting against the top of the first interior girder and the bottom of the exterior girder. In some cases, the paint or other protective coating can be damaged at the location between the girder and the pipe when the construction loads move along the bridge. Therefore, protective rubber is sometimes used at both ends of the pipes to prevent damage to the girder coating. A protective rubber was used in the McDonough Bridge to prevent damage to the galvanized girders, as shown in Figure 75. The low elastic modulus of the protective rubber may lead to larger lateral deflections at the top and bottom of the girder's web, which can increase the transverse rotation of the exterior girder. Thus, three FE models for McDonough Bridge were designed to assess the effect of the protective rubber: models without any temporary rotation-prevention system and with and without protective rubber.

(a) Protective rubber at the bottom of pipe

(b) Protective rubber at the top of pipe

Figure 75. Protective rubber used at both ends of pipes in McDonough Bridge.

The effect of the protective rubber at S1 is shown in Figure 76. When the pipes were in direct contact with the girders' web, without a protective rubber, rotation values were reduced by more than 80%, compared with the case in which no temporary bracing system was used. The field rotation concurred with the results from the FE model that used protective rubber, with only a 35% reduction in the exterior-girder rotation observed.

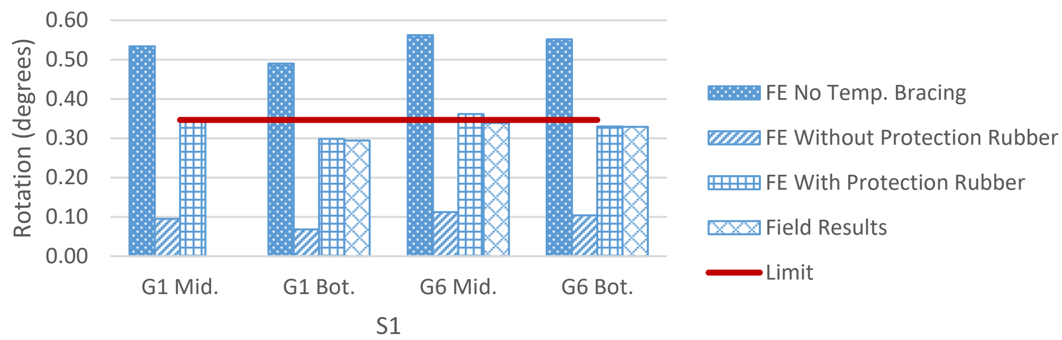


Figure 76. Effect of protective rubber at S1.

The transverse tie–diagonal pipe system decreased rotation values by more than 30% at all sensor locations in S2 when the protective rubber was absent, as shown in Figure 77. Using the protective rubber compromises the effectiveness of the rotation-prevention system, causing only a 15% reduction of exterior-girder rotation, compared with the model with no temporary rotation-prevention system.

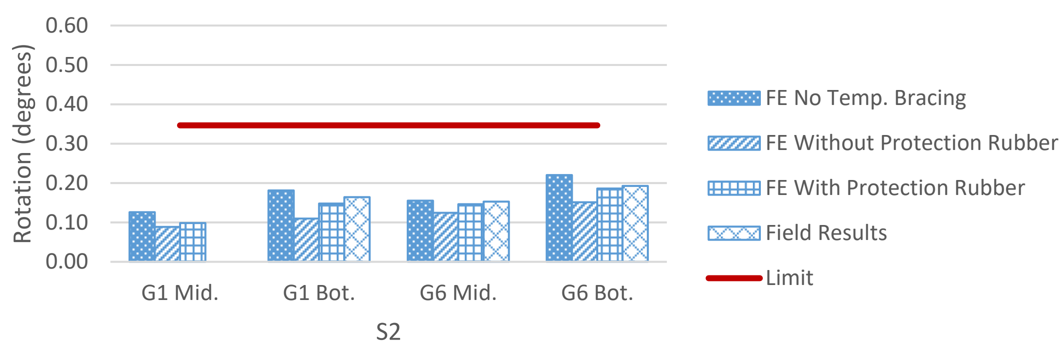


Figure 77. Effect of protective rubber at S2.

The exterior-girder rotation at S3 (shown in Figure 78) followed the same trend as at S1. Compared to the case without a temporary bracing system, the FE model without protective rubber showed an 80% reduction in exterior-girder rotation, while only a 35% reduction was observed when the protective rubber was used.

