Abstract

Paoinchanatara, Nuttapone. Measurement and Simplified Modeling Method of the Non-Composite Deflections of Steel Plate Girder Bridges. (Under the direction of Emmett A. Sumner PhD., P.E.)

Many of today's bridge construction projects are erected in stages to limit traffic interruptions or to minimize environmental impacts. Typically, one half of the bridge superstructure is constructed in the first stage and the other half constructed in the second stage. The final stage is to cast a closure strip to join the deck slabs of the two structures together. Due to the inability of current analysis methods to accurately predict the non-composite dead load deflection for steel plate girder bridge during construction, three-dimensional finite element bridge models have been developed to more accurately predict the bridge behavior. However, the bridge engineer must first consider the complexity of and the time spent modeling bridges in this way.

The objective of this research is to develop a simplified modeling method to predict the non-composite dead load deflections of the steel plate bridge girder. The study is focused on creating a modeling method for both single span and two-span continuous bridges. Parameters such as bridge skew, cross-frame stiffness, and stay-in-place metal deck forms were considered in the development of the simplified models. By measuring the deflections in the field during the construction, the true non-composite deflection behavior of the bridge was captured and compared to the results from modeling.

As a result, the predicted deflection of the simplified models agreed well with the measured results for the single span bridges. Additionally, predicted deflections from the simplified models correlated well with the deflection results from finite element models for

the single span bridges. However, the predicted deflections from the simplified models had a poor agreement with the measured results for the two-span continuous bridge. It was also concluded from both field measurements and modeling results that stay-in-place metal deck forms have a significant effect to the non-composite dead load deflection behavior for skewed steel plate girder bridges.

MEASUREMENT AND SIMPLIFIED MODELING METHOD OF THE NON-COMPOSITE DEFLECTIONS OF STEEL PLATE GIRDER BRIDGES

by
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Biography

Nuttapone Paoinchantara was born on September 29, 1980, in Bangkok Thailand. After he spent five years in Suankularb School, the best high school in Thailand, he went to Chulalongkorn University and received his bachelor degree in civil engineering in Summer 2002. Inspired by his father, he desired to pursue his master degree in the major of structural engineering at North Carolina State University in spring 2004. The author is moving to Dallas, Texas to work at Lopezgarcia Group. in bridge engineering. He is currently working to complete his degree in Greensboro, North Carolina.

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I would like to thank the other two students involved in this research topic, Seth Fisher and Todd Whisenhunt whose input and related work contributed a lot in my thesis. Special thank to my CFL friends for their kindness whenever I needed them. I would also like to thank the North Carolina State Department of Transportation (NCDOT) for funding this research.

Last, I would like to thank to my family, especially Ms. Renoo Paoinchantara and Miss. Wimolwan Paoinchantara for providing me with the much needed emotional support and encouragement. Without their support, my dream to finish the master degree will never come true.

Table of Contents

List of Tables

List of Figures

MEASUREMENT AND SIMPLIFIED MODELING OF THE NON-COMPOSITE DEFLECTIONS OF STEEL PLATE GIRDER BRIDGES

Chapter 1 – Introduction

1.1 Background

The inability to accurately predict the non-composite steel girder deflections that occur during the deck construction of skewed and stage-constructed bridges is a problem that has resulted in many costly construction delays and maintenance and safety issues for the North Carolina Department of Transportation (NCDOT). The current analysis method that has been used to predict the non-composite dead load deflections incorporates single girder line model without accounting for the transverse load distribution factors that occurs due to the cross frames or stay-in-place forms. In this study, the accuracy of the single line model and three-dimensional models is investigated.

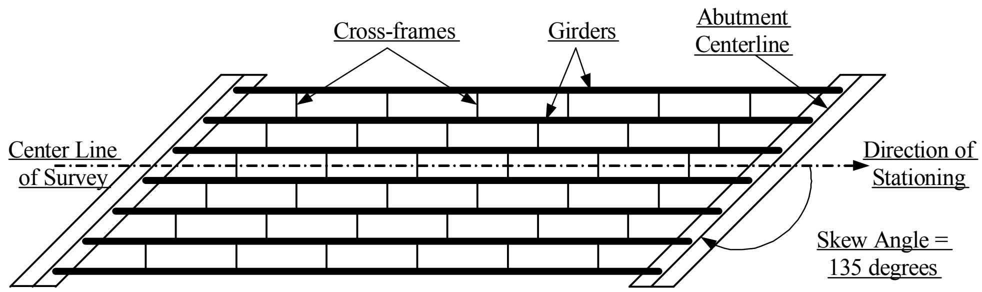
In order to predict the non-composite dead load deflection of the bridge structure more accurately, three-dimensional finite element modeling methods were developed to create bridge model by Whisenhunt (2004). By comparing the predicted results with the measured results, it was found that the results from the finite element modeling method can have a very good agreement with the measured deflections. However, the complexity of the modeling method and the time requires to operate the models is a serious concern for the design engineers. In an effort to improve the modeling method, this study involves the development of simplified modeling methods to predict the non-composite dead load deflections of the steel plate girder bridges. By using the measured results from previous research by Whisenhunt (2004) and Fisher (2005) and addition field measurement, the true deflection behavior of the bridge is captured and used to verify the modeling methods.

