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EVALUATION OF EXPERIMENTAL BRIDGES
IN TENNESSEE

A Dissertation
Presented for the
Doctor of Philosophy
Degree
The University of Tennessee, Knoxville

David P. Chapman

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DEDICATION

This dissertation is dedicated to my parents, Mike and Nancy Chapman.
Their unwavering support and encouragement has truly made this work possible.

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ABSTRACT

This dissertation presents an overview and evaluation of an effort of the Tennessee Department of Transportation (TDOT) to design multispan steel girder bridges that can be erected with minimal disruption to traffic passing under the proposed spans. TDOT has developed the Pier-Plate Moment Connection in order to facilitate the accelerated construction of steel girder bridges. This connection allows the girders of a multispan steel bridge to be erected as simple spans and then to be made continuous prior to the dead load of the deck being applied. Since 2003, the Department of Civil and Environmental Engineering at the University of Tennessee has instrumented and tested two bridges with Pier-Plate connections. The bridges were instrumented with strain gages and monitored under a variety of loading conditions.

The primary conclusions drawn from the above described research are (1) that the code specified methods for determining girder distribution factors produce conservative values, (2) that the structural contribution of concrete parapets is to stiffen the outside girders and thereby draw more load to them, and (3) that the behavior of the Pier-Plate Connection is consistent with the intent of its design.

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NOMENCLATURE

G_{int}	Distribution Factor for Interior Beams
S	Spacing of Beams (ft), Such That $3.5ft \leq s \leq 16ft$
L	Span Length of a Girder (ft), such that $20ft \leq L \leq 240ft$
t_s	Depth of Concrete Slab (in), such that $4.5in \leq t_s \leq 12in$
K_g	Longitudinal Stiffness Parameter (in^4), such that $10,000 \leq K_g \leq 7,000,000$
E_B	Modulus of Elasticity of Beam Material (ksi)
E_D	Modulus of Elasticity of Deck Material (ksi)
I	Moment of Inertia (in^4)
A	Area of the Beam (in^2)
e_g	Distance Between the Centers of the Basic Beam and Deck (in)
ε	Strain at a Point (in/in)
E	Modulus of Elasticity of steel with weldable strain gages (ksi)
C	Distance from the Center of Gravity of the Member

ABBREVIATIONS

TDOT	The Tennessee Department Of Transportation
UT	The University Of Tennessee at Knoxville, TN
AASHTO	American Association of State Highway and Transportation Officials
FEA	Finite Element Analysis
NCHRP	National Cooperative Highway Research Program
GDF	Girder Distribution Factor
LRFD	Load and Resistance Factor Design

CHAPTER 1:

INTRODUCTION AND GENERAL INFORMATION

Introduction

The costs of providing roads and bridges to the traveling public can be divided into two categories: construction cost and user cost. User or "soft" costs include added vehicle operating costs and delay costs to highway users resulting from construction or maintenance activity. Designers should attempt to reduce the user cost associated with any project, and in Tennessee one approach has been to develop construction systems that are designed to minimize disruptions to the traveling public. One such system is the Pier-Plate moment connection. This connection allows a multispan steel bridge to be erected as simple spans and then to be made continuous prior to the dead load of the deck being applied.

Since 2003, the Department of Civil and Environmental Engineering at the University of Tennessee has instrumented, tested, and evaluated two bridges with Pier-Plate connections. The DuPont Access Bridge is located in New Johnsonville, TN and spans over US Highway 70. The bridge primarily carries truck traffic to and from a large industrial facility. The bridge was instrumented with 42 strain gages, and data were collected during the deck pour and during controlled load tests after the bridge was completed. The Massman Dr. Bridge is located in Nashville, TN and spans over Interstate 40. The bridge was instrumented with 84 strain gages, and data were collected during the deck pour. Controlled load tests were conducted both before and after the parapet was poured.

Since 2003, Department of Civil and Environmental Engineering at the University of Tennessee has entered into three contracts with the Tennessee Department of Transportation (TDOT). The first contract was to instrument and test the DuPont Access Bridge, the second contract was to instrument and test the Massman Drive Bridge, and the third contract was to further reduce the vast

amount of data collected during contracts one and two. This Dissertation is a product of the body of work produced during the three contracts.

General Information

The body of this document consists of four manuscripts that are in various stages of publication (Chapters 3 through 6). The manuscripts have been edited to provide continuity within the chapter structure of this dissertation and to make reference to a combined set of tables and figures. All tables and figures appear in the appendix at the end of this document. Chapter 2 of this dissertation contains a literature review for the entire document, and is loosely organized by subtopics corresponding to individual chapters.

CHAPTER 2:

REVIEW OF LITERATURE

Development of Experimental Bridge Types

The Structures Division of TDOT is responsible for designing most of the bridges constructed on Interstates and State Routes in Tennessee. The ever increasing amount of traffic on Tennessee's roads has necessitated that TDOT develop construction systems that allow for accelerated construction of bridges. In Wasserman (2005), Mr. Wasserman details the development of two bridge types: (1) simple span for the deck dead load / continuous span for all other loads and (2) simple span for beam dead load / continuous for all other loads. Concrete bridges of the first type, described by their structural behavior during incremental stages of construction as simple span, non-composite for all dead loads / continuous span composite for live loads, exists widely and have prestressed concrete girders. TDOT has attempted to replicate this system using steel girders. Bridges of the second type, simple span for beam dead load / continuous for all other loads, are made possible thru the use of the Pier-Plate moment connection which is evaluated in this document.

Lateral Load Distribution

The lateral load distribution has a direct effect on the strength, economy, and serviceability of highway bridges. Many researchers have studied load distribution factors through full-scale testing and/or finite element analysis. Full-scale testing is a true evaluation of behavior because it includes all the parameters that affect the behavior of a particular bridge. Finite element models must be created carefully in order to model the bridge parameters accurately. Finite element analyses frequently produce unreliable results unless the finite element models are accurately calibrated.

NCHRP (National Cooperative Highway Research Program) Project 12-26 (Zokaie, 1992) reported an extensive study on "Distribution of Wheel Loads on

Highway Bridges." The study began in the mid 1980's and suggested that the specifications regarding load distribution should be updated to allow for more accurate calculation of loading effects on highway bridges. The study occurred in two phases with three levels of analysis for each bridge type. Level one of the analysis consists of using simple formulas. Level two uses simple computer methods, and level three uses detailed finite element models. The study provides guidelines and formulas for different methods of calculating load distribution factors.

Several studies have been conducted to determine appropriate load distribution factors. The 17th Edition of the Standard Specifications for Highway Bridges (2002) determined girder distribution factors based solely on girder spacing and bridge type, while the 4th Edition of the LRFD Bridge Design Specification (2007) takes into account more bridge parameters such as slab thickness, span length, and girder stiffness. Zokaie (2000) describes the development of the AASHTO 1998 LRFD code (similar to the AASHTO 2007 LRFD) and discusses its accuracy. Zokaie found that the newly developed formulas generally produced results that were within five percent of the results produced from finite element analysis.

Fu, Elhelbaway, Sahin, and Schelling (1996) conducted a study using field data to determine the effect of live load distribution for slab-and-beam bridges under the effect of a moving truck by using strain data to get moment fractions. They found the distribution factors for four different bridges to be within the limits set forth by other methods and codes. Kim and Nowak (1997) discuss the procedure and results of field tests that were performed on steel I-girder bridges to determine distribution factors. They too used strain data and concluded that their results were within the limits established by AASHTO 1994 LRFD values. The methodologies used in previous experimental work are comparable to the techniques used for analyzing the DuPont Access and Massman Drive Bridges for the Tennessee Department of Transportation (reported in this dissertation).

The Contribution of Secondary Structural Elements

Full scale testing of bridges suggests that the use of code analysis techniques does not accurately predict the structural behavior of highway bridges (Burdette and Goodpasture 1973; Buckle et al. 1985; Bakht and Jaeger 1992). Significant discrepancies in predicted and measured behavior, particularly that of lateral load distribution, have been reported. A likely source of this error in the results of code analysis techniques is the neglecting of the structural contribution of secondary elements such as parapets, sidewalks, and median barriers. The following studies report on various aspects of the structural contribution of secondary elements.

In Billings (1984) parapets were found to provide substantial stiffness under service loads. Billings used displacement transducers to measure the response of 27 highway bridges in Southern Ontario. The Baxter Creek Bridge, one of the 27 instrumented bridges, has a superstructure consisting of 6 simply supported AASHTO Type 2 girders and discontinuous Jersey type parapet. The girder deflections measured at the Baxter Creek Bridge were compared with that of a three span continuous bridge of similar cross section, except that no parapet was present. Billings suggest that the exterior girder of the Baxter Creek experienced significantly less deflection because of the presence of the parapet.

Smith and Mikelstiens (1988) conducted a grillage analysis of an idealized simple span bridge. The span length, girder stiffness, girder type, and type and orientation of secondary elements were varied for the purpose of determining the lateral load distribution characteristics of the idealized bridge. Smith and Mikelstiens found that, for all bridge decks studied, the presence of secondary elements significantly affected the load distribution by stiffening the outside girders and that edge stiffening secondary elements had the greatest effect on short span bridges. Smith and Mikelstiens is an early example from a collection of studies that have reported the structural contribution of secondary elements

based on analytical methods. The following studies report the contribution of secondary elements based on results from various analytical analyses.

Mabsout et al. (1997) reports the influence of sidewalks and railings on wheel load distribution in steel girder bridges based on Finite Element Analysis (FEA). A parametric study of 120 bridges was conducted by varying the span length, girder spacing, the presence and cross section of the sidewalks, and the presence and cross section of the railings (a.k.a parapets). It was found that, when included in the strength analysis of a bridge, the sidewalks and railings could increase the load carrying capacity by as much as 30%, and suggest that the NCHRP 12-26 formulas (predecessor to the AASHTO LRFD load distribution equations) are conservative as they do not account for secondary elements.

Eamon and Nowak (2002) report the effects of edge-stiffening elements and diaphragms on bridge resistance and load distribution based on FEA. The study found that, based on a parametric study of some 240 idealized bridges, in the elastic range the presence of secondary elements reduced the GDF to interior girders by 10% to 40% and that, in the inelastic range, presence of secondary elements reduced the GDF by 15% to 60%.

In Brenner et al. (2005) the strength of a typical highway bridge was analyzed with parapets. The study was conducted by modeling a single span of the Neponset River Bridge in Quincy, Massachusetts using FEA techniques. The Neponset River Bridge is a typical slab-on-girder type bridge with steel girders and a composite deck, and was modeled with and without parapets. The main conclusion drawn from the study was that the parapets stiffened the overall cross section of Neponset River Bridge (which has a significantly stiffer deck than the bridges considered in other studies).

Chung et al. (2006) reports the influence of secondary elements and deck cracking on the lateral load distribution of steel girder bridges in the State of Indiana. The study investigates 9 slab-on-girder type bridges using FEA by considering the presence of parapets and diaphragms. The study found that

the consideration of secondary elements produced a GDF up to 39% less than that of AASHTO LRFD.

Eamon and Nowak (2003), Conner and Huo (2006), and Akinci et al. (2008) all report on some aspect of the structural contribution of secondary elements based on FEA, indeed many studies have attempted to ascertain the structural contribution of secondary elements by analytical methods. Some conclusions common to all of the previously stated analytical studies are that the consideration of secondary elements produces lower GDFs and the fact that parapets and other secondary elements are not considered in design results in conservative GDFs.

The analysis of data collected during full scale testing is notably missing from the body of knowledge related to the structural contribution of secondary elements. Billings (1984) is the only study reviewed in this document to base any findings related to secondary structural elements on data from full scale testing, but his findings are based on the comparison of data from two significantly different bridges. Chapter 6 of this document reports the structural contribution of the parapets of the Massman Drive Bridge based on full scale testing.

CHAPTER 3:

ACCELERATED CONSTRUCTION OF STEEL BRIDGES IN TENNESSEE

Contribution of the Author

The authors of this chapter are David Chapman, J. Harold Deatherage, Edwin Burdette, and David Goodpasture. The contributions of Mr. Chapman to this chapter are as follows: serving as coordinator of all work related to the Massman Drive Bridge, assisting in the mobilization and demobilization of resources to and from Nashville, TN and Van Buren, AR, assisting in the installation of the gages on the Massman Bridge, assisting in the administration of all tests conducted on the Massman Drive Bridge, writing the chapter, seeing to the presentation of the chapter at the Accelerated Bridge Construction – Highways for Life Conference of 2008, and seeing to the publication of the chapter in proceedings of the previously stated conference.

Introduction

The costs of providing roads and bridges to the traveling public can be divided into two categories: construction cost and user cost. User or "soft" costs include added vehicle operating costs and delay costs to highway users resulting from construction or maintenance activity. Designers should attempt to reduce the user cost associated with any project, and in Tennessee one approach has been to develop construction systems that are designed to minimize disruptions to the traveling public. One such system is the pier-plate moment connection. This connection allows a multispan steel bridge to be erected as simple spans and then to be made continuous prior to the dead load of the deck being applied.

Since 2003, the department of civil and environmental engineering at the University of Tennessee has instrumented, tested, and evaluated two bridges with pier-plate connections. The DuPont access bridge is located in New Johnsonville, TN and spans over U.S. Highway 70. The bridge primarily carries

truck traffic to and from a large industrial facility. The bridge was instrumented with 42 strain gages, and data were collected during the deck pour and during controlled load tests after the bridge was completed. The Massman Dr. Bridge is located in Nashville, TN and spans over interstate 40. The bridge was instrumented with 84 strain gages and data were collected during the deck pour, controlled load tests were conducted both before and after the parapet was poured.

Multispan steel bridges exist widely and, if designed to be continuous under the dead load, typically have splices at the inflection points. A potential disadvantage of this type of structural system is that shoring towers or multiple cranes must be employed to stabilize the structure during erection. In the case of a two span girder bridge that consists of three pieces, the first piece to be erected would need to be supported by a crane or shoring tower while the second piece is erected and spliced to the first, and bearing at the abutment and the pier is achieved. The structure cannot be considered stable without outside support until at least four of the six pieces of two girders have been erected and the lateral braces have been installed. In the case of constructing this type of bridge over active traffic, the contractor would need to close traffic for the period of time required to erect the four pieces of two girders and the cross frames. This period of time is widely variable and a function of many conditions, but would typically range from 15 to 30 minutes. In the case of a two span girder that consists of two pieces and a pier-plate connection, two pieces of two girders and the lateral braces between them would need to be erected before the structure would be stable, and traffic could resume passing underneath. The pier-plate connection alleviates the need for splices at the inflection points, and greatly reduces the period of time that traffic under the proposed structure would need to be stopped. The period of time that traffic was stopped for one cycle of erecting two girders and installing three cross frames for the Massman Drive Bridge was approximately six minutes. The construction details of the DuPont access and Massman drive bridges are similar to that of bridges with prestressed concrete girders in that the girders are

erected as simple span and behave as simple spans under the dead load of the girder. If the DuPont access bridge had been constructed with AASHTO type 3 girders instead of rolled steel girders, the contractor would have needed to employ a significantly larger crane. Type 3 girders weigh approximately 580 lbs per foot, considerably more than the W 30x241s used at the DuPont access bridge. Similarly, had the Massman drive bridge been constructed with 72" deep bulb tee girders, they would have weighted approximately 800 lbs per foot as opposed to the plate girders weighing 311 lbs per ft that were used. The use of a lighter girder also reduces the cost of shipping the girders to the jobsite.