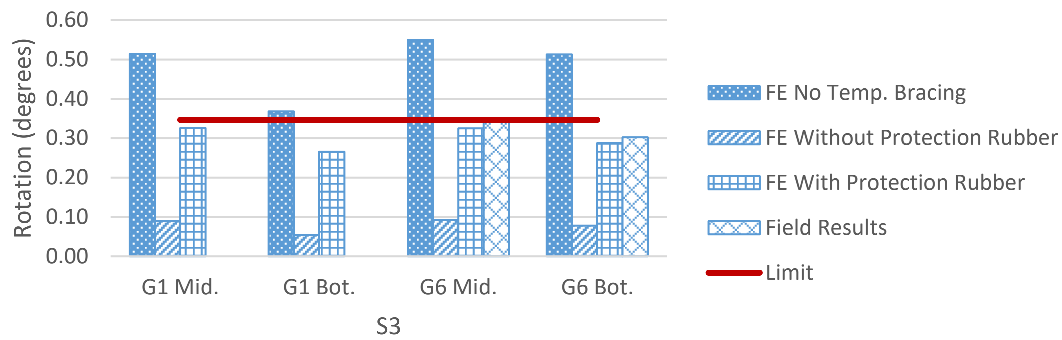
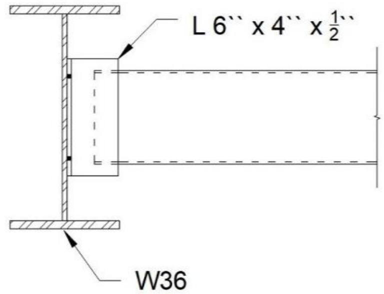


Figure 78. Effect of protective rubber at S3.

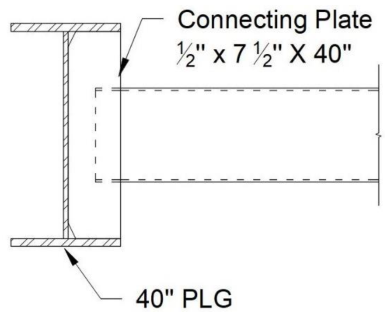
The FE analysis indicated that the protective rubber had a significant impact on the effectiveness of the transverse tie–diagonal pipe rotation-prevention system. Rotation on the exterior girder can be dramatically reduced (by more than 50%) when the protective rubber is removed. Therefore, it is recommended to use other materials, such as hardwood or hard plastic pieces that have a larger elastic modulus, if protection is desired during deck construction.

5.3 ASSESSMENT OF UNBRACED-LENGTH-TO-GIRDER-DEPTH RATIO

The B/D ratio has been demonstrated to be an effective and efficient factor in evaluating the capability of a girder section to resist transverse rotation. Generally, the B/D ratio is calculated based on the spacing between the diaphragms/cross-frames and the girder depth. However, several other factors, such as the type of lateral bracing system and depth of the connecting plate, may influence evaluation of exterior-girder rotation. Figure 79 shows the connection plates used in McDonough and Madison Bridges. McDonough Bridge used L-angle steel bolted to the girders' webs, with a connection depth of 30 in. The connecting plates for Madison Bridge were welded on the webs of the girders, with a full depth of 40 in., equal to the depth of the girders. Also, the top and bottom of the connecting plates were welded to the flanges.



(a) Partial-depth connection plate in McDonough Bridge



(b) Diaphragm with full-depth connecting plate in Madison Bridge

Figure 79. Diaphragm connection of McDonough and Madison Bridges.

Table 8. Case Description for FE Analysis

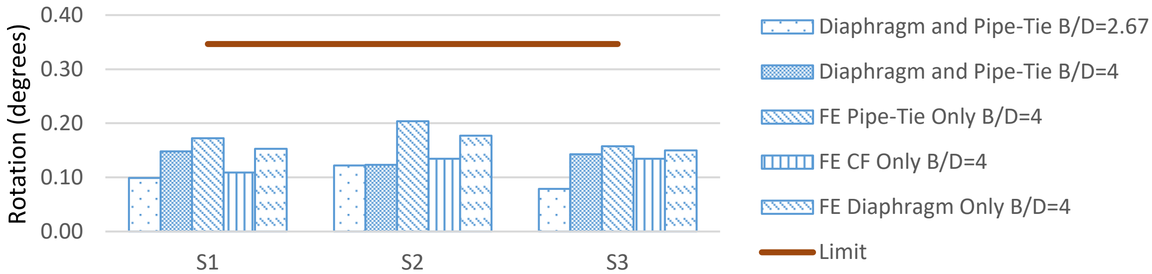
Case No.	McDonough Bridge	Madison Bridge
1	Including both diaphragm and pipe–tie system, B/D = 2.67	Including both diaphragm and pipe–tie system, B/D = 2.4
2	Including both diaphragm and pipe–tie system, B/D = 4	
3	Pipe–tie system only, B/D = 4	
4	Cross-frame only, B/D = 4	
5	Diaphragm only, B/D = 4	

Note:

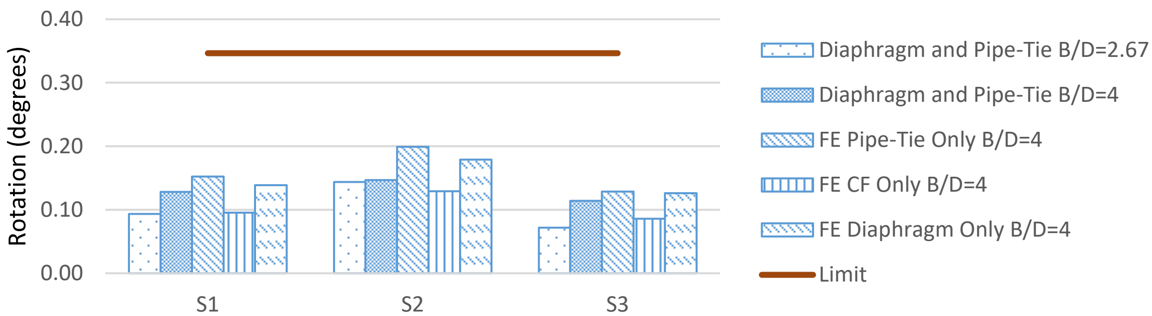
For Cases 1, 2, and 5, details of the connecting plates between the diaphragms and girders are based on the original bridge design.

For Cases 1, 2, and 3 of McDonough Bridge, the pipe–tie systems were modeled without considering the effect of the protective rubber at the ends of the diagonal pipes.

To evaluate the effects of the different types of lateral bracing systems, the depth of the connecting plates, and the B/D ratio, five cases for McDonough and Madison Bridges were simulated using the FE method. All the cases for both bridges are shown in Table 8. Case 1 considered the original design of the rotation-prevention systems, using transverse tie–diagonal pipe systems spaced every 8 ft in both bridges. Because the maximum allowable B/D ratio was recommended as 4 in Phase I of this study, cases 2 to 4 assessed different types of lateral bracing systems with a B/D ratio under 4. Furthermore, the effect of the depth of the connecting plate was evaluated because different connecting plates were used in the two bridges. The protective rubbers at the ends of the diagonal pipes were not considered in the FE analysis to ensure similarity between McDonough and Madison Bridges.



(a) Middle of the web of exterior girder

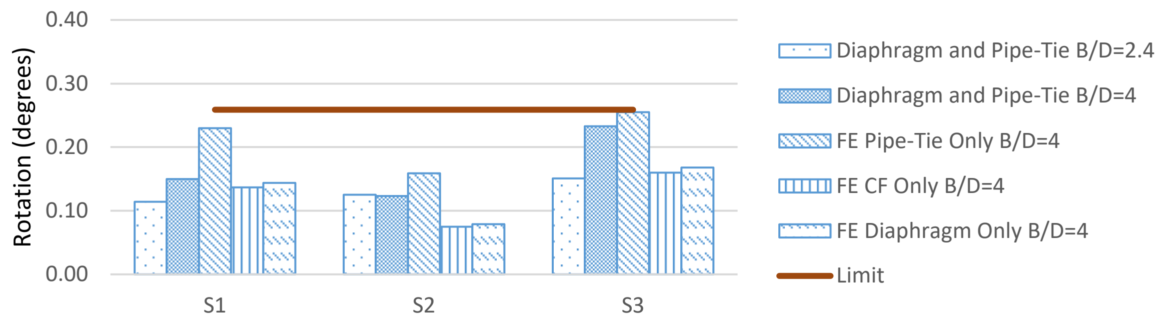


(b) Bottom of the web of exterior girder

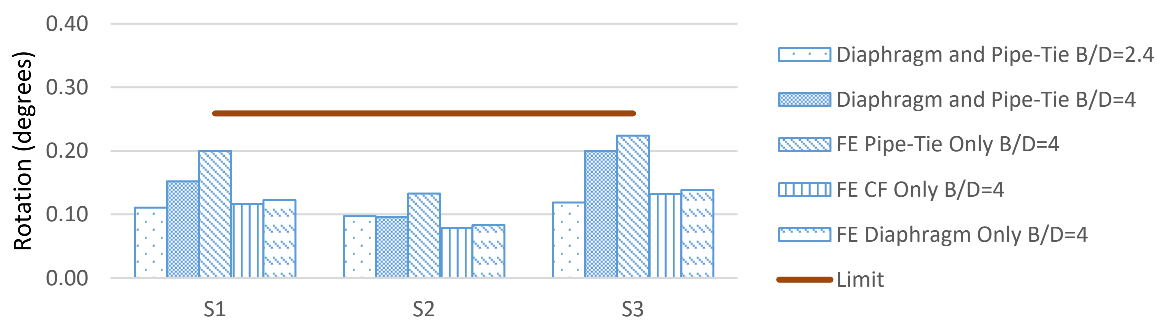
Figure 80. Comparison of rotation values for McDonough Bridge.