1.2 Terminology

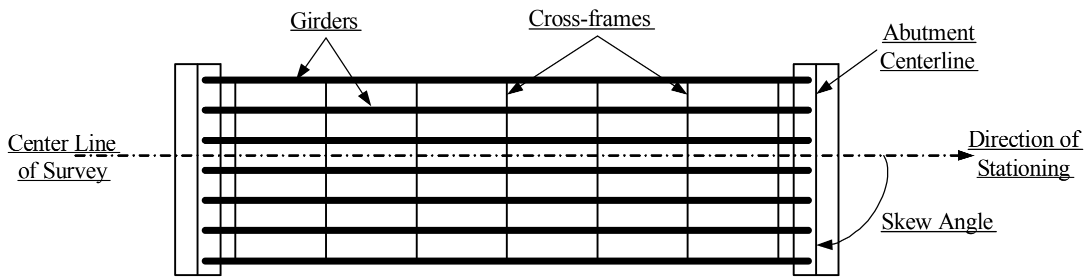
With different characteristics, the vertical deflection of several bridges were recorded the vertical deflections during construction of the bridge deck and the results compared to from the analytical models. The bridges included in this study have varying degrees of skew angle and were either a single staged construction or part of a staged construction bridge. A discussion of terms used in this study, bridge skew angle and staged construction, are included herein.

1.2.1 Skew Angle

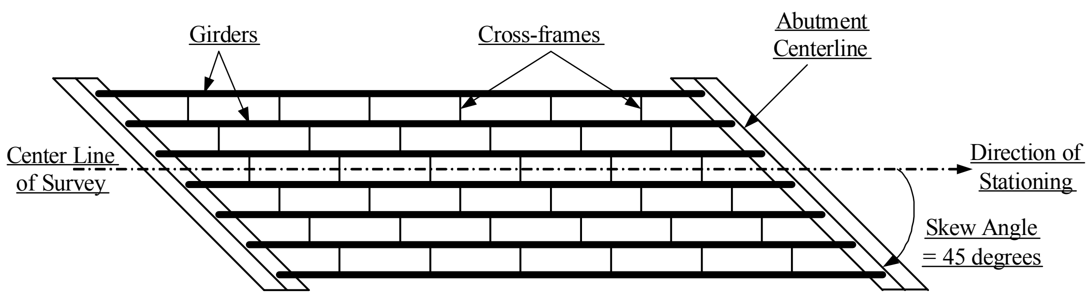
Since several bridges with different skew angles were included in this study, the method to define the skew angle of the bridge is presented. The nominal skew angle defines as the angle below the survey line (generally the survey line runs left to right in the bridge plan) measured clockwise to the supporting abutment line as shown in Figure 1-1. However, all bridges in study will be compared on an equivalent skew angle definition based on the orientation shown in Figure 1-1a. For example, if the nominal skew angle is equal to 135 degree, the equivalent skew angle is equal to 45 degree.



a) Orientation for a Skew Angle > 90 degrees
(basis for equivalent skew angle)



b) Orientation for a Skew Angle = 90 degrees



c) Orientation for a Skew Angle < 90 degrees

Figure 1-1 Skew Angle, Plan View

1.2.2 Staged Construction

Many of today's bridge construction projects are erected in stages to limit traffic interruptions or to minimize the environmental impacts. Typically, one-half of the bridge superstructures are constructed in the first stage and the other half constructed in the second stage. The final stage is to cast a closure strip to join the deck slabs of the two structures together. (see Figure 1-2)