Description of the DuPont Access and Massman Dr Bridges

The DuPont access bridge has two composite spans and is supported by integral abutments. The bridge's foundation consists of steel piles which support both the abutments and the three pile caps for the three columns at the center pier. All piles are HP 10x42's. The girders of the DuPont access bridge are W33x240s (Grade 50, weathering steel) spaced at 7ft. - 4 in. on center. The girders are braced against lateral torsional buckling by channels (C15x33.9) bolted to web stiffeners which are in turn welded and bolted to the girders. Cross-section and elevation views of the bridge are shown in Figure 1 (All Table and Figures are shown in the Appendix). At the pier the north and south girders are connected at the top flange by a 1 5/8 in. thick cover plate that is 11ft. - 3 1/2 in. long with 40 bolts into the top flange of both girders. The compression forces at the pier are transferred between girders by a 1 7/8 in. thick wedge kicker plate that is two inches wider than the bottom cover plate and bears against the inside of the bottom flange. After bearing is achieved the wedge is welded to the girders. A one foot thick reinforced concrete diaphragm exists at the pier. The north girders have shear studs on 6 in. centers for the first 8 ft. of the span and on 10 in. centers for the next 47 ft. - 6 in. of the span measured from the centerline of bearing at the abutment and towards the pier. The south girder has shear studs on 6 in. centers for the first 8 ft. of the span and on 10 in. centers for the next 47 ft. - 6 in. of the span measured from the centerline of the

abutment towards the pier. The deck is 8 ¼ in thick and acts compositely with the girders. The DuPont Access Bridge has a 2% slope laterally in both directions from the center line of the deck, and has a jersey type concrete parapet rail on both sides. The DuPont Access Bridge is not skewed.

The substructure of the Massman Drive Bridge consists of foundations on rock and foundations on piles. All piles are HP 10 x 42's. The abutments are integral abutments supported by piles. At the pier, two existing columns from the old bridge, which was demolished, were used and two new columns were added. The bearing for the girders consists of riser blocks at the pier and at the abutment; also neoprene bearing pads are present at the pier and at the abutment. The superstructure of the Massman Drive Bridge consists of 5 steel plate girders and a composite deck. The girders are approximately 64" tall and are spaced 9'6" on center, and were fabricated using 50 ksi weathering steel. The span lengths for the north and south spans are 138.5 ft and 145.5 ft respectively. After the girders were erected and the metal decking had been installed, the shear studs were installed. The contractor elected to install the shear studs on site for safety reasons. The Massman Drive Bridge has intermediate bracing between the girders spaced at 25' on center over the entire length of the bridge. The bracing consists of angles bolted to web stiffeners. Lateral bracing at the abutment was added at the request of the contractor. At the pier each girder was braced with a single C 15 x 33.9 channel that is bolted to the web stiffener at the end of the girders. A 1'-0" thick concrete diaphragm was poured monolithically with the deck between the girders at the pier. The pier-plate connection consists of cover plates 2" thick, 1'-6" wide and 10'-6" long, and wedge plates 2" thick and 1'-11.5" long with varying width. A ½" thick, 1'-7 ½" wide, and 4'-9" long bottom cover plate was welded to the bottom of the bottom flange of each girder in the vicinity of the pier to make up the difference in the thickness of the bottom flange and the wedge plates, and to increase the cross section. The cover plates connect the top flange of each girder with 18 lines of 4 A325 bolts. In the case of the Massman Drive Bridge, the bolt holes were only drilled in the top flange on one of

the girders at the fabrication plant. After the second girder was erected the cover plate was used as a template to field drill the holes in the top flange of the opposing girder. The deck of the Massman Drive Bridge is 8 ¼ in. thick, and acts compositely with the girders. The deck has a 2% slope laterally in both directions from the center line, and has a jersey type concrete parapet rail on both sides. The Massman Dr. Bridge is not skewed. Figure 2 is an elevation and cross section of the Massman drive bridge.

Design Concepts

The Massman drive bridge was designed by the structures division of the TDOT. The information presented in this section was gathered during extensive interviews with structures division engineers.

The Massman drive bridge was designed in accordance with the AASHTO standard specifications for highway bridges (2002 edition), and the following is a list of key concepts that set the Massman drive bridge apart from a typical multi-span steel plate girder bridge:

- 1) The plate girders have a constant cross section. Typically, TDOT designs plate girders with a web of constant depth and thickness, and with flanges of varying thicknesses and widths.
- 2) The use of a pier-plate connection allows the girders to be erected as simple spans, and made continuous for the dead load. The girders were designed to carry their self weight in a simple span condition, to carry the dead load of the deck in a continuous condition, and to carry the live load in a continuous composite condition.
- 3) The pier-plate connection utilizes a cover plate in tension connecting the top flanges; this, along with wedge plates in compression connecting the bottom flanges, allow the girders to develop negative moment capacity at the pier.

- 4) The cover plate is bolted to the top flange of the girder. The connection was designed to use the number of bolts required to develop the full tensile capacity of the cover plate, but the number of bolts was later increased to meet the stitching or sealing requirements of AASHTO.
- 5) In the case of catastrophic loading, the pier-plate connection adds an extra measure of redundancy to the structure.

In some cases the use of a pier-plate connection can offer design efficiency because the girder can be designed for a lower maximum moment. The maximum moment experienced by a 2 span, uniformly loaded, continuous girder is the negative moment at the middle reaction. When a pier-plate connection is used, the presence of top and bottom cover plates in the region of maximum moment increases the moment of inertia sufficiently, from that of the girder alone, to accommodate the maximum moment. The increase in moment of inertia in the region of maximum negative moment allows the girders to be designed for the maximum positive moment.

Structural Behavior

Strain gage data collected during the placement of the concrete deck were analyzed in an effort to determine the presence of continuity created by the pier-plate connection at both the DuPont access and the Massman drive bridges. Conclusions were drawn regarding the performance of the connection by comparing the measured behavior to results predicted by computer models.

The DuPont Access Bridge

Data collection at the DuPont access bridge began at the start of the placement operation and continued, uninterrupted, for 4:45 min. Data collection ended approximately five minutes after completion of the deck pour. The deck was placed from the south end of the bridge to the north end of the bridge. Data taken during the last 2 minutes of data collection, after the entire deck had been placed, was used to determine the performance of the

connection. Data taken from cross sections IA 34 and p3 are presented in Figure 3. Cross sections IA 34 and p3 are named IA 34 and p3 because they are approximately located 34 ft from the integral abutment and 3ft from the pier, respectively

Figure 3 is a moment diagram for girder g. The x axis represents the distance along the bridge measured from north to south from the centerline of bearing at the pier to the centerline of bearing at the abutment, and the y axis represents the moment in kip-ft. Three different model outputs are plotted on the diagram: pinned reactions with no continuity (BC1), pinned reactions with full continuity (BC2), and pinned reactions at the abutment with full continuity and rotational springs (BC3), as well as the measured moments. Typically, in Tennessee when a bridge with integral abutments is constructed, the backwall of the abutment will be poured monolithically with the deck. At the DuPont access bridge the backwall of the abutments were poured, up the top of the pavement shelf, two weeks prior to the deck pour, thereby developing some measure of resistance to rotation at the abutments. This resistance to rotation was incorporated into the structural model by adding a rotational spring to the reactions at the abutments. The measured moments are reported from gage locations P3 and IA34, and are labeled as such.

Figure 3 shows that measured results are most accurately predicted by BC3, which is the model that incorporates continuity at the center pier and a rotational spring at the integral abutments. By comparing measured results to those obtained from BC3, it is apparent that the load used in the models may have been slightly overestimated. The model is appropriately shaped to the measured data, but over-predicts moment magnitude at locations P3 and IA34, leading to the conclusion that the load was overestimated. Boundary condition set BC2 also over predicts moment magnitudes at locations P3 and IA34. The magnitude of BC2's over-prediction is greater than that of BC3. The measured data at location IA34 is the most reliable of all of the locations due to the absence of any added moment of inertia due to the rolled shape and the

measurement of a complete strain profile through the depth of the section. Therefore, the large over-prediction of moment at location IA34 indicates that BC2 is not an accurate representation of the behavior of the girders.

Boundary condition set BC1 does not appear to accurately represent measured data anywhere along the length of the girders. Therefore, the assumption that the bridge may behave as two simply-supported spans is dismissed.

The Massman Drive Bridge

Data collection at the Massman drive bridge began at the start of the placement operation and continued uninterrupted for 31 hours, ending approximately ten minutes after completion of the deck pour. The duration of data collection at the Massman drive bridge was considerably longer than that of the DuPont access bridge because the deck was placed in a sequence over a period of two days. The positive moment region of the south span was poured 1st (morning of day 1), the positive moment region of the north span was poured 2nd (afternoon of day 1), the negative moment region was poured 3rd (morning of day 2), and the abutment back walls were poured 4th (afternoon of day 2). Data taken during the two minutes following the completion of the placement of the positive moment region of the south span were used to determine the performance of the connection. These data were chosen because the cross sectional properties of the girders, loaded with fluid concrete only, could be clearly defined. Additionally, data from interior girders only were used to isolate the load from the concrete screed used during the deck placement.

Figure 4 is similar to Figure 3 in that moment diagrams are plotted with measured data and data from model results. Two different model outputs are plotted on the diagrams: pinned reactions with no continuity (BC1) and pinned reactions with full continuity (BC4), as well as the measured moments. The model outputs labeled BC 4 and BC 1 are for a girder loaded in the positive

moment region consistent with the load on the bridge after the 1st pour with and without continuity over the center pier.

A comparison of measured results to those obtained from BC 4 in Figure 4 shows that the model accurately predicts the measured results. Furthermore, the presence of a significant negative moment at location P 5.5 suggests that the cover and wedge plates are behaving as designed.

Boundary condition set BC1 does not appear to accurately represent measured data for girders 4 or 5. Therefore, the assumption that the bridge may behave as two simply-supported spans is dismissed.

Conclusions

This chapter has presented an overview of one of the methods used by TDOT to accelerate the construction multispan steel girder bridges. The development the pier-plate moment connection represents a significant innovation with respect to the construction of multispan steel bridges, because it allows for decreased construction time and therefore lower user cost.

The pier-plate connection has been found to behave as designed based on a comparison of measured results and results generated by structural models.

CHAPTER 4:

EVALUATION OF THE DUPONT ACCESS BRIDGE

Contribution of the Author

The authors of this chapter are David Chapman, J. Harold Deatherage, Edwin Burdette, and David Goodpasture. The contributions of Mr. Chapman to this chapter are as follows: assisting in the mobilization and demobilization of resources to and from New Johnsonville, TN, assisting in the installation of the gages on the DuPont Access Bridge, assisting in the administration of the controlled load tests, writing the chapter, and seeing to the publication of the chapter in the Journal of Experimental Techniques.

Introduction

A significant portion of the cost of newly constructed roads is the user cost, and state departments of transportation are sensitive to issues related to user cost. The innovative design of the DuPont Access Bridge is an attempt to allow the bridge to be constructed faster, thus reducing the user cost. The subject of this paper is the testing and evaluation of the first steel bridge in the state of Tennessee to be designed for continuous action under the dead load while being erected as simple spans. The DuPont Access Bridge was investigated for three main reasons: to assure continuity for dead loads, to assure continuous composite behavior for live loads, and to compare the measured GDFs (girder distribution factors) to other GDFs calculated from commonly accepted methods. This paper focuses on (a) the instrumentation used to measure the response of the bridge and (b) the adequacy of the connection design to assure continuity under the weight of the newly poured concrete deck. Figure 5 is a plan view of the DuPont Access Bridge.

Description of Bridge

The DuPont Access Bridge is a two span slab-on-girder type bridge with integral abutments. The bridge's foundation consists of steel piles which support

both the integral abutments and the three columns at the center pier. The girders of the DuPont access bridge are W33x240s spaced at 7'-4 13/16" (2.26m) on center as shown in a typical cross section in Figure 1. The girders are braced against lateral torsional buckling under the dead load by channel stiffeners. At the pier, the north and south girders are connected at the top flange by a 1 5/8" (41 mm) thick cover plate that is

11'-3 1/2" (3.44m) long with 40 bolts into the top flange of each girder. The girders are connected at the bottom flange by two wedge plates that bear against the bottom flanges at the ends of the girders. After bearing is achieved, the wedge plates are welded together and to the girders. The deck is 8 1/4" (210 mm) thick, and acts compositely with the girders.

Instrumentation

The general design assumption was that the connection would allow the girders to be erected as simple spans, and behave as two continuous spans under the dead load. To determine whether the connection behavior was consistent with the design assumptions, a test was planned to collect strain data at various longitudinal locations during the deck pour. For the remainder of this text, this will be referred to as the connection test. The general design assumptions were that the connection at the center pier of the DuPont Access Bridge allowed the bridge to behave as two continuous spans under the dead load and two continuous composite spans under the live load.

For all tests conducted on the DuPont Access Bridge, the data collection hardware was located in an office trailer placed just west of the south abutment. The strain gages used in the testing of the DuPont access bridge were model number HBW-35-500-6-20VR weldable strain gages manufactured by Hitec Products. The weldable strain gages were coated in rubber and attached to a thin piece of stainless steel which permitted the gages to be spot welded to the girder. The wires connecting the gages to the Megadac were temporarily fixed to the girders by C-Clamps that were installed on the bottom flanges of the

instrumented girders, and were contained in a conduit for the portion from the bridge to the trailer. The Megadac is a data acquisition system that is capable of capturing data at up to 25,000 readings per second, and is commonly used in the fields of automotive, aerospace, and structural testing. For the connection test the Megadac was configured to capture data from approximately 75 channels at a rate of 2 readings per second, and was controlled by a host computer (laptop) via a RS-232 type interface. The interface consisted of a control module installed on the face of the Megadac that allowed for a 25 pin serial connection. The Megadac was grounded by way of a copper rod driven into the ground adjacent to the trailer. The software used to administer a test is known as TCS (Version 3.4.0). TCS defines the test parameters, runs the test, and formats the data.

The DuPont access bridge has six girders. Each girder was identified by a letter, beginning with "E" (for exterior). The girders were labeled from west to east, E being the first, F being the second, and so on. Girders E, F, and G had strain gages located at several cross sections along their length. Each gage was identified by a number, and each number corresponded to a specific location on a beam. Gages 0 were the gages that were located just north of the pier on the bottom flange of each girder. Gage E0 is the gage at position zero on girder E. This system of letters and numbers was used to identify all gages (see Figure 6, beam cross section, and Figure 7 for longitudinal gage location). Gages 0, 2, 3, 7, and 11 are the gages on the bottom flange at the 5 gages cross sections along the bridge. Unfortunately, due to the progress of construction at the time the gages were placed, no gages were installed directly on the connection plate itself, and metal deck panels had to be temporarily removed to access certain points for gage installation.

Data Collection During Deck Pour

Before the deck pour was initiated, the gages were "zeroed" so that only strains from the concrete deck and the construction loads were recorded. The construction loads consisted mainly of the screed and the laborers who were

pouring the deck. During the connection test, a significant amount of noise was observed in the data and it was initially believed that as many as 15 gages were defective or installed improperly. A plan was initiated to replace the gages. During preparations for the controlled load test it was discovered that only three gages were deficient, and that the noise was most likely due to the vibrators used to consolidate the deck concrete. An articulating boom man-lift provided access to the three gages that were replaced. A partial lane closure was provided by TDOT in order to facilitate access to the gages. It was also discovered that one of the gages had faulty wiring, and it was replaced. The rest of the noise was attributed to the vibrators used to consolidate the concrete during the deck pour.

The data used to analyze the connection, as previously stated, were the data taken during the deck pour. After the deck pour had been completed, the only load on the bridge was the fluid concrete. The strain readings from gages 7 through 10 for the last 2 minutes of the test were used to determine the performance of the connection. Gages 7 through 10 were chosen because they exhibited only a small amount of noise at the end of the test.

All 240 readings that were taken at a specific gage for the 2 minute interval were averaged and taken as the maximum value for that gage for that interval. This was done to obtain the average maximum value for strain at a given gage. Based on these strain values for each gage, plots of strain versus depth (see Figure 8) were created to identify and eliminate erroneous readings. No erroneous readings were found in the data from gages 7 through 10 in the last 2 minutes of the test.