Comparisons among all five cases, as well as the rotation limit, for McDonough Bridge is shown in Figure 80. At the center of the web of the exterior girder, a change in the B/D ratio from 2.67 to 4 resulted in an increase in the rotation value by 50% and 82% at S1 and S3, respectively. However, the change in the B/D ratio had minimal effect on exterior-girder rotation at S2 because permanent diaphragms were installed at this section and the rotation was mostly resisted by the diaphragms. Rotation values increased by 37% and 60% for S1 and S3 at the bottom of the girder's web, while it increased by only 2% for S2. Changing of the lateral bracing systems also had a significant impact on exterior-girder rotation. When the B/D ratio was maintained as 4, the FE analysis indicated that using the transverse tie–diagonal pipe systems and diaphragm independently led to increase of the rotation values by more than 50% and 40%, respectively, when compared with the case using only cross-frames.

The results from the FE analysis for Madison Bridge are shown in Figure 81. Rotation values increased by more than 40% for S1 and S3 at both the top and bottom of the girder's web when the B/D ratio was increased from 2.4 to 4. When comparing among the three different lateral bracing systems, the cross-frames and diaphragms performed similarly, with a less than 5% difference between rotation values. However, when implementing only the transverse tie–diagonal pipe systems, rotation values dramatically increased by more than 60% for all three sections.



(a) Middle of girder web



(b) Bottom of girder web

Figure 81. Comparison of rotation values for Madison Bridge.

The diaphragms used in Madison Bridge performed similarly to cross-frames in resisting transverse rotation. However, the performance of the diaphragms showed different behaviors between McDonough and Madison Bridges, which is because full-depth connecting plates were used in Madison Bridge, and the whole web of the girder was restrained by the diaphragms/cross-frames. McDonough Bridge, which used partial-depth connecting plates between the web and diaphragms, showed a significant reduction in preventing exterior-girder rotation. Therefore, the B/D ratio can be modified based on the depth of the connecting plate for conservative consideration. When the partial-depth connecting plates are used during bridge deck construction, the effect of the diaphragms should be excluded from calculation of the B/D ratio. However, the lateral stiffness provided by the diaphragms can be considered when using cross-frames or diaphragms with full-depth connecting plates.

The results also indicated that rotation values for both bridges were below the allowable limit when the temporary rotation-prevention systems were spaced with a B/D ratio of four. Lawrence Bridge, which did not implement any temporary bracing system, had a B/D ratio of 3.95; and rotation values were also less than the limit value, based on the field data and FE analysis. Thus, it is suggested that the spacing between the transverse tie–diagonal pipe systems should be less than four times the depth of the girder, resulting in a maximum allowable B/D ratio of 4.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

In this chapter, the results of this research are summarized based on the field-monitoring data and FE analysis. Four bridges were monitored in the field during bridge deck construction, and FE models were validated with the field data. Additional FE analysis was also conducted to evaluate the effects of the dimensions of the rotation-prevention system and the depth of the connecting plates between the diaphragms and the girders' web. A recommended B/D ratio was also assessed and recommended for all types of bridge girders to limit exterior-girder rotation. The results of this study can be used to improve understanding of the bridge deck-overhang construction and the structural behavior of the bridge girder systems during deck construction. The conclusions are provided in the following subsections.

6.2 FIELD-MONITORING DATA AND FINITE-ELEMENT ANALYSIS

Based on the field-monitoring data from the four selected bridges and the FE analysis, the following conclusions can be drawn:

- According to the field monitoring results of Lawrence Bridge, temporary rotation-prevention systems can be omitted when deep-plate girders are used during deck construction provided the B/D ratio is no larger than the maximum value of 4.0.
- Improper installation or inadequate tightening of transverse ties can result in significantly reduced effectiveness of rotation-prevention systems.
- The location of the finishing machine plays an important role in affecting rotation of the exterior girder. When the rails of the finishing machine are located at the center of the exterior girders, rotation can be dramatically reduced. However, it is important to be cautious when installing the rails of the finishing machine because improper measurement of the rail locations can result in the finishing machine wheels shifting, causing inward rotation of the exterior girder.
- Skew angle can affect the rotation of the exterior girders significantly. For highly skewed bridges, differential deflections play an important role in exterior girder rotation. The temporary bracing system cannot reduce the excessive girder rotations caused by differential deflection. However, the temporary system can reduce the temporary rotations caused by the finishing machine loading.

6.3 ADDITIONAL FINITE-ELEMENT ANALYSIS FOR ASSESSING THE B/D RATIO

Additional FE models were developed in SAP2000 to evaluate the effects of geometric properties of the rotation-prevention system, the protective rubber, and the diaphragm-to-girder connections. The following conclusions are made based on results from the FE analysis.