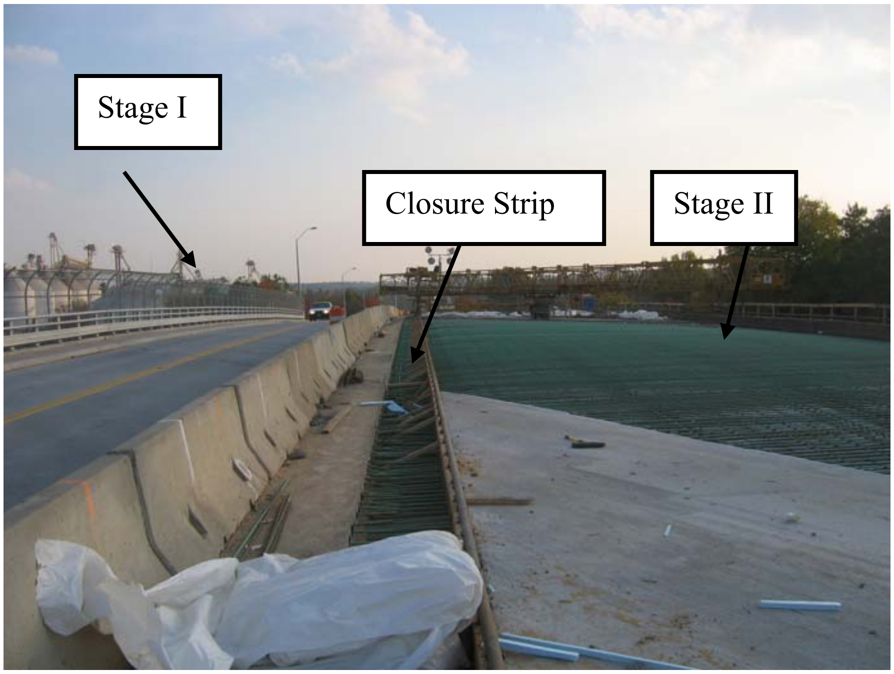


Figure 1-2 Staged Construction

In staged construction problem, the poor correlation between predicted and actual bridge deflection behavior has created numerous problems in the bridge construction industry today. By using an improper prediction method, misalignment of the concrete deck between stage I and stage II can occur. (see Figure 1-3) Construction may be delayed because concrete grinding to reduce the thickness of concrete surface or structure re-analysis may be required to solve this problem.

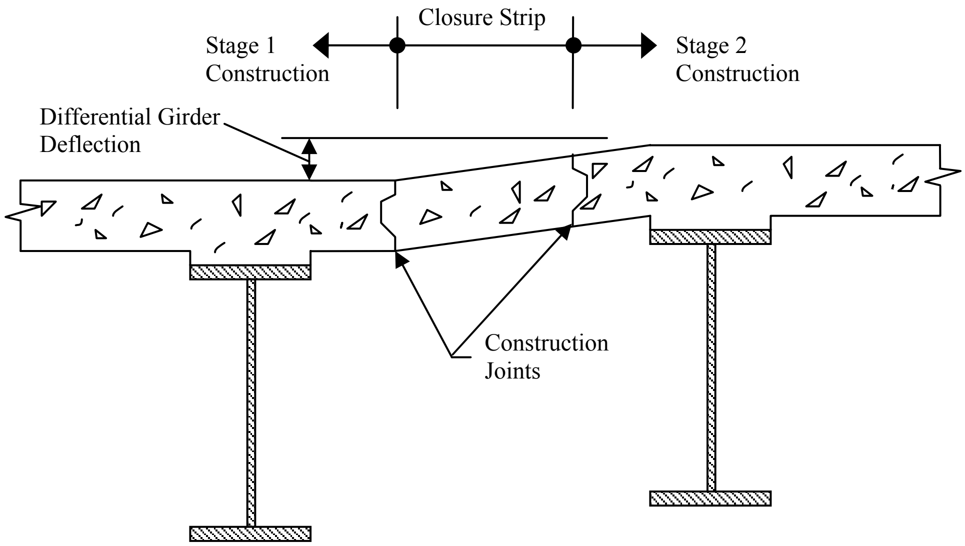


Figure 1-3 Misalignment of Deck Slab due to Differential Non-composite Deflections

1.3 Current Analysis Method and Finite Element Analysis Method

Single girder line model is the current analysis method that North Carolina Department of Transportation (NCDOT) uses to predict the non-composite deflections. The magnitudes of the non-composite deflections predicted by single girder line models are directly dependent on the amount of dead load acting on the girder. Based on the tributary area of deck slab for each girder, the non-composite deflections are equal for interior girders (assuming equal girder spacing) but vary according to the overhang conditions for the exterior girders. Without any concern of the load transferring factor such as cross frame or stay-in-place metal deck forms (SIP forms), the predicted results and measured results are markedly different as illustrate in Figure 1-4.