For the purpose of comparing the measured results with model outputs, the strain values were converted to moments by assuming the strain at a given cross-section to be the average of the strain at the top and bottom of the girder. The effective modulus of elasticity, based on tensile tests with the weldable gages used, was assumed to be 32,000 Ksi.

Bridge Model

The first step in developing a model of the DuPont Access Bridge was to determine the load on the girders during the deck pour. The load consists of the weight of the fluid concrete being placed. For calculating the weight of the concrete deck the average thickness of the deck was taken as 9.25 inches (23.5cm). The thickness of the deck was shown on the plans as 8.25 inches (21cm), but this did not account for the concrete filling the corrugations in the metal decking. The depth of the corrugations was 2 inches. This depth was present over approximately half of the area of the deck, so the average depth of the deck was taken as 9.25 inches (23.5cm). The tributary width of a girder was assumed to be the spacing of the girders except for the fascia girder. For the fascia girder the tributary width was assumed to be half the spacing plus the width of the overhang which is 2.5 feet (.76m). The density of the fluid concrete was assumed to be 150 lb/ft³ (2403 kg/m³) which gives an average load of 116 lb/ft² (5.55kpa) for the entire deck. These assumptions resulted in a uniform load of 856 pounds per foot of span (1273 kg per meter of span) on the interior girders.

The second step in modeling the DuPont Access Bridge was to define a structural model. The model was analyzed to generate results that were compared to the measured data to estimate the amount of continuity present in the connection at the pier. The structural model considered only one girder, and was idealized in Visual Analysis. The boundary conditions that defined the behavior of the bridge in the model were varied, starting with the reactions pinned and no continuity over the pier, and concluding with fixed reactions at the abutments and full continuity over the pier.

The bending moment 34 feet (10.36m) from the south abutment, measured at the end of the deck pour, was compared to the model output for a similar loading condition. The model had a node 34 ft (10.36m) from the south abutment so that the bending moment could be compared directly.

The moment in girder G 34 ft (10.36m) from the abutment was calculated from field data to be 287.6 kip-feet (389.9kn-m). Figure 9 is a plot of bending moment verses longitudinal location on the bridge. In the positive moment region the upper bound represents the model results for a pinned end boundary condition, and the lower bound represents the model results for a fixed end condition. The single point, plotted as a circle, is the moment calculated from field data. In Table 1 the input conditions for the model and the bending moment that the model reported are presented in tabular form.

Results

The DuPont Access Bridge behaved in a fully continuous manner under the dead load. A predicted moment of 287 kip-feet (389.9kn-m) was reported by the model when the boundary conditions were set such that the bridge would act continuously with pinned reactions that were restrained by a rotational spring with a stiffness of 5500 kip-feet / degree, and the measured moment in girder G was 287.6 kip-feet (389.9kn-m) at the end of the connection test. Since the measured results closely compare with the model results the conclusion is drawn that the bridge behaved continuously. This point is further proven by the presence of tension strains at the top of each girder (negative moment region) at the pier at the end of the tests. At the time of the deck pour, the abutments had been poured, and the integral action was accounted for by the presence of a spring at the end reactions.

Conclusion

The primary conclusion is simply that the method works. The data collected as described herein clearly showed that the design of the connection at the pier led to continuity of the girders under the weight of the freshly poured concrete deck.

CHAPTER 5:

COMPARISON OF LATERAL LOAD DISTRIBUTIONS OF TWO EXPERIMENTAL BRIDGES

Contribution of the Author

The authors of this chapter are David P. Chapman, Kyle P. Scoble, J. Harold Deatherage, Edwin G. Burdette, and David W. Goodpasture. The contributions of Mr. Chapman to this chapter are as follows: assisting in the mobilization and demobilization of resources to and from New Johnsonville, TN and Nashville, TN, assisting in the installation of the gages on the DuPont Access Bridge and Massman Drive Bridges, assisting in the administration of the controlled load tests on the DuPont Access Bridge and Massman Drive Bridges, editing the chapter, and seeing to the publication of the chapter in the Journal of Bridge Engineering.

Introduction

The University of Tennessee (UT) entered into a research contract with TDOT on September 1, 2003. The research involved the instrumentation, testing, and analysis of two experimental bridges. The DuPont Access Bridge and the Massman Drive Bridge, both in the central portion of Tennessee, were considered in this study.

The knowledge of girder load distribution factors is important for the design and evaluation of bridges. The overall bridge construction cost is a function of the loads supported by the girders; lower distribution factors indicate a beam is subjected to smaller loads. Smaller loads result in smaller beams which lead to lower costs. Load distribution is affected by the position of the applied load on the superstructure of the bridge. There are several methods for evaluating load distribution, the current method being the 2007 AASHTO-LRFD Specifications. In addition to this code, Henry's Method and the AASHTO 2002 code were used to compute distribution factors for comparison. Experimental data are needed to assess the accuracy of any method used to predict lateral distribution of wheel loads on highway bridges.

Scope

The two instrumented bridges are experimental in that the girders were erected as simple spans but were designed to act as continuous beams under the dead load of the concrete deck and as continuous composite girders under the live load. The girders are made continuous at the pier by using a cover plate in tension and wedge plates in compression. The purpose of this chapter is to present the results of full scale field tests performed to assess the load distribution factors and compare the measured distributions to analytical methods of determining load distribution.

Bridge Geometry

The DuPont Access Bridge is a two span bridge supported by integral abutments and a pier located between the east and west bound lanes of U.S. Highway 70 near New Johnsonville, TN. The north span of the bridge is 76 ft in length while the south span is 87 ft. The bridge consists of six rolled steel girders and a concrete deck. The girders are spaced 7 ft - 4 13/16 in on center. The concrete deck is 8 1/4 in thick and acts compositely with the girders under live loads. The bridge is not skewed. The girders are braced against lateral torsional buckling under the dead load of the deck by cross braces bolted to web stiffeners and spaced at 25ft oc. Figure 10 is a picture of the DuPont Access Bridge.

The Massman Drive Bridge spans over the east and west bound lanes of Interstate 40 in Nashville, TN. The superstructure of the bridge consists of five steel plate girders and a concrete deck. The girders are spaced 9 ft 9 in on center. The north span of the Massman Drive Bridge is 140 ft and the south span is 147 ft. The bridge is not skewed. The girders are braced against lateral torsional buckling under the dead load of the deck by cross braces bolted to web stiffeners and spaced at 25ft oc. Views of the bridges are shown in Figures 2 and 11.

Girder Designation and Strain Gage Location

Each of the six girders in the south span of the DuPont Access Bridge was designated with a letter. The girders are labeled from west to east, starting with the letter "E" for exterior. The second girder is labeled "F," the third "G," and so on. Girders E, F, and G have multiple strain gages located at several cross sections along their length. Each gage is identified by a number, and each number corresponds with a specific location on a beam. Gages 0 are the gages that are located just north of the pier on the bottom flange of each girder. Gage E0 is the gage at position zero on girder E. This system of letters and numbers was used to identify all gages in the DuPont Access Bridge.

Eighty - four gages were placed at several different cross-sections along three of the five girders (5, 4, and 3 in Figure 3) in the south span of the Massman Drive Bridge. The girders in the Massman Drive Bridge are numbered 1-5, with 1 being the easternmost girder. A numbering system was devised such that each gage had a unique three digit number. The first number was the girder number. Girder number 5 was the westernmost girder in the south span, and girder 1 was the easternmost girder in the south span. The next number in the gage title was the cross-section number. There were eight cross sections where gages were placed. Cross section 1 was at the center of the connection between the girders, and cross section 8 was approximately 1 ft from the face of the abutment. The final number in the gage title was the gage number. Gage number 1 was located on the top of the upper flange, and gage 6 was located on the top of the bottom flange. For example, gage number 586 is located on the top of the bottom flange of girder 5 about 1 ft – 6in from the face of the abutment. Views of the gage locations for each bridge are shown in Figures 6, 7, 12, and 13.

Gages, Data Equipment, Software, and Other Equipment

For both the DuPont Access Bridge and the Massman Dr Bridge an Optim Megadac was used to collect data. The wires connecting the gages to the

Megadac were contained in a conduit that ran from in front of the abutment to the inside of a mobile data collection laboratory. Data were stored in the Megadac and later downloaded to a computer. The software used to administer a test is called TCS (version 3.4.0). TCS defines the test parameters, runs the test, and formats the data.

Controlled Load Tests

Controlled load tests were conducted primarily to determine the lateral load distribution in the girders. The controlled load tests for the DuPont Access Bridge included 14 individual tests, each with the truck in a different lateral position. The truck used in each test was a four axle tandem dump truck provided by TDOT. The truck was loaded with aggregate and weighed 73.5 kips. In order to concentrate the loads, the movable axle was raised, making the truck illegal for normal road operations. Thus, the load was supported on three axles. The front axle of the truck is 15.83 ft in front of the second axle. The second and third axles are spaced 4.42 ft apart. The first individual test, Test 1, was conducted to determine the locations on the bridge where the truck would be located to provide the maximum strain at the near mid span gage locations and at the pier gage locations. These points were located by moving the truck slowly across the bridge from north to south and monitoring the strain readings at several gages. When a maximum reading occurred, the truck was stopped and the point was marked on the deck with chalk. Point D was where the front axle of the truck was located on the bridge to produce a maximum strain near mid span; a point located approximately 32 ft from the face of the south abutment. The truck was traveling in the southward direction when this point was marked. The remaining 13 individual tests were conducted to determine the moments on the bridge with the truck in the various lateral and longitudinal positions.

The controlled load testing of the Massman Drive Bridge consisted of two phases: tests conducted before the parapet was poured and tests conducted after the parapet was poured. This paper reports the results of the tests conducted after

the parapet was poured. A tandem dump truck weighing 73 kips was used for the controlled loading in a similar fashion as the DuPont Access bridge. The maximum strain near the mid span of the south span occurred when the location of the front axle was located at a point labeled, D, approximately 40 ft from the south abutment. The truck was driving in the southward direction when this marking was made. At the Massman Drive Bridge 15 tests were conducted. Figure 14 is a picture of a typical scene during a controlled load test with the axle loads superimposed.

Current Lateral Load Distribution Methods

The following methods are or have been used to determine lateral load distribution on highway bridges.

AASHTO 2002

The distribution of moments on interior and exterior girders is the product of wheel load moment and the factor $S/5.5$ (Equation 1), where S is the spacing between girders in feet (S may not be greater than 14').

AASHTO 2007 LRFD

More accurate results for girder distribution factors can be achieved by using formulae which take into account bridge parameters such as span length and stiffness properties. For concrete decks on steel beams, the lateral load distribution factor for interior girders with one design lane loaded can be determined using the following equation for interior girders:

$$g_{int} = 0.06 + (S/14)^{0.4} (S/L)^{0.3} \left(K_g / 12 L t_s^3 \right)^{0.1} \quad (\text{Equation 2})$$

Where:

g_{int} = distribution factor for interior beams

S = spacing of beams (ft), such that $3.5 \leq S \leq 16$

L = span length of a girder (ft), such that $20 \leq L \leq 240$

t_s = depth of concrete slab (in), such that $4.5 \leq t_s \leq 12$

K_g = longitudinal stiffness parameter (in⁴), such that $10,000 \leq K_g \leq 7,000,000$

$$K_g = n(I + Ae_g^2) \quad (\text{Equation 3})$$

$$n = E_B / E_D \quad (\text{Equation 4})$$

E_B = modulus of elasticity of beam material (ksi)

E_D = modulus of elasticity of deck material (ksi)

I = moment of inertia of beam (in⁴)

A = area of beam (in²)

e_g = distance between the centers of the basic beam and deck (in)

AASHTO 2007 LRFD for Exterior Girders

For one design lane loaded, the girder distribution factor for an exterior girder is computed with the lever rule. The lever rule is a method that sums moments about the first interior girder to get the reaction at the exterior girder, assuming there is a rotational hinge in the bridge deck directly above the first interior girder. Field tests were carried out according to the guidelines set forth by AASHTO. The outer most wheel of the truck was placed directly above the exterior girder and the moment was taken about the first interior girder. The wheels were 1.83 m (6 ft) apart.

Henry's Method

Henry's Method was developed in 1963 by Director of Structures for the Tennessee Department of Transportation, Henry Derthick. It was created to calculate lateral load distribution of live load moment in longitudinal girders and

assumes equal distribution to all girders. The calculation of live load moment distribution factors for prestressed I-beams and steel beams is as follows.

(a) A 3.05 m (10 ft) traffic lane width is assumed, and the fractional number of design traffic lanes is obtained by dividing the roadway width by 10.

(b) The Live Load Resistance Factor (LLRF) expressed as a percentage is obtained by linearly interpolating the number of traffic lanes obtained in step (a) from the scale below:

2 lanes = 100%

3 lanes = 90%

4 lanes = 75%

(c) Multiply the LLRF by the number of traffic lanes obtained in (a) and divide the product by the number of beams.

(d) Multiply (c) by 2 for number of rows of wheels per beam.

(e) Multiply (d) by the ratio 6/5.5 to get the Live Load Moment Distribution Factor for girders.

Results

The raw data collected during the controlled load testing of the DuPont Access and Massman Drive Bridges consist of strains measured at multiple cross sections over the length of a girder. In the case of both the DuPont Access and Massman Drive Bridges, the raw data were reduced according the following procedure:

- 1) The average strain value recorded for each gage for the first and last 10 seconds of a test (no load present on the bridge) were determined to measure the drift in a gage over the duration of a test. The drift was calculated by finding the difference between the average value over the beginning and ending periods of each test. The average among a

sample of drift values was less than 1 microstrain for the Massman Drive Bridge.

- 2) Plots of strain versus time for a series of gages were created and used to identify the time periods when the truck was stationary at the location that produced the maximum strain reading in the gages located on top of the bottom flange near midspan (positive moment region).
- 3) A single value of strain was determined for a given test and longitudinal load position by taking an average over 10 seconds of a time period described in step 2.
- 4) For a given truck position and girder the distribution factor was calculated as the percentage of the sum of the strains at the top of the bottom flange measured at that girder. For example: when the truck was stopped overtop of girder 5, and at point D on the Massman Dr Bridge the total strain for all the bottom flange gages was 190 microstrain with 68 microstrain measured at the top of the bottom flange at girder 5. The GDF for girder 5 is .36 or $68 / 190$. Since only 3 of the 5 girders were instrumented, the data were mirrored about the centerline. In the case of the test described above, the strains at the top of the bottom flanges of girders 4, 5, and 6 were measured during that test, and the strains at the top flange of girders 2 and 3 were taken to be equal to the strains measured at the top of the bottom flanges of girders 5 and 6 when the load was stopped at point D and directly above girder 3.

The lateral load distributions in the girders for different load positions for the DuPont Access Bridge are shown in Figures 15 through 19, and those for Massman Drive Bridge are shown in Figures 20 through 23. The calculated and measured load distribution factors are tabulated in Tables 2 and 3.

Discussion of Results

Girder distribution factors calculated using AASHTO 2002, AASHTO 2007 LRFD, and Henry's Method are higher in the Massman Drive Bridge than in the

DuPont Access Bridge due to fewer girders and larger spacing in the Massman Drive Bridge. The AASHTO 2007 LRFD values for interior girders compare similarly between bridges with values of 0.466 for Massman and 0.430 for DuPont. The load distribution factors from field measurements for DuPont Access and Massman Drive were consistently below the values set forth by AASHTO 2002 for both the interior and exterior cases. The field measurements for interior girders are closer to the standard AASHTO 2007 LRFD value than are those for the exterior girder cases. The cantilever method for distributing loads to exterior girders is used in AASHTO 1996 and AASHTO 2007 LRFD. Based on the test results reported herein, this method is conservative. This conservatism results from the assumption in the cantilever method that the slab is pinned at the first interior girder. Both bridges experience a girder distribution factor between 0.4 and 0.5 for the load case nearest the exterior girder. Load factors decrease in value as the truck is moved closer to the centerline of each bridge. This was expected because as the truck moves toward the center of the bridge, the load is dispersed through more girders.