- The size of transverse ties and dimensions of diagonal pipes has limited effects on exterior-girder rotation. However, enough axial force capacity is required to resist the imposed compression and tensile stresses in the pipe and the tie, respectively.
- Protective rubber or other compressible material used at the ends of the diagonal pipes reduces effectiveness of rotation-prevention systems. With use of protective rubber, the rotation values are dramatically increased up to 300%, compared to not using protective rubber.
- Diaphragms with partial-depth connecting plates are less effective in resisting exterior girder rotations and should be discounted or ignored when determining the effective B/D ratio. However, when full-depth connecting plates are used between the diaphragms and the web of the girders, the diaphragms should be included in determining the B/D ratio.
- Performance of the diaphragms with full-depth connecting plates is similar to using cross-frames to prevent transverse rotation. The FE analysis showed a difference of less than 5% between the two systems.
- Typical IDOT cross frames proved to be effective in resisting exterior girder rotations during deck slab construction and can their spacing be included when determining effective B/D ratio.

6.4 RECOMMENDATIONS

Based on the results of this study, the following recommendations can be made for deck overhang construction.

- Quality control should be conducted when installing temporary rotation-prevention systems and prior to pouring the concrete. Transverse ties need to be properly installed and tightened to ensure the effectiveness of the rotation-prevention systems.
- When the rails of the finishing machine are installed on the top of exterior girders, the location should be carefully measured to prevent the rails from shifting, which may cause inward rotation in the exterior girders.
- Based on the findings of this study and current practice, the size of rebar used for transverse ties should be no less than #4; and the diameter of diagonal pipes should be at least 1.5 in.
- The load capacity of steel hangers, which are used to hold the transverse ties, is recommended to be at least 3,000 lbs. A safe working load should be determined for the anticipated construction loads and following the manufacturer’s recommendations. A safety factor may be added to the recommended force to ensure the safety of the bridge and the effectiveness of the temporary bracing systems during bridge deck construction.
- Protective rubber or similar compressible materials at the ends of diagonal pipes are not recommended due to their negative impact on the effectiveness of rotation-prevention systems. However, some alternative materials with higher elastic modulus, such as hardwood and hard plastic pieces, may be used.

- During bridge design, full-depth connecting plates are recommended to be used between the diaphragms and girders to increase the ability of the bridge girder systems to resist rotation.
- Diaphragms with partial-depth connecting plates should be discounted or ignored when calculating the B/D ratio. However, the cross-frames and diaphragms with full-depth connecting plates can be included during determination of the B/D ratio.
- The spacing between the lateral bracing systems, including the effective permanent bracings and temporary rotation-prevention systems, should be limited to four times the depth of the girder (B/D less than or equal to 4).

6.5 FEEDBACK FROM CONTRACTORS

After conducting the field monitoring for the bridges that utilized the temporary bracing system (pipe–tie), we surveyed contractors that used this system for their feedback and opinions. This process is a very important task in this research because, as they have used different bracing systems in the past, their feedback helps in understanding the efficiency and the practicality of the proposed temporary bracing system. The list of questions (survey) was provided to the contractors to get their opinions on matters such as cost, implementation, installation, and future use.

Overall, the contractors of the three bridges that utilized the pipe–tie system saw benefits of the system. They believed that the system performed very well. All three bridges were reported as having no leaking of the concrete from the formwork along the exterior girders. These reported results are an indication that the rotation of the exterior girders was minimal, and the system was able to resist these rotations to assure a uniform deck-slab thickness in the transverse direction. Moreover, the contractors noted quick and easy installation of the system with no issues during construction. The cost of the system's fabrication and materials was not very high; and because the diagonal pipes can be designed to be reusable and adjustable for different girder spacing, the total cost of the system will drop for future applications.

A summary of the survey with the corresponding answers and observations for each monitored bridge that utilized the pipe–tie system is shown in Appendix A. Detailed questions and answers from the contractors are also provided in Appendices B to D.

6.6 FUTURE STUDIES

A future study is recommended to evaluate and refine the rotation-prevention system when inward rotation of the exterior girder occurs. The study would determine circumstances under which inward rotation may occur and provide recommendations to limit inward rotation. Heavily skewed bridges could also be the subject of future studies to evaluate the relation between the skew angle and the rotation of exterior girders during construction. A future study could further evaluate current practice in the construction field, standards, specifications, and details related to the rotation-prevention system. Alternative materials to protective rubber may be tested and assessed in order to maintain the effectiveness of the rotation-prevention system while preventing damage to the girder.