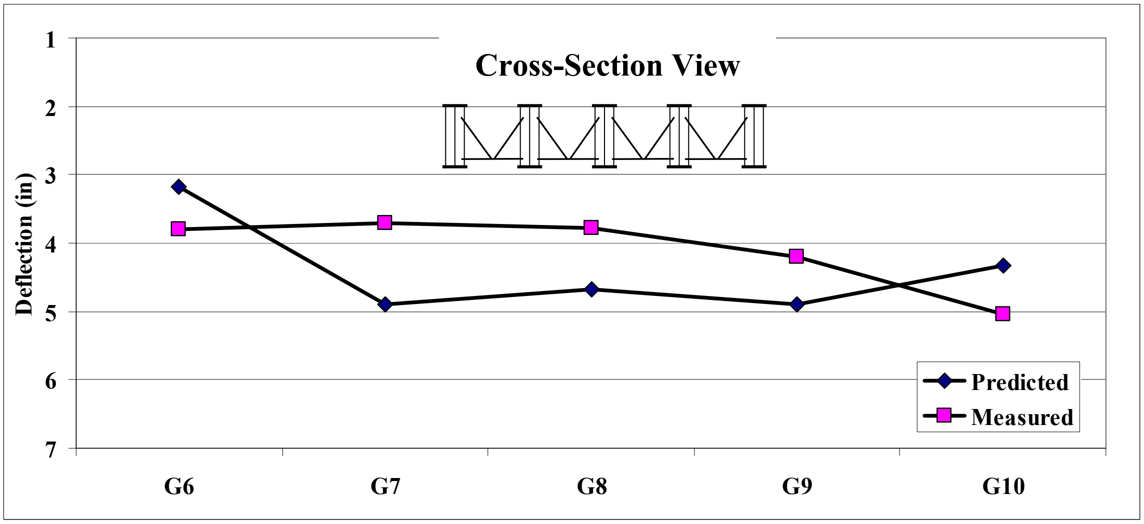


Figure 1-4 Deflections Shapes Compared between Single Girder Line Model and Measured Results at Mid Span of Wilmington Street Bridge

Since it was revealed that the single girder line analysis is not effective, NCDOT fund the project, "Developing a simplified Method for predicting deflections of the steel plate girder under non-composite load for stage-constructed bridges" to North Carolina State University. Whisenhunt (2004), the student who involved in the project, presented the finite element modeling method to predict the non-composite deflection in his study. By using the finite element analysis code, ANSYS, he created the three-dimensional finite element models of the complete bridge structures. (see Figure 1-5)

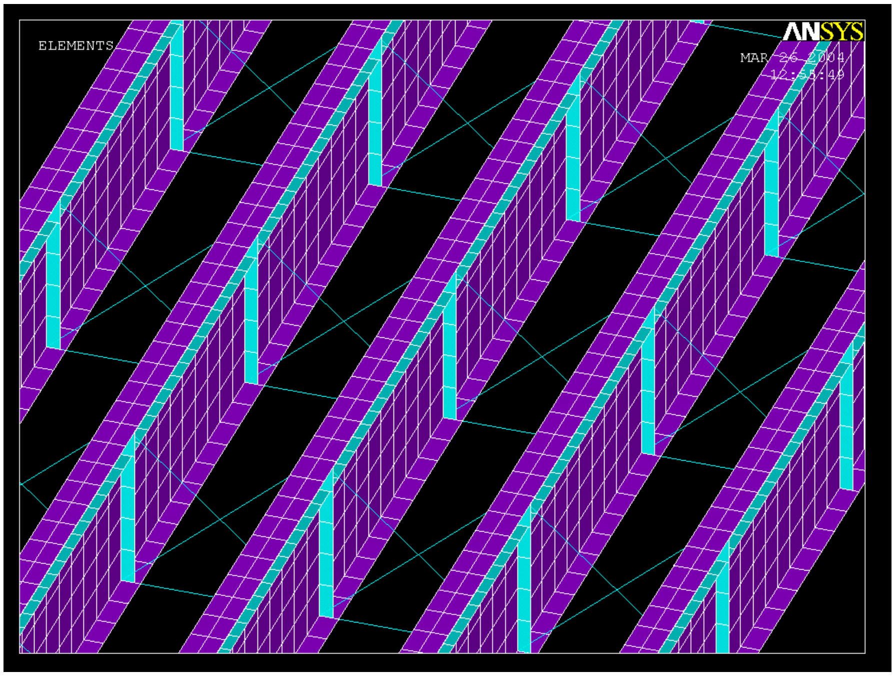


Figure 1-5 ANSYS Finite Element Model of Eno River Bridge

By including many parameters that might have effects to the bridge behavior such as cross frames, SIP forms, stiffener plates and concrete decks, the predicted results have a very good agreement with the measured results. However, it is apparent that the finite element bridge modeling has many details and can be very complicate.