Conclusions

The purpose of this paper was to compare the load distribution factors for two experimental two-span highway steel girder bridges. The girder distribution factors from field measurements were consistently less than those obtained by any of the design methods. The AASHTO 2007 LRFD values were closer than those obtained by any other method when comparing interior girders. On the other hand, exterior girder distribution factors were closer to the values produced from Henry's Method. The AASHTO 2007 LRFD values for exterior girders obtained by the lever rule are consistently higher than those obtained from Henry's Method and significantly higher than those measured. The long-used cantilever method is extremely conservative. AASHTO 2002 distribution factors were shown to be conservative across the board when compared with field measurements. The girder distribution factors obtained for the two bridges were reasonably consistent.

CHAPTER 6:

INFLUENCE OF A SECONDARY ELEMENT ON THE LATERAL LOAD DISTRIBUTION OF A STEEL GIRDER BRIDGE

Introduction

The AASHTO LRFD Specifications do not consider the effect of secondary structural elements, such as parapets, diaphragms, or lateral bracing on lateral load distribution of live load on slab-and-girder bridges. The purpose of this research is to evaluate, through full-scale testing, the structural contribution of concrete parapets as they affect lateral distribution of live loads. Controlled load tests were conducted on the Massman Dr Bridge in Nashville, TN before and after the parapets were constructed. Through a comparison of the wheel load distribution factors (LDF) from the 1st and 2nd controlled load test, the contribution of the parapet was determined.

Research Methodology

The Massman Drive Bridge has a TDOT standard jersey type parapet with 1 inch deep sawed joints on 25 ft. centers (see Figures 24). A cross section of the TDOT standard jersey type parapet is shown in Figure 25. This analysis assumes the parapet and the bridge deck act compositely based on the facts that two #4 reinforcing bars protrude from the deck into the parapet on 1 ft centers along the entire length of the bridge ("J" shaped bars referred to as Bars B471E in Figure 25), and the surface between the deck and the parapet was not finished smooth at the time of the deck pour, as shown in Figure 26. The joints in the parapet were created by saw cutting a 1" deep groove in the partially cured concrete shortly after the parapets were poured.

The controlled load testing consisted of two phases: tests conducted before the parapet was poured and tests conducted after the parapet was poured. Two sets of data were taken so that the lateral load distributions could be compared and the effect of the parapet determined. A loaded dump truck

weighing approximately 68,020 lbs was placed at various locations on the Massman Drive Bridge in order to determine the lateral distribution of the load to the girders during the first set of tests (prior to the parapet). A similar truck was used during the second set of tests (after the parapet was poured), but the weight was approximately 71,990 lbs. At the beginning of the testing, the truck was slowly moved from the north end of the bridge to the south end of the bridge. As the truck moved, several gages were monitored, and the truck was stopped when the maximum strain at the pier or the maximum strain near the middle of the south span occurred. The location of the front axle of the truck was labeled A, B, or C when one of the points of maximum strain was reached. Points A and B correspond to the location of the truck where the maximum strain at the pier was produced, and C corresponds to the location of the truck where the maximum strain near midspan (gages 476, 576, and 676 as shown in Figure 12) of the south girder was produced.

Plots of strain v. time were created for several key gages so that time intervals could be established for each time period where the truck was stopped at position A, B, or C during a given test. After all time intervals were established, the strains that occurred in a portion of each interval were averaged to establish a single value of strain for each truck position, gage, and test. For each truck position, plots of strain v. depth were then created to identify erroneous readings. After the values of strain were established and checked for erroneous readings, they were converted to moment. Equation 5 was used to convert the strain value at a given cross section into a moment, and was derived by substituting ϵE for σ in the equation for maximum bending stress, and solving for M.

$$M = \frac{\epsilon EI}{c} \quad \text{Eqn 5}$$

In which, ϵ is the strain at a point; E is the Modulus of Elasticity and is taken as 32,000 ksi in all cases (32,000 ksi is the measured Modulus of Elasticity for the steel using weldable gages); I is the Moment of Inertia. The distance to the gage

location from the neutral axis of the member is denoted by c . The LDF for each girder was calculated by determining the percentage of total moment in the bridge that occurred in a particular girder when the truck was at a specific location. Not all girders were instrumented, so some of the data were mirrored about the center line. For example, when the truck was stopped at position C and between girders 2 and 3, the strains reported for girders 2 and 3 were the strains measured for girders 5 and 6 when the truck was between 5 and 6, and stopped at C.

Results

The results of the analysis, shown in Figures 27 through 31, are presented in the form of the plots of LDF for each girder before and after the parapets were poured. The y axes of Figures 27 through 31 represent the ratio of the moment that was measured in a single girder near midspan to the total moment measured in the bridge near midspan. The numbers 1 through 5 along the x axis correspond to the girders where the moments were measured. In the process of calculating the LDF, the total measured moment in the bridge was found. Figures 27 and 32 report negative distribution factors of the girders farthest from the location of the load as the girders deflected up rather than down in those load cases.

Visual Analysis, a frame analysis program, was used to determine a range of moments in the Massman Drive Bridge. A group of three axle loads, totaling 68,020 lbs and 71,990 lbs respectively, were applied to the models. Four models were used to determine a reasonable range of values of moment. The models consisted of two continuous spans of 140 ft and 147 ft with pinned or fixed reactions representing the abutments and a knife edge reaction at the pier, and a moment of inertia equal to that of the entire cross section of the bridge with and without the parapet. The models with pinned reactions at the piers were used to determine the upper limit of reasonable values of measured moments near midspan, and models with fixed reactions at the piers were used to determine the lower limit of reasonable values of measured moments near

midspan. A range of values was determined in order to account for the presence of integral action. Integral action occurs because the deck and the abutments were constructed monolithically with no expansion joint. This in effect forms a moment connection between bridge superstructure and the piles supporting the abutment. This moment connection is not fully effective as the piles allow some rotation. Table 4 summarizes the comparison between the measured moments and the range of calculated moments. As previously noted, and as observed from Table 4, the results from the test with the truck load over girder 3 during load test 2 differ substantially from the calculated values of moment and no conclusions are drawn from that specific test, and no plotted comparison has been presented.

While the difference in the percentages of moment taken by the exterior girder with and without the parapets was not large, the results were consistent. The effect of the parapet, as expected, was to stiffen the outside girders and thus attract a larger percentage of the total load to the outside girders. In some cases the addition of the parapet caused lower loads on the interior girders.

Conclusions

This chapter has presented the results of two controlled load tests on a slab and girder type bridge. The primary thrust of the research reported in this chapter was to evaluate the effect of concrete parapets on lateral load distribution. These effects are illustrated in Figures 27 through 32. The parapets stiffened the outside girders thereby attracting more load to the outside girders.

CHAPTER 7

SUMMARY AND CONCLUSIONS

The primary theme of this dissertation has been to extend and refine the conclusions drawn in previous reports related to the Massman Drive Bridge, and to a lesser extent, the DuPont Access Bridge. Conclusions furthering the level of understanding related to the girder distribution factors, the structural contribution of secondary elements, and the performance of the Pier-Plate Connection form the primary thrust of this dissertation.

The conclusions detailed in this report are summarized as follows:

- 1) The measured girder distribution factors for the DuPont Access Bridge and the Massman Dr. Bridge were consistently less than those obtained by any of the design methods.
- 2) Among the design methods for determining girder distribution factors the AASHTO 2007 LRFD method compared well for interior girders, and Henry's method worked best for the exterior girders.
- 3) The Lever Rule (also known as the cantilever method), as long suspected, was proven to be highly conservative.
- 4) The parapets stiffened the outside girders, attracting more load to them, and in some cases led to lower loads on the interior girders.
- 5) The design process has been successful because the bridges have behaved as they were designed. The use of the Pier-Plate connection detail has decreased the impact of the girder erection process on the traveling public.

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APPENDIX

A. Tables

Table 1: Model outputs related to the evaluation of the DuPont Access Bridge.

Case	Moment 34' from the South Abutment (kip-feet)
Simply supported , Pinned at the pier	752
Continuous action, Pinned at the pier	435
Continuous action, Pinned at the pier, Rotational spring at the North and South abut. w/ K=5500 kf/deg	287 * as tested

Table 2: Load Distribution Factors for DuPont Access Bridge.

AASHTO 1996 (Interior)	AASHTO LRFD (Interior)	AASHTO LRFD (Exterior)	Henry's Method (Int/Ext)	Research Analysis		
				Lateral Truck Location	GDF	Located on Girder Type
0.529	0.418	0.595	0.539	E & F	0.415	Exterior
				F	0.378	Interior
				F & G	0.338	Interior
				G	0.329	Interior
				G & H	0.294	Interior

Table 3: Load Distribution Factors for Massman Drive Bridge.

AASHTO 1996 (Interior)	AASHTO LRFD (Interior)	AASHTO LRFD (Exterior)	Henry's Method (Int/Ext)	Research Analysis		
				Lateral Truck Location	GDF	Located on Girder Type
0.696	0.457	0.692	0.690	1 & 2	0.5	Exterior
				2	0.37	Exterior
				2 & 3	0.28	Interior
				3	0.41	Interior

Table 4: Summary of measured and calculated moments for the Massman Drive Bridge.

Load Test	Total Load	Measured		Calculated Moment (kft)
		Load Location	Moment (kft)	
1	68020 lbs	1 and 2	1080	Upper: 1484 Lower: 794
		2	1108	
		2 and 3	1129	
		3	1090	
		3 and 4	1136	
		4	1107	
		4 and 5	1089	
2	71990 lbs	1 and 2	1342	Upper: 1512 Lower: 808
		2	1348	
		2 and 3	1222	
		3	969	
		3 and 4	1306	
		4	1355	
		4 and 5	1345	

B. Figures

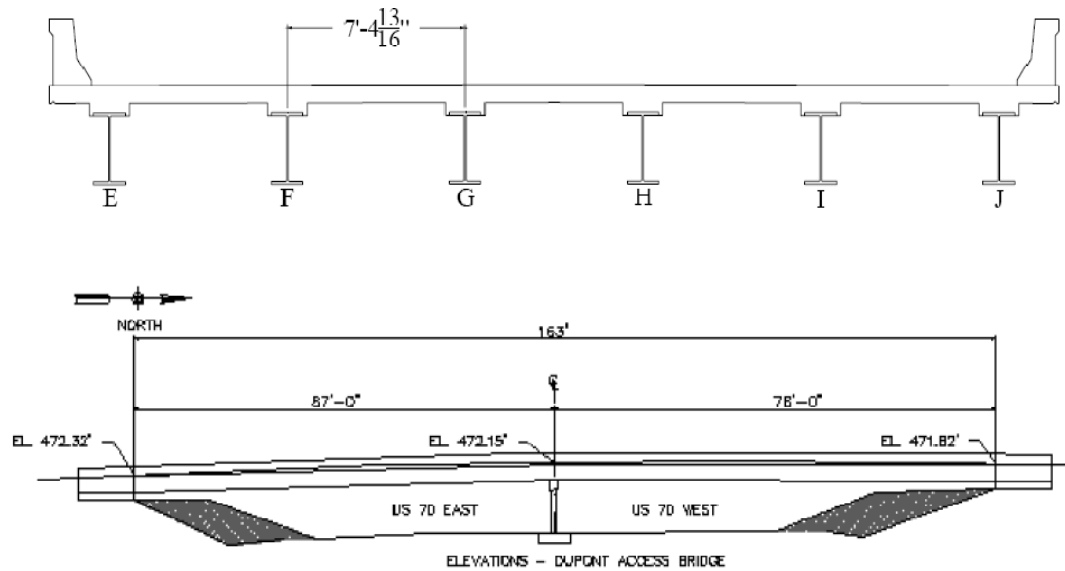


Figure 1: Cross section and elevation of the DuPont Access Bridge.

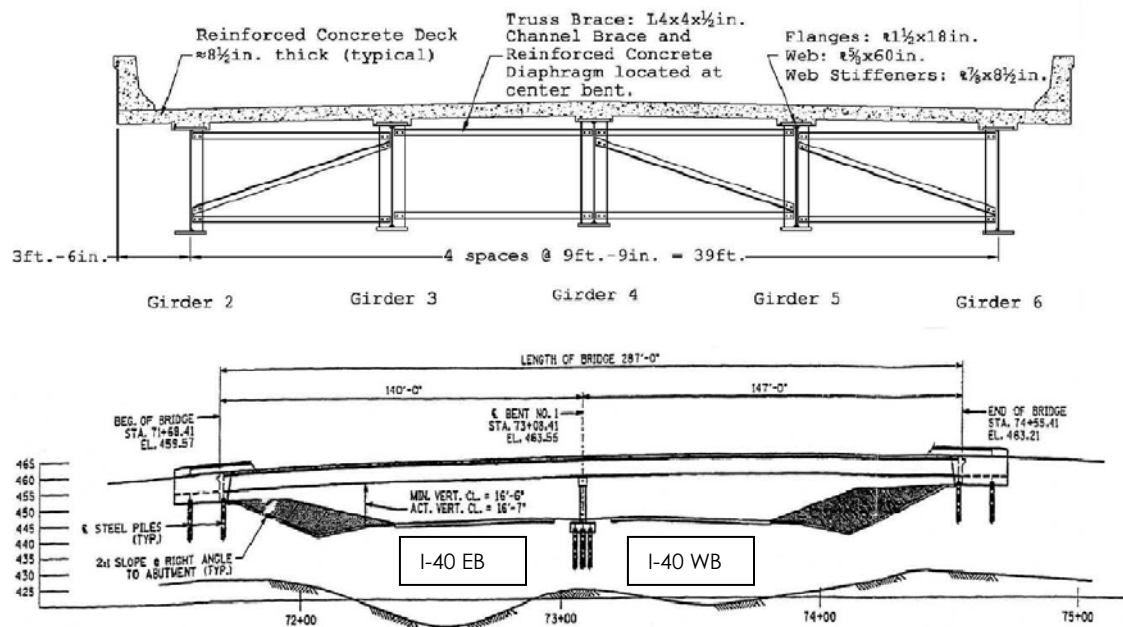


Figure 2: Elevation and cross section of the Massman Drive Bridge.