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APPENDIX A: SUMMARY OF THE SURVEY

	Madison Bridge	Ford Bridge	McDonough Bridge
Diagonal pipe section	HSS 2.5x2.5x0.25 square pipe	HSS 2.5 schedule-40 round pipe	HSS 1.5 schedule-40 round pipe
Diagonal pipe fabrication	—Not difficult —A little pricy	—Not difficult —Material is available. —The cost is initial only.	—Not difficult —Used available materials and sections —Small cost
TB system spacing	Between 7.5 ft and 5.48 ft to avoid formwork components	8.3 ft according with IDOT special provision for shallow beams	8 ft to account for the smaller pipe section used
Horizontal tie section	#5 rebar	#5 rebar	#4 rebar
Hanger type	Dayton Superior BH67HD	Dayton Superior C67	Dayton Superior 1/2-inch hanger with 6,000 lb capacity
Hanger–tie connection	Ties were threaded; high strength nuts were utilized for connecting and ensuring tightness	Ties were clamped to coil rods that went through the hanger hole. The ties were tightened using the nut on the clip.	The ties were threaded in each end to be connected with the hanger using a nut and tightened until no sagging was observed.
Finishing (screed) machine	GOMACO C-450. six wheels (1119.7 lb per wheel)	GOMACO machine with 8 wheels	GOMACO machine with 8 wheels of approximately 1,250 lb per wheel
Difficulties	None experienced	None	None
Finishing machine dry run	Possible to recheck the tightening of the ties' connections	Possible to recheck the tightening of the ties' connections. Difficult to check the pipes.	Possible to recheck the tightening of the ties' connections. Difficult to check the pipes.
Overall performance of the system	—Great performance —No concrete leakage was observed.	—Great performance —No concrete leakage was observed. —More cost effective than other systems	—Great performance —No concrete leakage was observed. —Less expensive than other systems —Girders were not damaged.
Ease of installation	Easy to install and remove but slightly heavy	Easy to install and fabricate	Easier to install and control during construction than other systems and no interference with formwork
Recommendations	Utilizing materials and sections to be reused in future projects	Make the pipes uniform and adjustable for reuse in different projects.	No recommendations; the contractor will revise per spec if to be used in future projects. The cost of high-capacity tie-bar hangers is a concern.

APPENDIX B: MADISON BRIDGE SURVEY

1. **What were the sections of the diagonal pipes used?**
   HSS 2.5x2.5x0.25 square section
   a. **How were the pipes' sections chosen?**
      We selected a 2-1/2″ because it was the minimum size allowed in the special provisions. We chose to use 1/4″ wall thickness to ensure we were in excess of the wall thickness of schedule-40 pipe.
   b. **Was the fabrication of the diagonal pipes difficult or costly?**
      —Not difficult, just tedious
      —We fabricated 70 each of these, using mostly newly ordered domestic steel; some of the miscellaneous angles and plates were left over from previous projects.
      i. Materials—$6,623.00
      ii. Misc. fabrication materials—welding supplies/cutting wheels/grinding wheels—about $200.00
      iii. Labor for fabrication—budgeted $6,800.00; actual—closer to $9k
      iv. Labor for installation—budgeted $2,560.00; actual very close to that

2. **What was the spacing of the temporary bracing systems (diagonal pipe–horizontal tie)?**
   Variable—some at 7.5′/some at 5.48′—the variable spacing was to avoid components of our forming system for the deck
   a. **Was the spacing less than 4 x D and why?**
      Yes, it was less. We paved on the overhang forms, so we could not exceed 8.3′. When we did a layout at 8.3′, it conflicted with our forming whalers, diaphragms, and (in some places) deck drains. We prepared a layout that was less than 4xD and also allowed us to avoid each of the conflicts.
   b. **Did you consider placing the finishing machine over the beams (regardless of the beam size) to increase/maximize the bracing system spacing? (Does this approach provide more savings?)**
      It was considered, but we prefer to run on the overhang unless the bridge is too wide, making our paver too heavy, or there is an interior parapet putting rebar in our way. We find that running on the beams makes finishing behind the paver more difficult and can cause deck covering to be slower. In this case, it may have saved us some money to run on the beams; but it may have sacrificed quality, and we didn't like that tradeoff. Now, we own this bracing system, so in future projects we can continue to run on the overhang without increasing costs. We saw it as worth taking a one-time hit on profit to benefit the quality. Also, if we would have run on the beams, we wouldn't have been able to gauge the performance of the bracing system as well as running on the overhang.

3. **What was the size of the horizontal tie (#)?**
   #5 rebar
   a. **If less than #5, why?**
      N/A—rebar size was chosen per IDOT special provision

4. **What was the hanger type used for connecting the horizontal ties to the girders' flanges?**
   Dayton BH67HD ties were utilized.
   a. **How was the horizontal tie connected to the hangers, and how did you make sure that the ties were properly connected to the hangers?**
      Horizontal ties were threaded; high-strength nuts were utilized to make the connection and ensure it was tight.
5. **What finishing machine (brand and model) did you use, and what was the wheel spacing and load per wheel (of the controlling side) of the finishing machine?**
   GOMACO C-450
   Wheel spacing: Wheel 1 @ 0, Wheel 2 @ 36", Wheel 3 @ 72", Wheel 4 @ 113.92", Wheel 5 @ 149.92", and Wheel 6 @ 185.92"
   Load per wheel = 1,119.7 lb per wheel
6. **What were the difficulties that the workers faced during the installation of the temporary bracing system?**
   None experienced
7. **Is it possible to check the beam-rotation bracing while performing a dry run with the finishing machine?**
   I believe it would be possible
   a. **Can the transverse ties be tightened if the dry run shows problems?**
      Yes, they can.
8. **Do you have any suggestions or recommendations regarding the temporary bracing systems?**
   We felt it worked great. It was expensive this time, but it will be expensive only the first time unless you have a very long bridge that requires you to buy more. This was the tightest deck we have ever had. I inspected the underside of the deck during the pour and was unable to find a single spot where the concrete had leaked through onto the beams. That in my opinion means that the beams were not rotating excessively, and the tie/bracing system performed just the way it was supposed to. Our crews liked it, it was easy to install and easy to remove—a little heavy, but they were able to get them in pretty easily. We did utilize materials that would allow us to modify our system into a telescopic system to provide more variation in beam spacing without too much difficulty or cost; this may be something to look into further.

APPENDIX C: FORD BRIDGE SURVEY

1. **What were the sections of the diagonal pipes used?**
   HSS2.5 schedule-40 round section
   a. **How were the pipes' sections chosen?**
      According with IDOT special provision for the temporary bracing system for this project
   b. **Was the fabrication of the diagonal pipes difficult or costly?**
      Finding the material was not difficult through local suppliers. The additional cost of the material and the fabrication was already taken care of in the bill of material of the project. The cost of the pipes could vary; depends on the span (length) of the bridge.
      The fabrication was not difficult. The pipe was adjusted with heavy solid-coil rods slid inside the pipe for ease of adjustment. One end of the pipe was fixed and the other was adjustable in length for ease of installation. The pipes were designed to be slid inside another pipe to be adjustable in length and hence be used in the future for a different project with different girder spacing and depth.
      The pipe cost is initial because the pipes can be used in future projects. Additional cost might be required for other projects, depending on the length; but it would be minimal.

2. **What was the spacing of the temporary bracing systems (diagonal pipe–horizontal tie)?**
   No greater than 8.3 feet. Unless there was a diaphragm, the system was placed next to it if the diaphragm was in the way of the system.
   a. **Was the spacing less than 4 x D and why?**
      Yes, it was less. Based on IDOT special provision for shallow girders.
   b. **Did you consider placing the finishing machine over the beams (regardless of the beam size) to increase/maximize the bracing system spacing? (Does this approach provide more savings?)**
      Finishing machine was placed directly over the exterior girders because the depth of the girders is below 30 inches, W27 beams. (per IDOT special provision)

3. **What was the size of the horizontal tie (#)?**
   #5 rebar
   a. **If less than #5, why?**
      N/A–rebar size was chosen per IDOT special provision.

4. **What was the hanger type used for connecting the horizontal ties to the girders' flanges?** Dayton Superior C67 hangers. Contractors chose to go with hangers with the higher capacity as they were not more expensive than the ones with lower capacity. There were no issues observed in the hangers. **a. How was the horizontal tie connected to the hangers, and how did you make sure that the ties were properly connected to the hangers?** Dayton Superior C67 hangers were hooked to outside edge of the top flange of the beam, with hot-dipped galvanized coil rod running through the hole of the hanger, which was clamped to the #5 rebar. On the outside of the clip, there is an adjustable nut to tighten up the rebar. This will assure that there was no sagging in the tie. There were no issues observed in the hangers or in the ties. The workers tried to vertically align the ties and pipes as close as possible.
5. **What finishing machine (brand and model) did you use, and what was the wheel spacing and load per wheel (of the controlling side) of the finishing machine?** GOMACO finishing machine with 8 wheels (4 wheels) per side, with 21" spacing between the wheels.
6. **What were the difficulties that the workers faced during the installation of the temporary bracing system?** None experienced, but it was easy to install, just some additional time for installation and fabrication. There was also no observed damage to the girders at the location of the temporary bracing system.
7. **Is it possible to check the beam-rotation bracing while performing a dry run with the finishing machine?** Yes, it is possible. **a. Can the transverse ties be tightened if the dry run shows problems?** If sagging of the ties was noticed, yes, the ties could be tightened. However, it will be more difficult for the pipes because of the temporary working surface underneath; and the beams were shallow, which did not provide enough access to the pipes.
8. **Do you have any suggestions or recommendations regarding the temporary bracing systems?** The system worked great, and there was no noticeable leaking of the concrete from the formwork. It would be good to make something (the system) that can be uniform and reusable (adjustable) so the cost of the material will be minimal, and the pipe–tie system can be used in different projects during the deck-slab construction. This system (pipe–tie) would be easier to install and more cost effective than the diagonal ties system that is currently in IDOT specs, especially as the pipes can be designed to be reused in different projects.