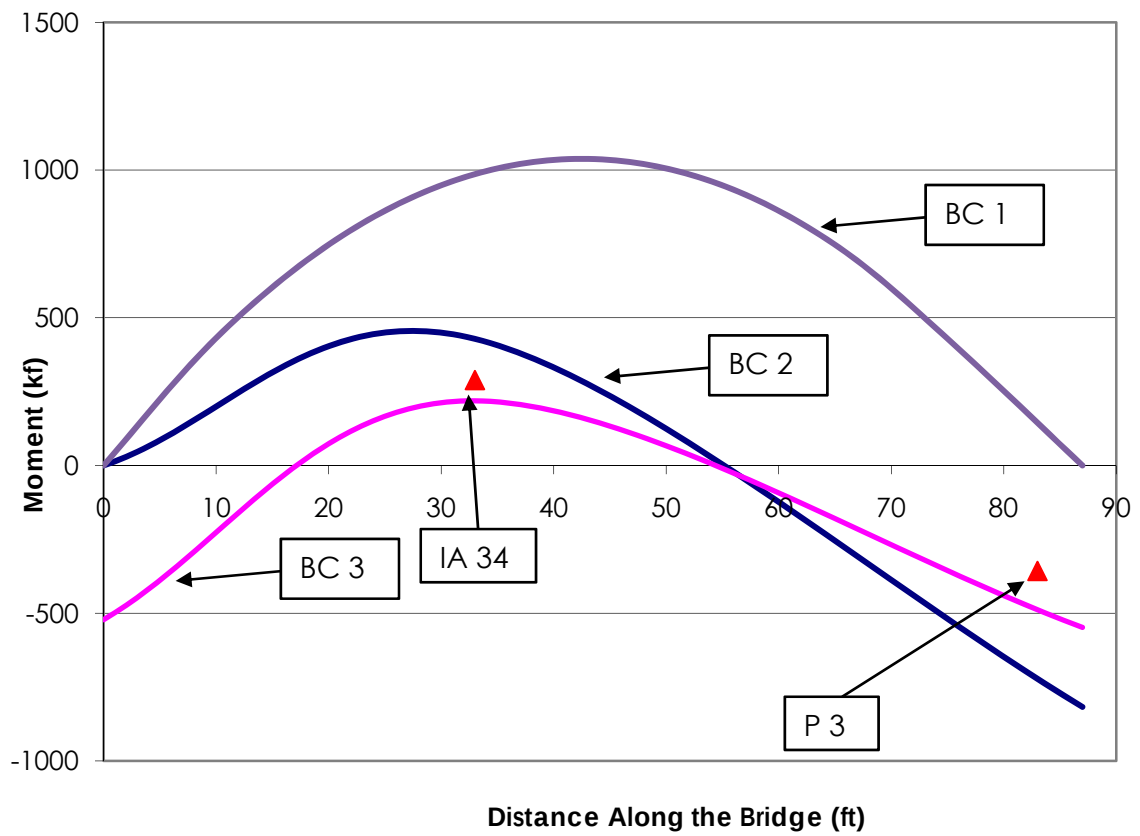


Figure 3: Moment diagram for girder G of the DuPont Access Bridge.

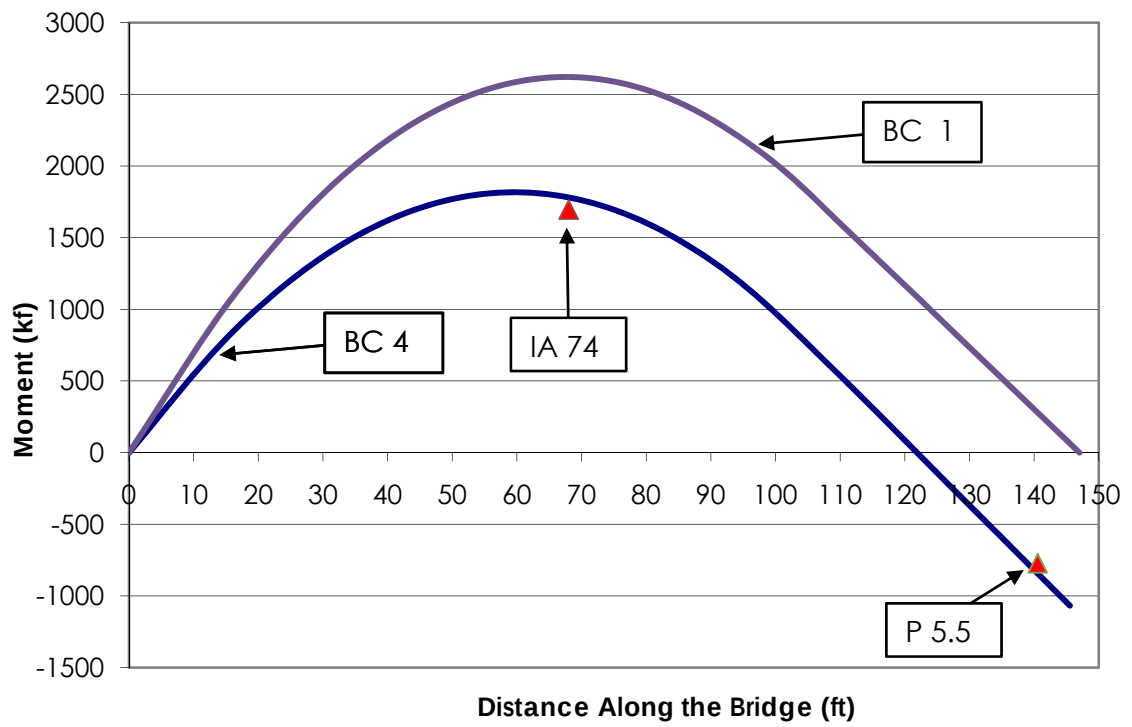


Figure 4: Moment diagram for Girder 5 of the Massman Drive Bridge.

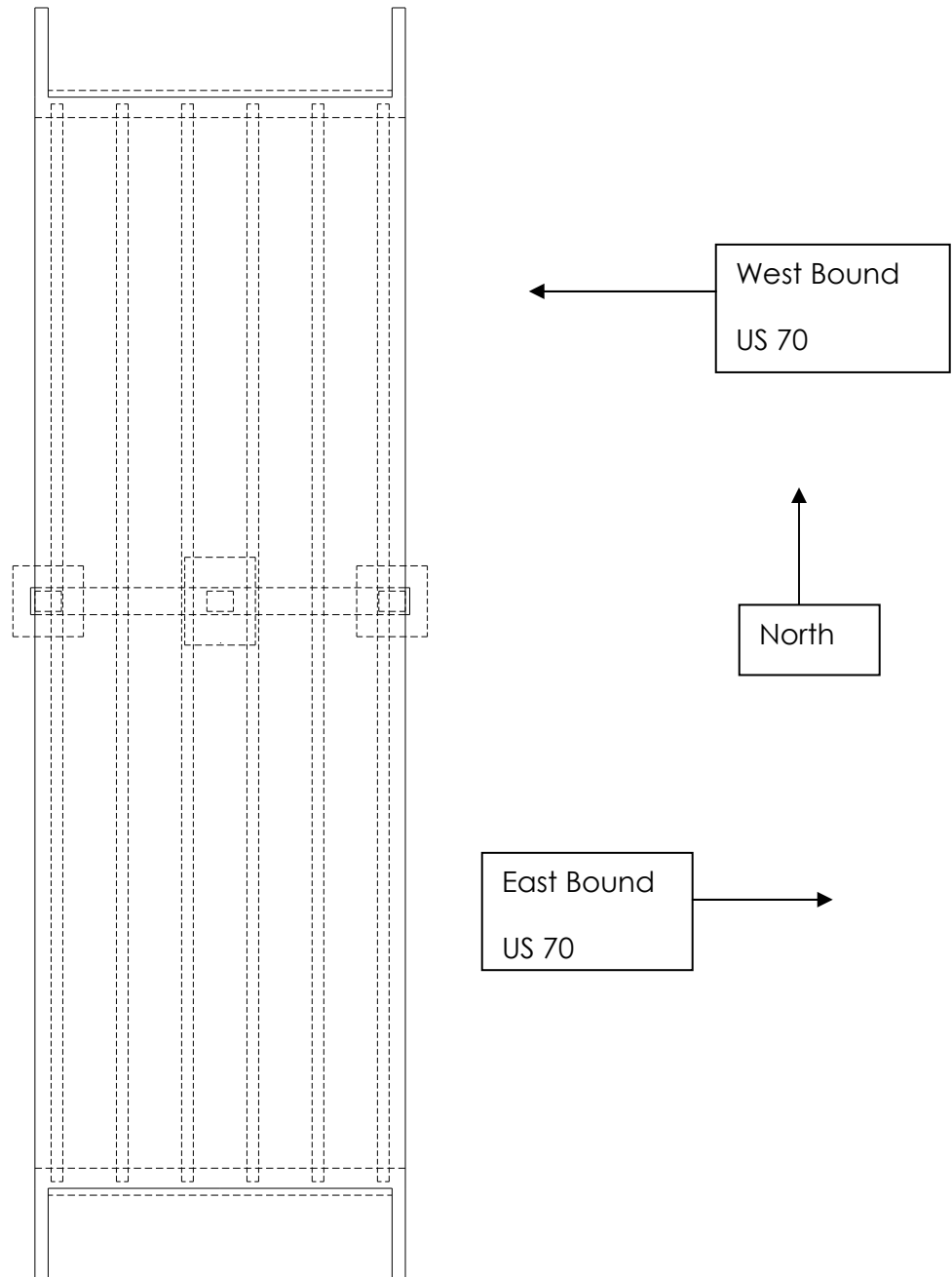


Figure 5: Plan View of the DuPont Access Bridge.

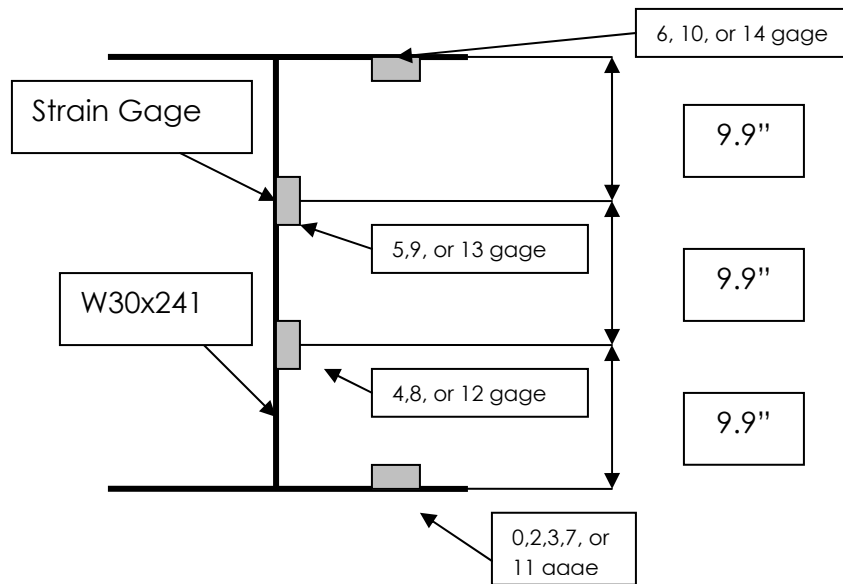


Figure 6: Section Showing Gage Position on Girder (DuPont Access Bridge).

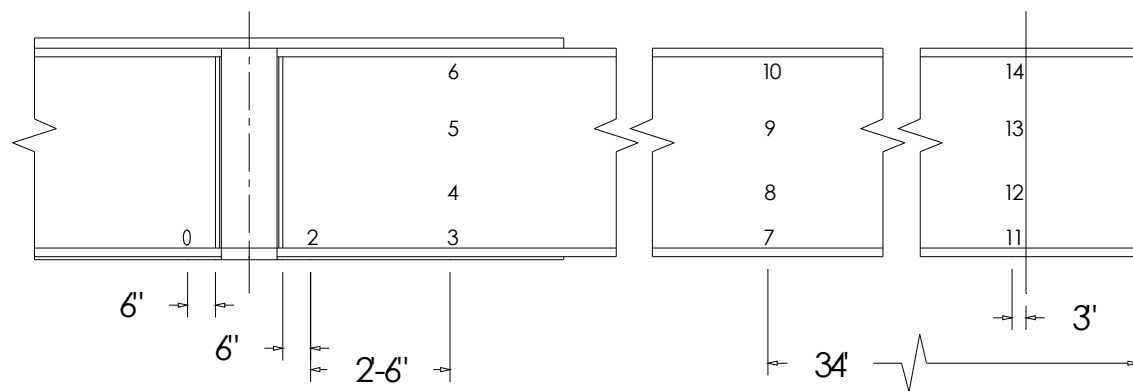


Figure 7: Longitudinal Gage Position of the DuPont Access Bridge.

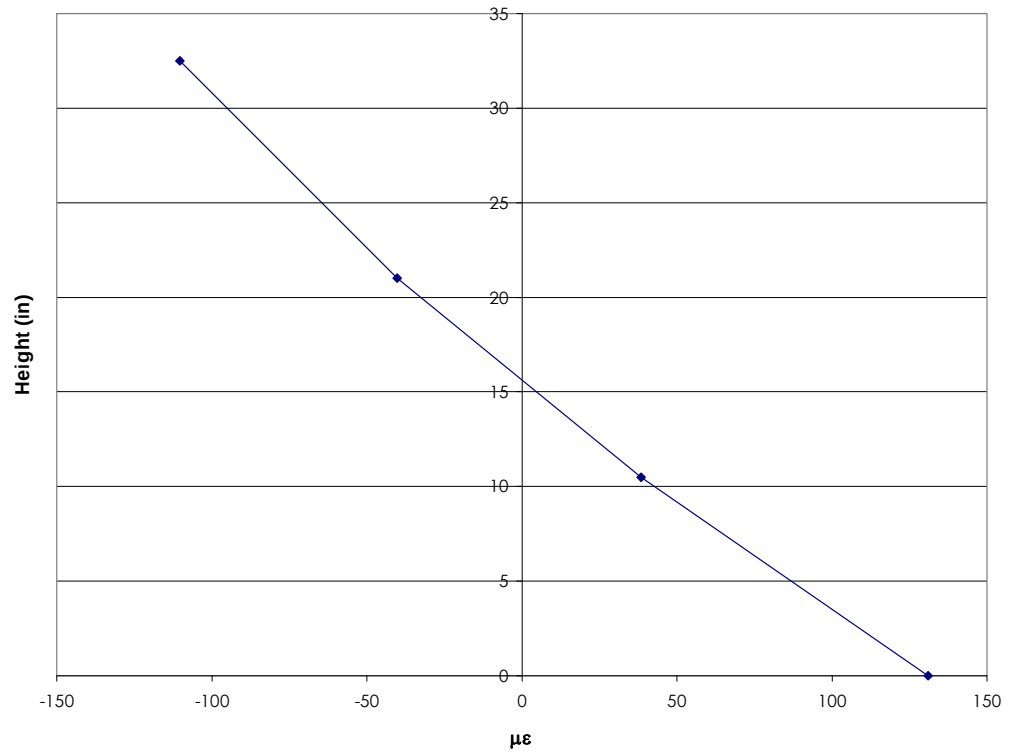


Figure 8: Strain versus Depth for Gages E7 through E10.

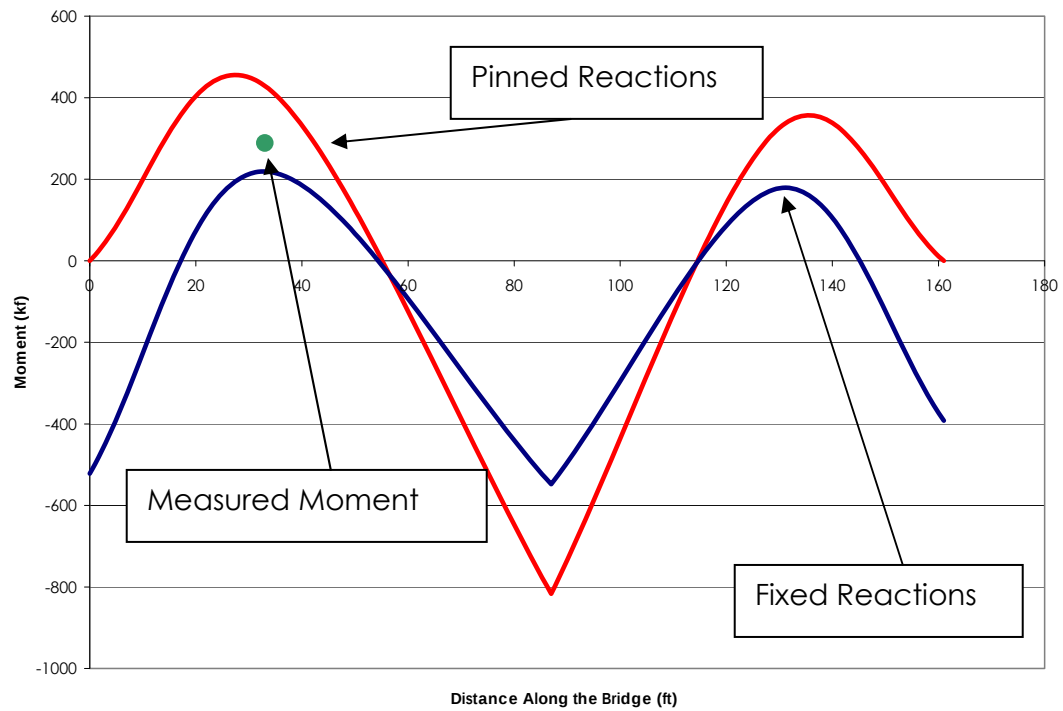


Figure 9: Moment Diagram Showing Upper and Lower Bounds of Model Results.