APPENDIX D: MCDONOUGH BRIDGE SURVEY

1. **What were the sections of the diagonal pipes used?**
   HSS1.5 schedule-40 round section
   a. **How were the pipes' sections chosen?**
      The contractor used what they had available in the yard, which makes it difficult to estimate the total cost of the material or the fabrication, but as the used material was available, the cost was small. It would have been more expensive if the contractor had to buy all material needed for the system.
   b. **Was the fabrication of the diagonal pipes difficult or costly?**
      The fabrication was not difficult. The pipe section utilized also 3/4-in. bolt with a rebar welded at 90° on the end of the bolt. The rebar was covered with a piece of rubber hosing that was tightened well to the rebar in order to protect the galvanization of the steel girders at the points of contact. The galvanizing was not damaged at all after the pouring process. The hardest part in the pipe's fabrication process was threading the pipes to place the adjustable ends on it. The cost also was very minimal.

2. **What was the spacing of the temporary bracing systems (diagonal pipe–horizontal tie)?**
   No greater than 8 feet.
   a. **Was the spacing less than 4 x D and why?**
      Yes, the contractor used the low value specified in the special provision because of the use of a lower pipe section than the one specified. IDOT bridge office and the contractor coordinated these changes. There were no problems with spacing them at 8 feet, and the ties were aligned with diagonal pipes.
   b. **Did you consider placing the finishing machine over the beams (regardless of the beam size) to increase/maximize the bracing system spacing? (Does this approach provide more savings?)**
      Finishing machine was placed on the overhang as allowed per the IDOT specs for girders more than 30 inches in depth. W36 girders were utilized in this bridge.

3. **What was the size of the horizontal tie (#)?**
   #4 rebar
   a. **If less than #5, why?**
      When the contractor contacted the supplier (Dayton Superior) to ask for hangers that fit in #5 rebar, the supplier did not have the 5/8″ (9,000 lb) hangers available in their inventory. The 5/8″ hangers had a cost of $102 each. Therefore, the supplier needed to make the hangers for the #5 bars. Hence, the price of the hangers was almost 6 times higher than the ones for #4 bars. The contractor chose to use #4 bars and the 1/2″ (6,000 lb) hangers by the supplier to lower the cost of the system after coordinating with IDOT bridge office. The 1/2″ hangers had a cost of $16 each.

4. **What was the hanger type used for connecting the horizontal ties to the girders' flanges?**
   Dayton Superior 1/2-inch hangers with 6,000-lb capacity
   a. **How was the horizontal tie connected to the hangers, and how did you make sure that the ties were properly connected to the hangers?**
      Tightening the ties to the hangers was easy and more efficient than with previous systems used; especially the access to the ties was easier. This helped in making the process quicker and reduced or eliminated any expected sag in the ties. There was no observed loosening or break in ties or in the hangers. The bars were longer than the hangers' spacing with the 6" ends of the bars threaded; then they were tightened to the hangers. The pipes were placed very tight to the girders, but rechecking the pipes for tightness is more difficult than for the ties due to the need for a man lift to access the pipes below the deck formwork.

5. **What finishing machine (brand and model) did you use, and what was the wheel spacing and load per wheel (of the controlling side) of the finishing machine?**
   GOMACO, 2 legs with 2 wheels at 21 in. center to center with roughly 1,250 lb per wheel, and the legs are 85.5 in. apart.

6. **What were the difficulties that the workers faced during the installation of the temporary bracing system?**
   Not experienced, but it was easy and quick to install; and there was no interference with the formwork of the deck slab. There was also no observed damage to the girders at the location of the temporary bracing system.

7. **Is it possible to check the beam-rotation bracing while performing a dry run with the finishing machine?**
   Yes, it is possible.
   a. **Can the transverse ties be tightened if the dry run shows problems?**
      Yes, it was easy to tighten the ties, so if any sagging or loosening was noticed, it was taken care of.

8. **Do you have any suggestions or recommendations regarding the temporary bracing systems?**
   The system was fast and easier to install, and it was less expensive than other systems used in the past. The contractor is storing the pipes systems to be used in the future for other projects. However, the contractor will work on adjusting the pipe to be adjustable in length at the center of the pipe, which will make them easy to install regardless of the girder spacing. The contractor will refabricate the pipes according with IDOT specs to be 2.5 in. in diameter; and they think it will be worth investing the money in it, as they can be used in future bridge deck pouring. Contractor liked the system and would like to see it implemented. The system might cost the contractors a little bit more money at the beginning, but it will be very economical long-term.