Figure 10: Photograph of DuPont Access Bridge Elevation Looking East.



Figure 11: Photograph of Massman Drive Bridge Elevation Looking West.

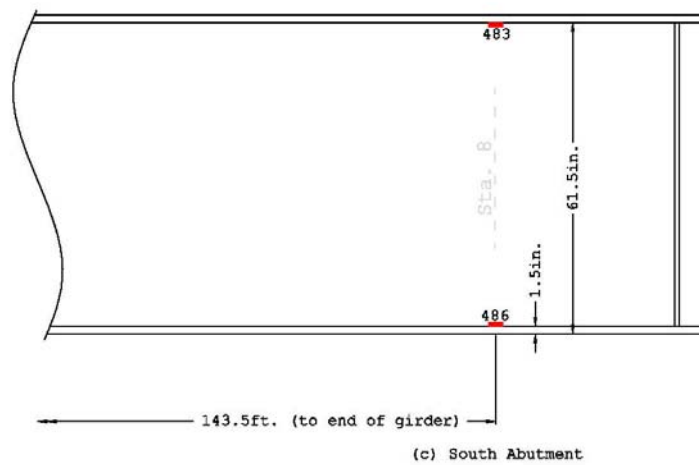
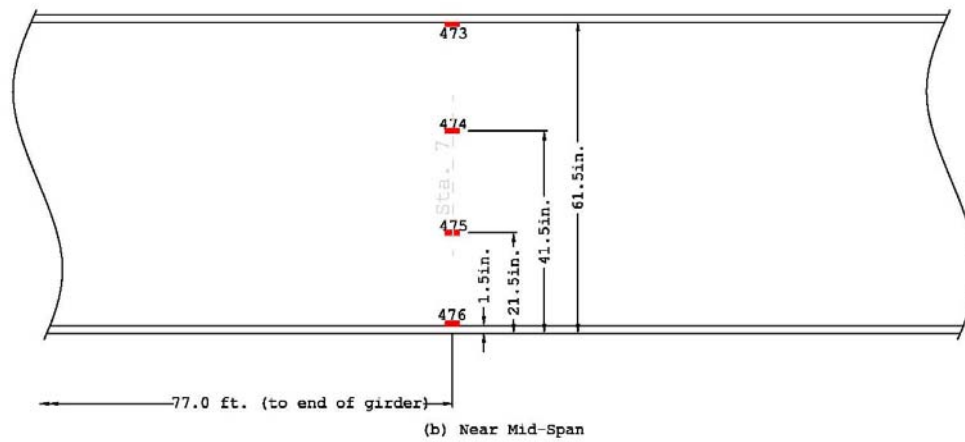
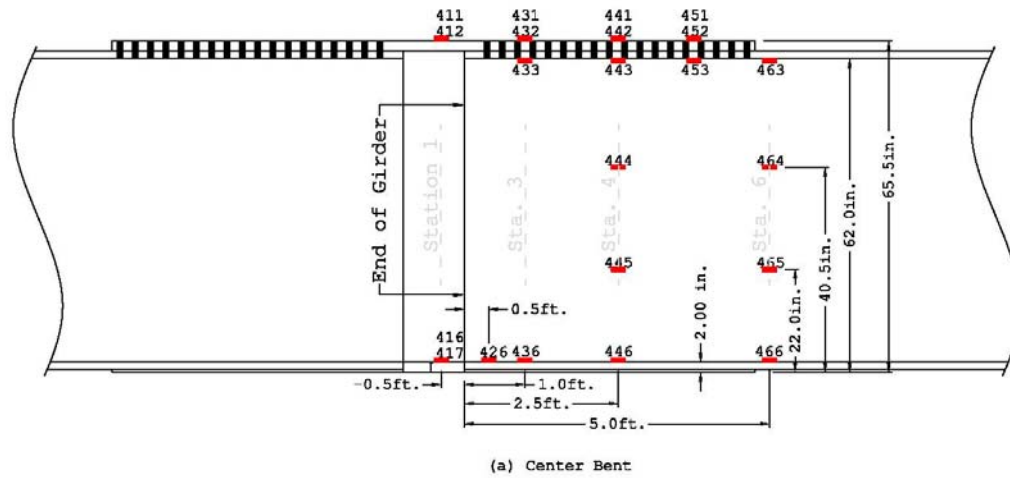


Figure 12: Longitudinal Gage Position of Massman Drive Girder.

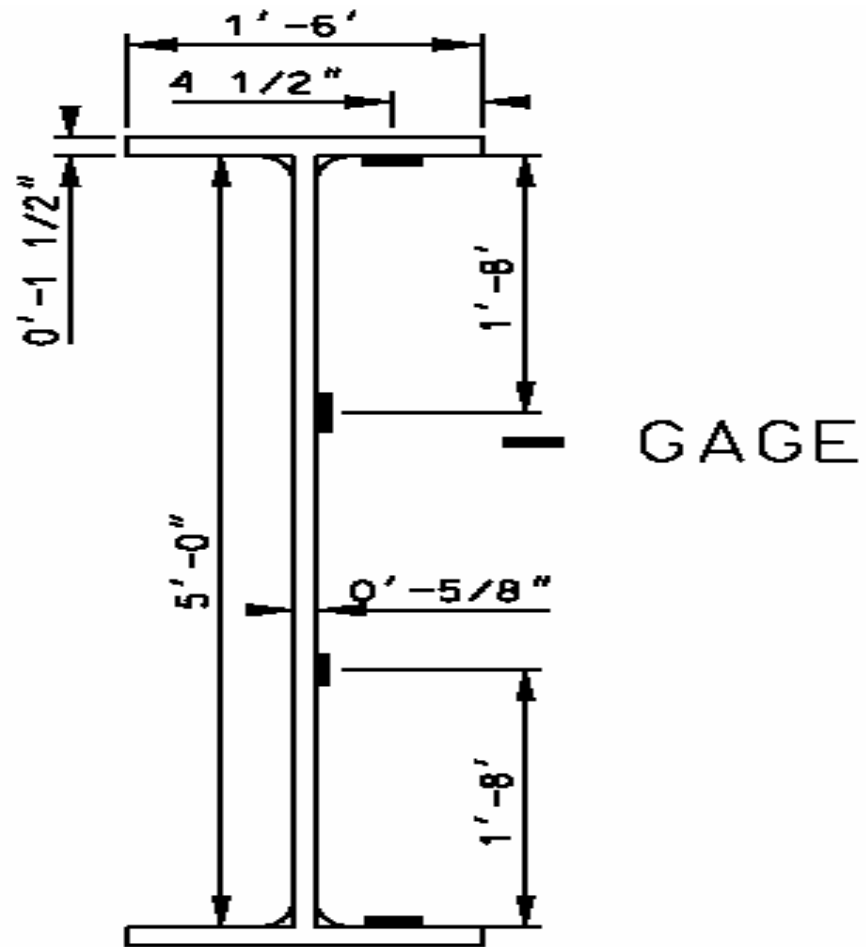


Figure 13: Cross Section of Massman Drive Girder with Gage Position at Mid span.



Figure 14: Typical scene of a controlled load test (Massman Dr Bridge). Note the white line perpendicular to the double yellow line marking the position of the front axle when the truck was stopped at position D.

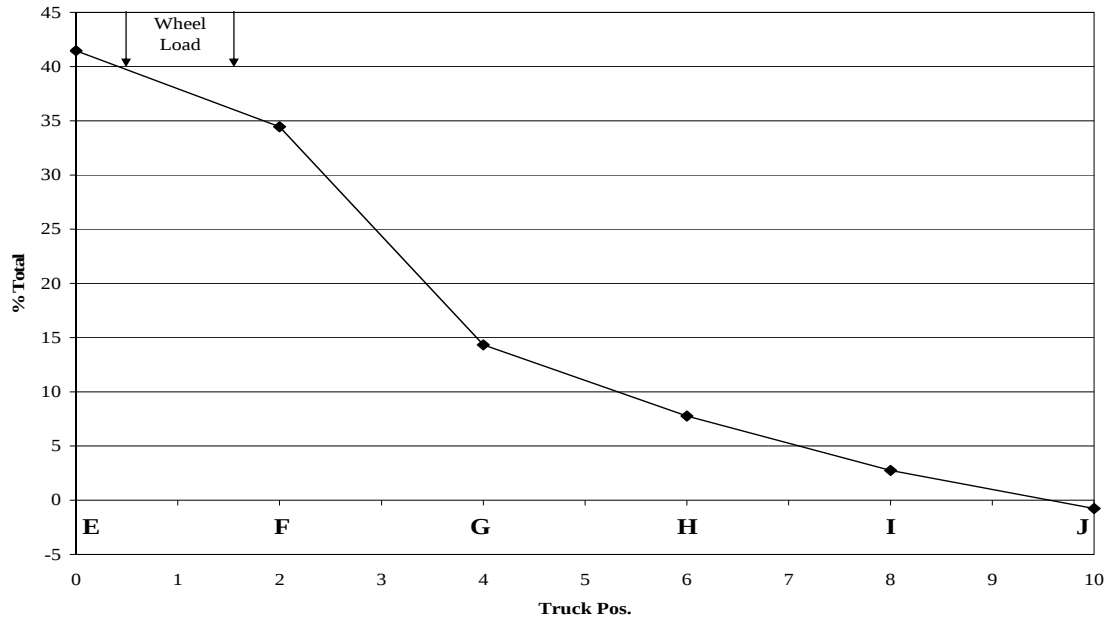


Figure 15: DuPont Access Load Distribution Factors for Load between Girders E and F (Positive Moment).

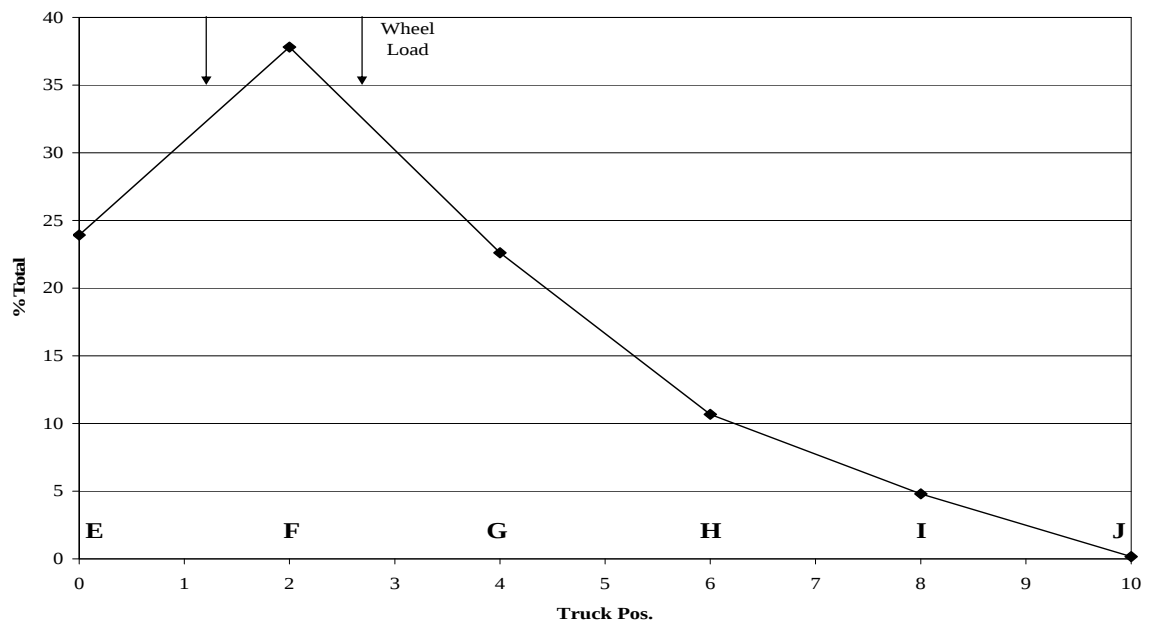


Figure 16: DuPont Access Load Distribution Factors for Load over Girder F (Positive Moment).

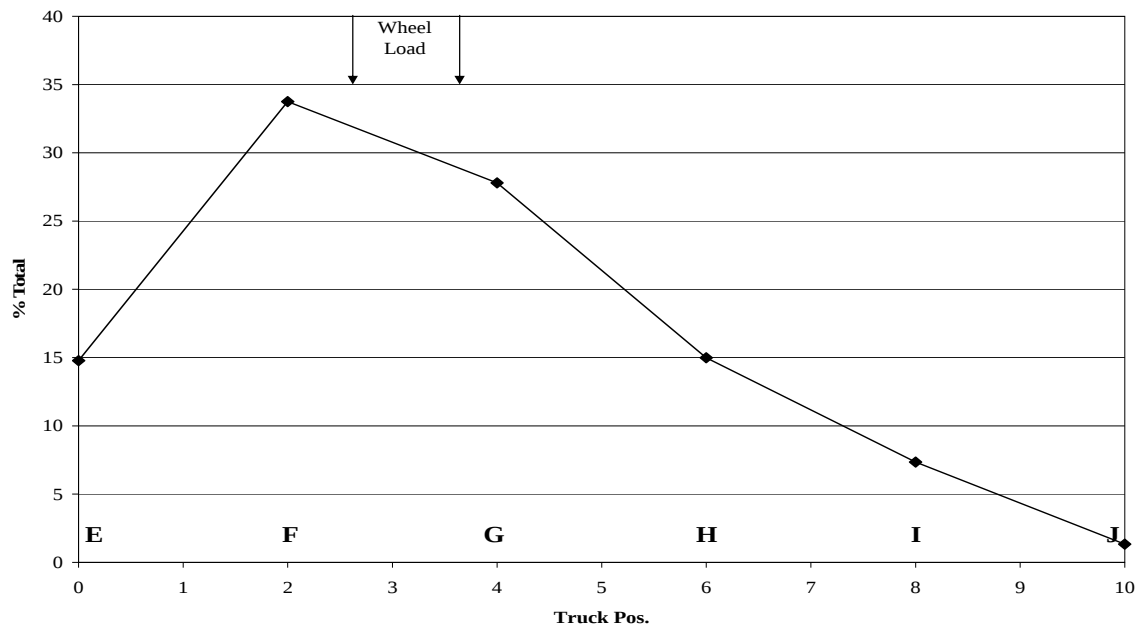


Figure 17: DuPont Access Load Distribution Factors for Load between Girders F and G (Positive Moment).

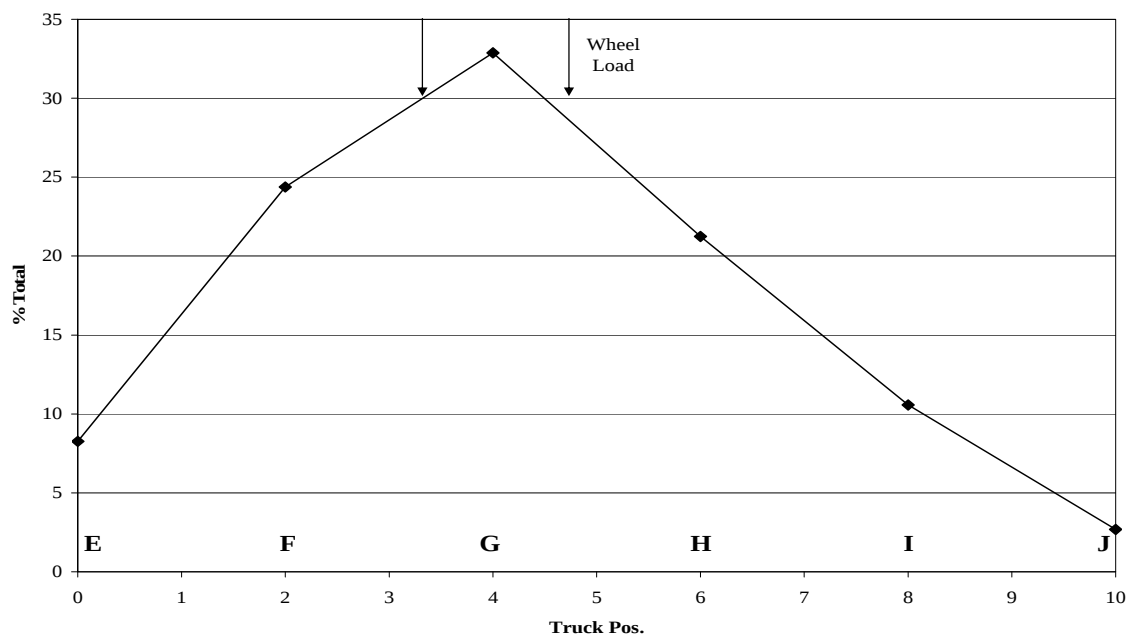


Figure 18: DuPont Access Load Distribution Factors for Load over Girder G (Positive Moment).

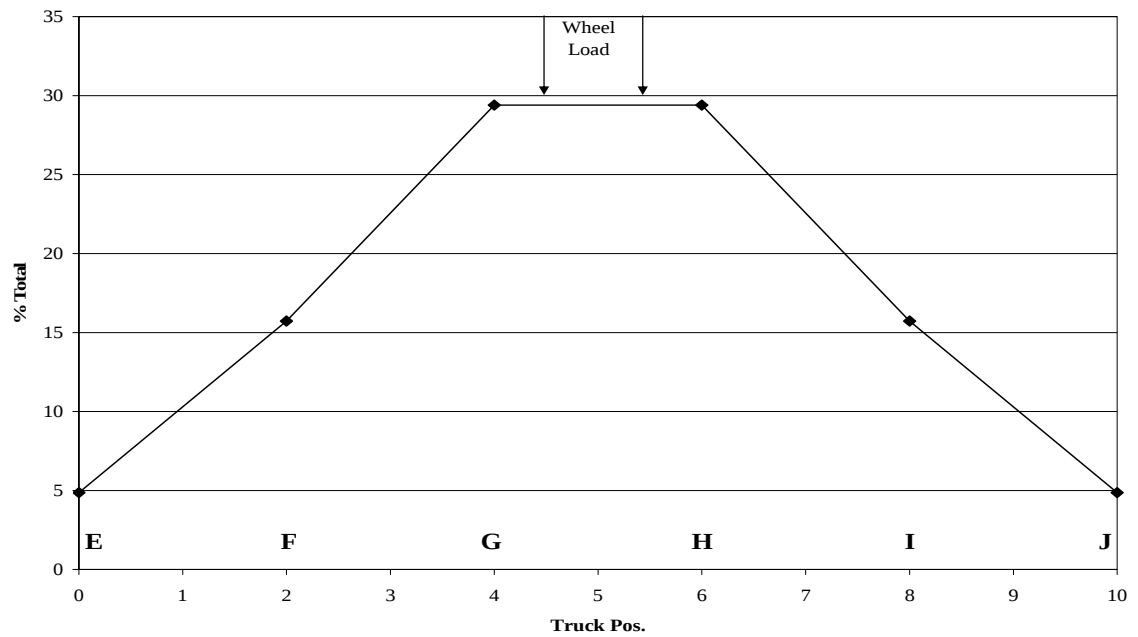


Figure 19: DuPont Access Load Distribution Factors for Load between Girders G and H (Positive Moment).

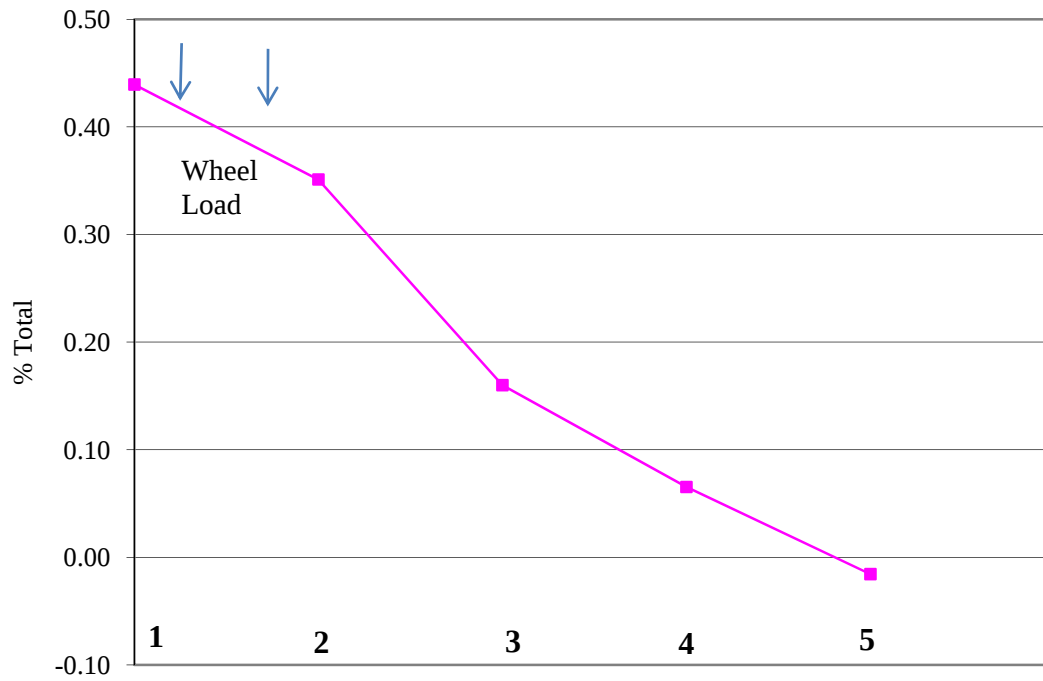


Figure 20: Massman Drive Load Distribution Factors for Load between Girders 1 and 2 (Positive Moment).



Figure 21: Massman Drive Load Distribution Factors for Load Over Girder 2 (Positive Moment).



Figure 22: Massman Drive Load Distribution Factors for Load Between Girders 2 and 3 (Positive Moment).

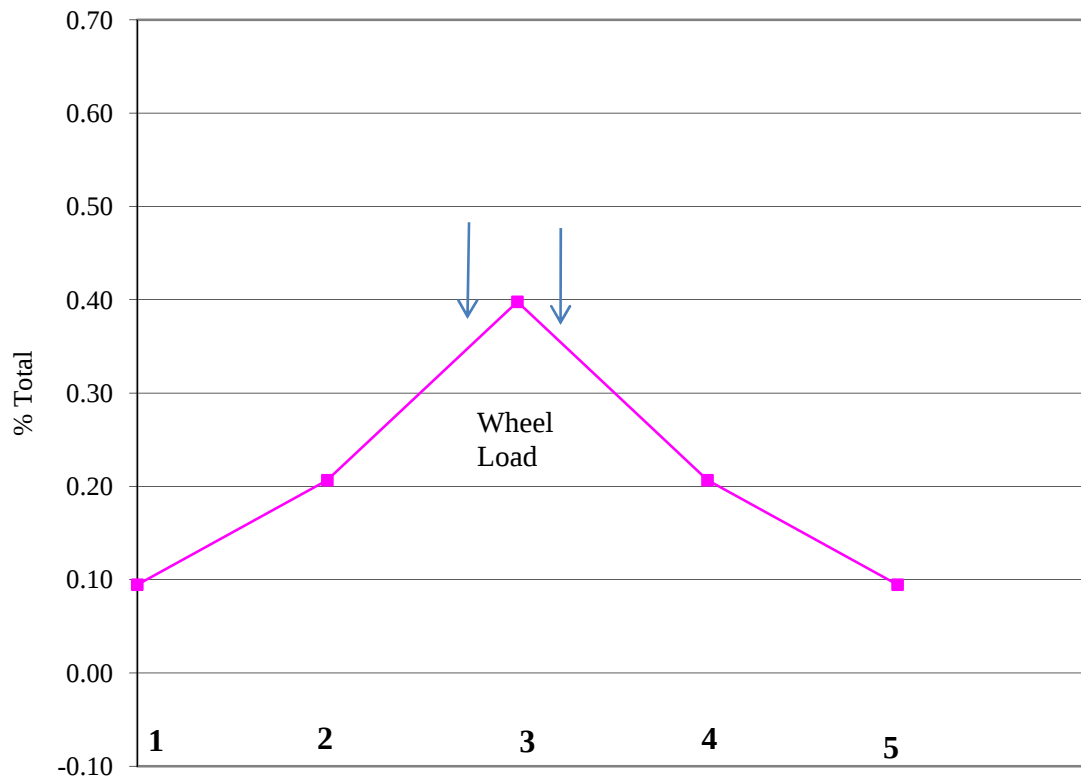


Figure 23: Massman Drive Load Distribution Factors for Load over Girder 3 (Positive Moment).



Figure 24: Typical scene (looking North) during 2nd controlled load test showing the parapet on the East side of the Massman Drive Bridge. Note the sawed joint in the parapet in the right foreground of the figure.

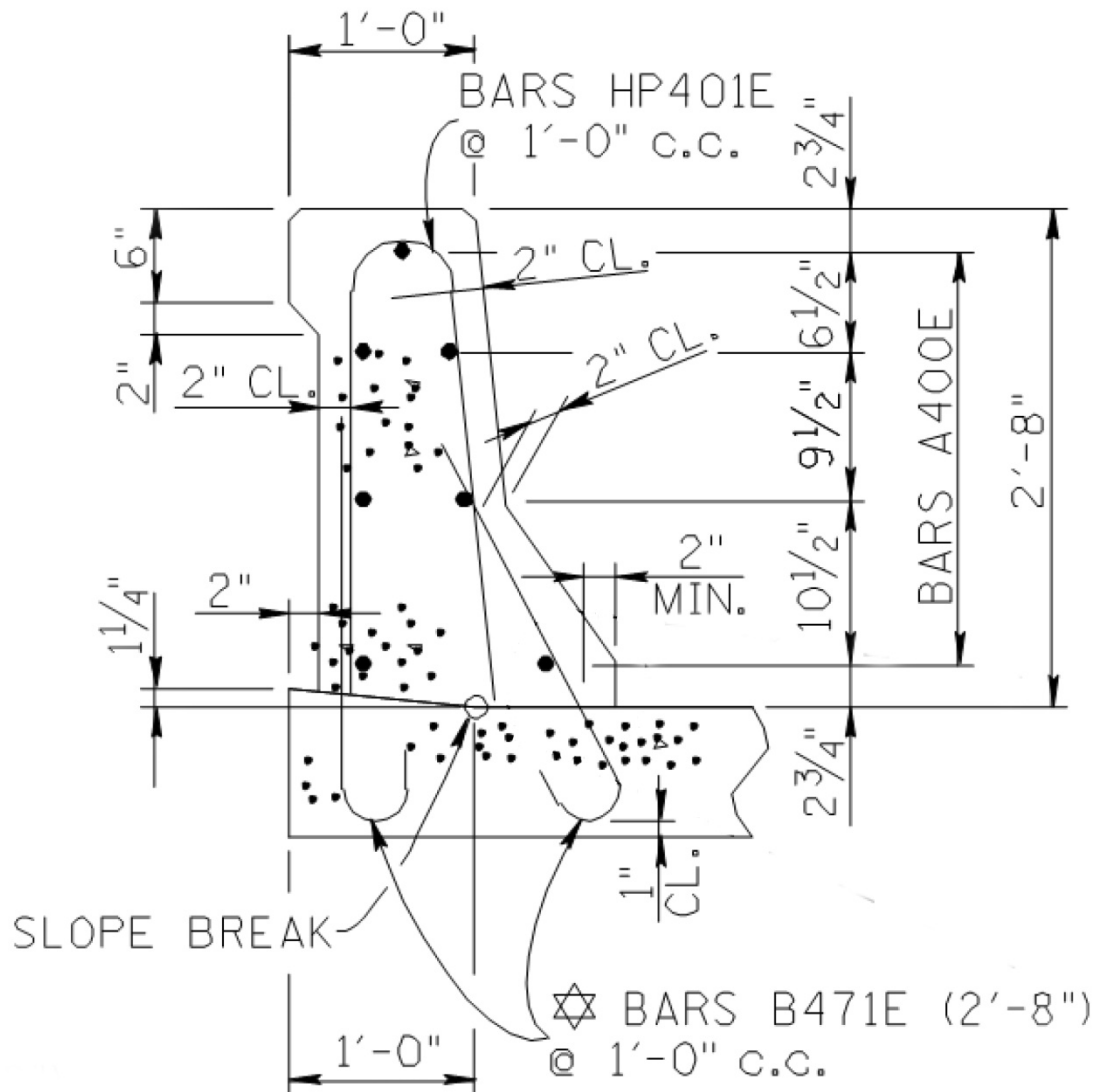


Figure 25: Cross section of TDOT standard "Jersey Type" bridge parapet.



Figure 26: Typical scene at the start of the 2nd day of the deck pour of the Massman Drive Bridge. Note that no attempt has been made to finish or smooth the surface of the deck that that is enclosed between the “J” shaped reinforcing bars (B 471E bars) protruding from the deck.

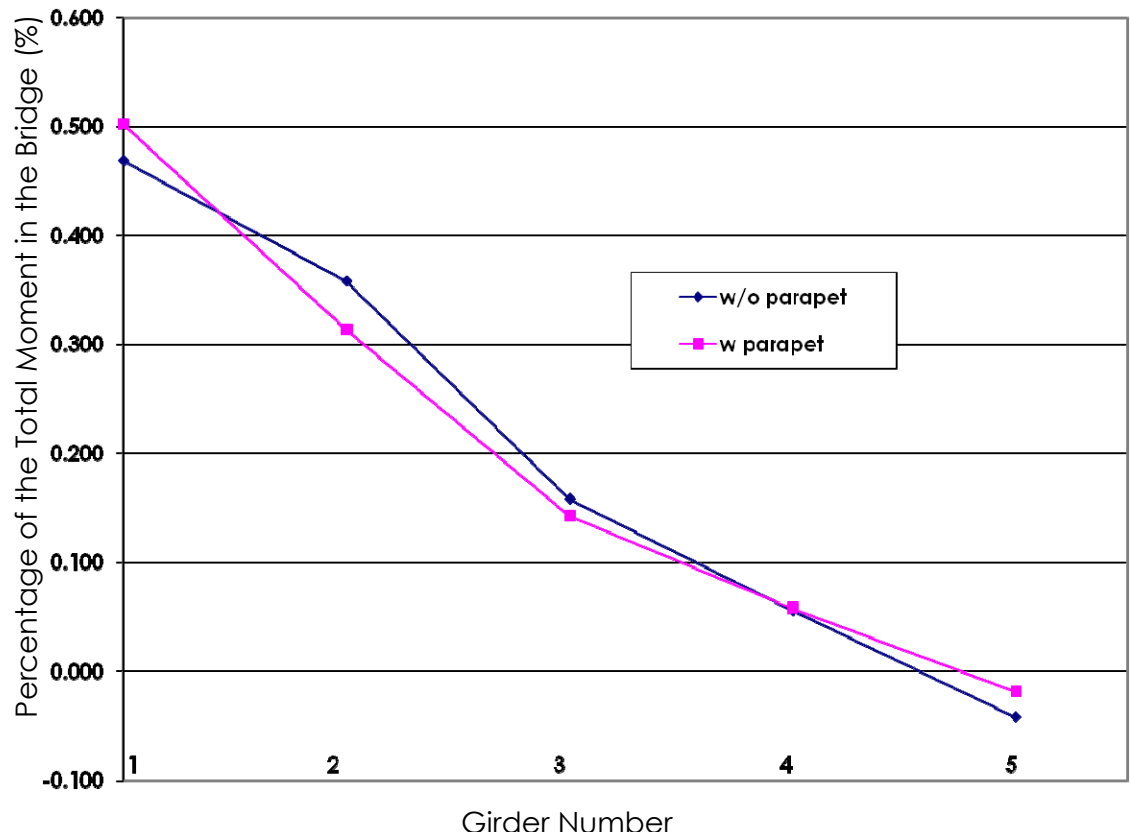


Figure 27: Comparison of girder distribution factors with the load located between girders 1 and 2 at longitudinal location C.

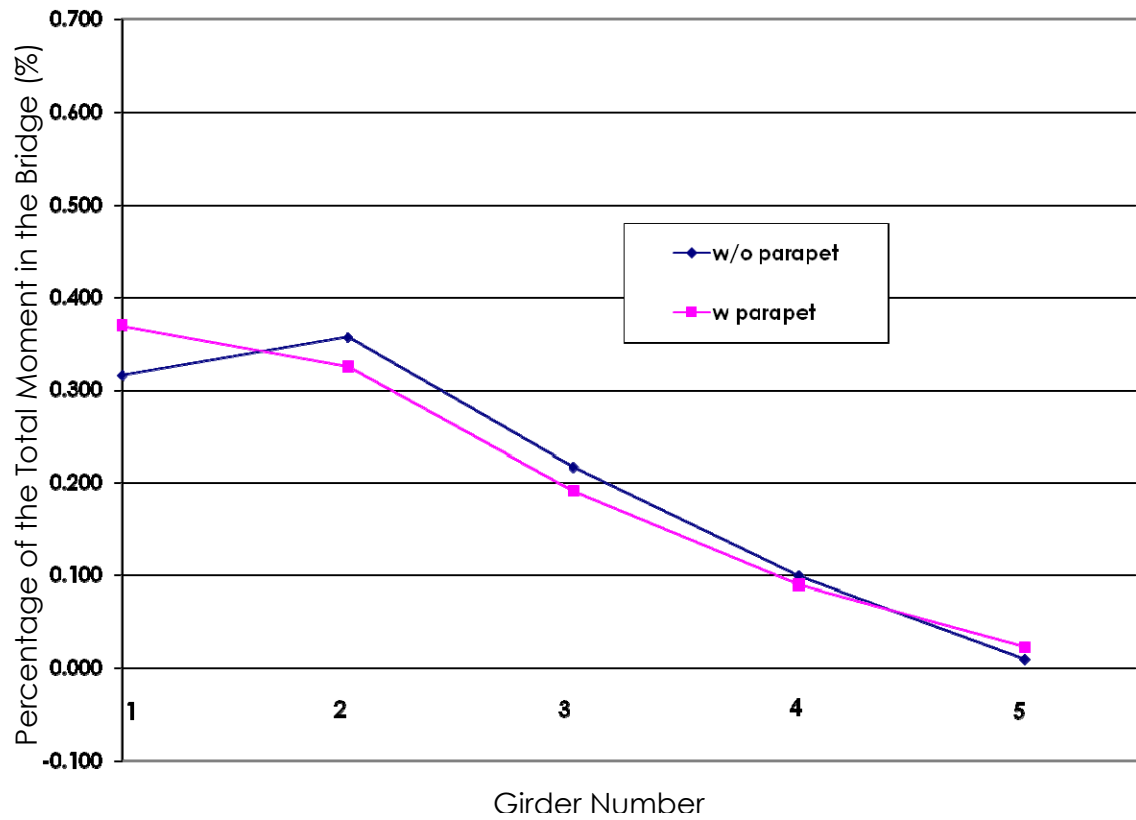


Figure 28: Comparison of girder distribution factors with the load located over girder 2 at longitudinal location C.

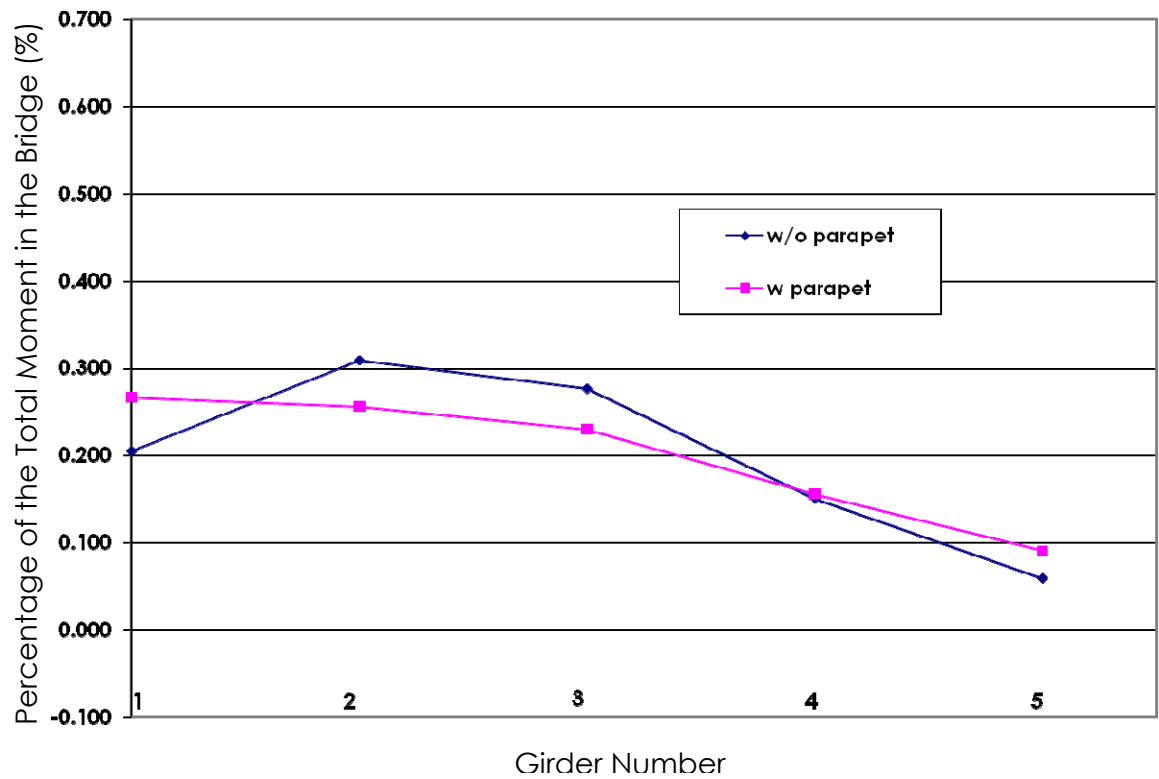


Figure 29: Comparison of girder distribution factors with the load located between girders 2 and 3 at longitudinal location C.

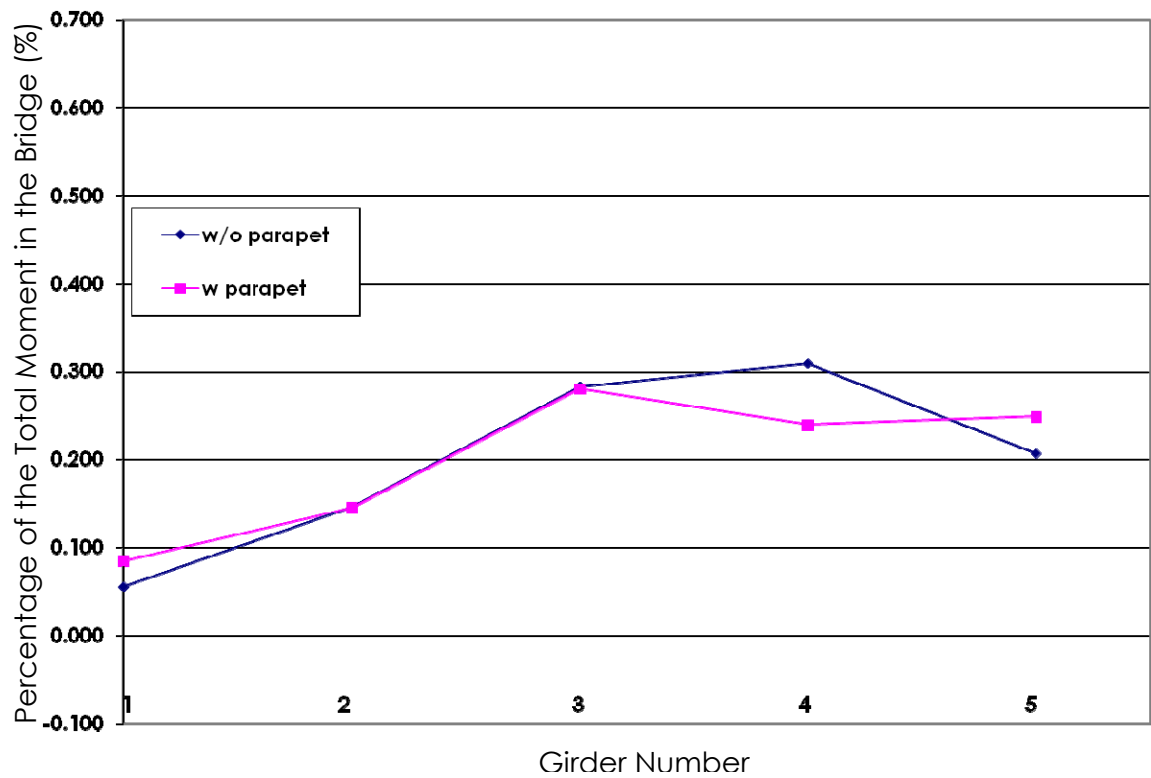


Figure 30: Comparison of girder distribution factors with the load located between girders 3 and 4 at longitudinal location C.

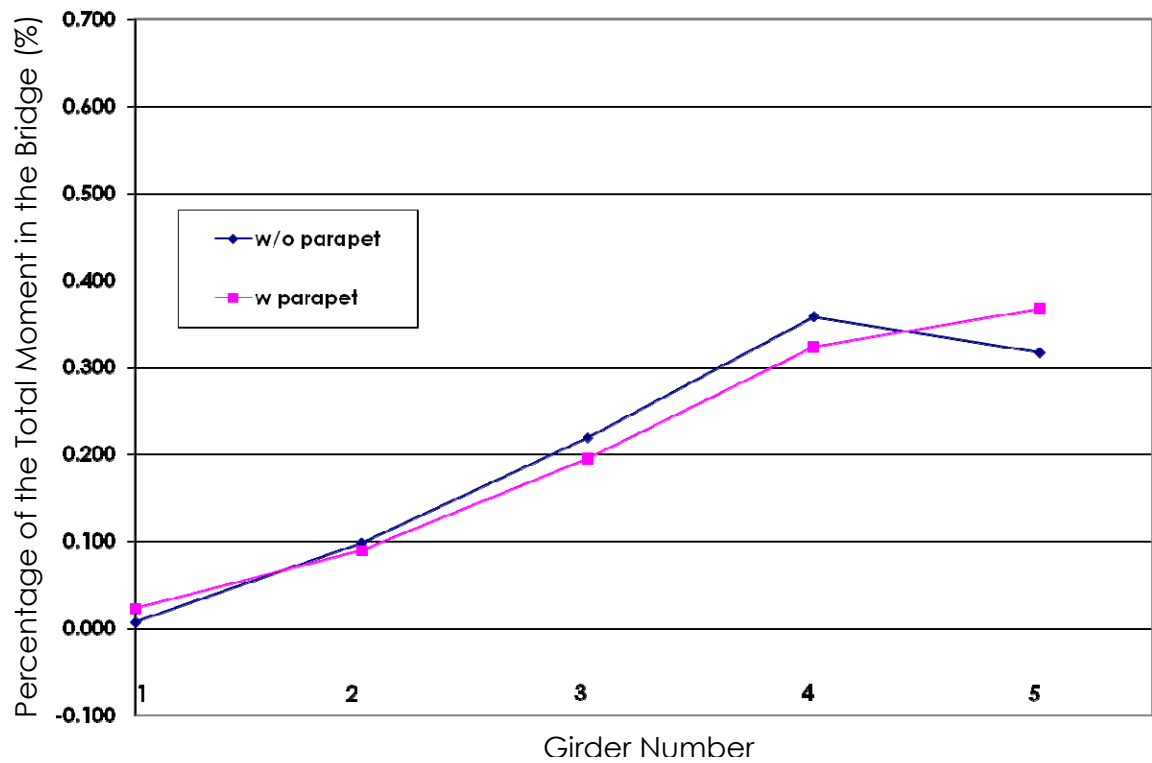


Figure 31: Comparison of girder distribution factors with the load located over girder 4 at longitudinal location C.

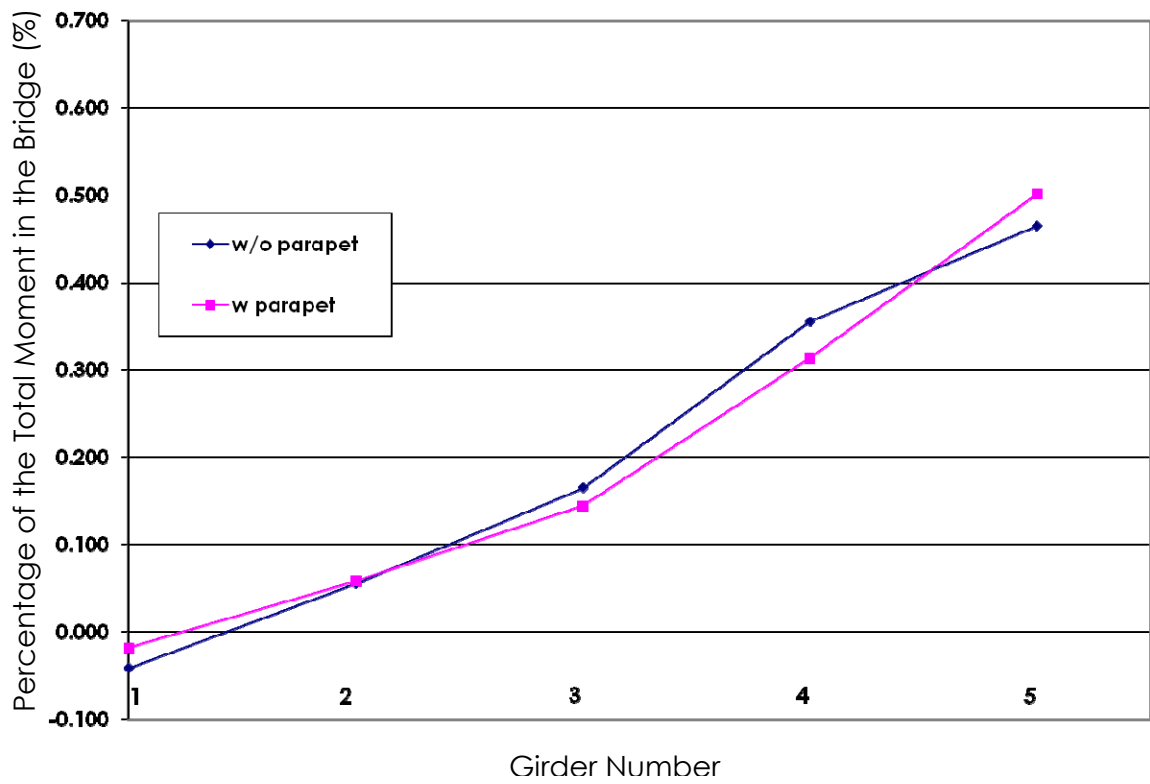


Figure 32: Comparison of girder distribution factors with the load located between girders 4 and 5 at longitudinal location C.

VITA

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