1.4 Objective and Scope of Research

This study includes the field measurement of girder deflections during the concrete deck placement of four of single span and one of two spans continuous, steel plate girder bridges. In addition, it includes the results of the analytical portion of the research that involved the development of simplified models for each of the bridges and the correlation of these models to the field measured deflections.

The objective of this research is to develop a simple modeling method by using SAP 2000 to predict the non-composite dead load deflection of steel plate girders. The effects of skew, girder length and size, girder spacing, cross frame and stay-in-place form will be considered. Modeling will be focused on single span and continuous span bridge configurations. Instead of creating the three-dimensional finite element computer model that is more complicated and time consuming, the simplified modeling method will provide the user with an efficient and simple bridge modeling method to predict the non-composite girder deflections.

1.5 Outline of Thesis

The following is a brief outline of the major topics covered in this thesis:

- Chapter 2 contains a summary of the previous research related to this topic. Several reports about bridge construction problems and solutions are included. The summary of previous research studied the effects of each component of steel plate girder bridge and details about bridge modeling by several software programs are aslo included.
- Chapter 3 contains a discussion of the parameters affected to bridge behavior and details of each bridge in this study. This chapter also includes the details of two different field measurement methods used in this study.
- Chapter 4 outlines the development of the simplified modeling methods and discusses how to create each bridge component in SAP 2000. A summary of the results predicted by SAP simplified modeling methods is included.
- Chapter 5 contains a discussion of the details of the comparisons between the deflection behavior of steel plate girder bridges predicted using single girder model and the actual bridge behavior observed from the field. In addition, a comparison

between deflections predicted by SAP simplified models, ANSYS results and measured results is included.

- Chapter 6 contains observations and recommendations made during this research. A list of the conclusions drawn from this study is also included in this chapter.

Chapter 2 - Literature Review

2.1 Overview

A summary of the previous research related to the design, analysis and construction of steel plate girder bridges is presented in this chapter. Detail of the parameters, which affect behavior of bridges such as diaphragms, cross frames, skew angle, and stay-in-place metal deck forms are discussed herein. Because of the complex behavior of the bridge structures, the majority of the past research has focused on creating model analysis to represent the bridge behavior. Several modeling techniques have been revealed and verified. A summary of these techniques is also presented in this chapter.

2.2 Construction Problems

The inability to accurately predict the steel girder deflections that occur during the deck construction of skewed and stage-constructed bridges is a problem that has resulted in many costly construction delays and increase maintenance costs. For DOT's throughout the nation, these construction problems have been reported by several researchers. Design guidelines recommended by the researchers and AASHTO are summarized herein.

The predicted vertical deflections included in the bridge girder camber tables are normally calculated using a single line model. The single line model assumed that the girders are independent of the adjacent girders. The deflection is calculated using the tributary width proportioned to each girder. Due to the connection between each girder by the cross-frame and permanent metal deck form, the bridge behaves as a system instead of individual components. This behavior is contrary to the assumption of the single girder line model. This behavior has been observed for many years.. Hilton (1972) measured the vertical deflection of the simple skewed bridge to better understanding the parameters affecting the bridge behavior. Hilton observed that the field-measured deflection was approximately fifty percent lower than predicted.

To develop methods for the successful design and construction of the composite steel plate bridge, Swett (1998) and Swett et al (2000) collected the data related to several bridge construction issues and the recommended solutions from several states. They performed a finite element analysis of the bridge structures to better understand the bridge behavior. The goal of the research was to develop new construction techniques to achieve the correct vertical alignment of the girder. They mentioned two case studies in Washington State. One of the cases showed the differential elevation of the bridge deck construction between stage I and stage II construction, which led to construction delay. After completion of the concrete deck in Stage II, the vertical elevation of the girder was not as predicted. Since there are no diaphragms or cross frames between Stage I and Stage II, the girders in Stage II rotated away from girders in Stage I and caused a 90 mm differential elevation of the deck at the mid span. The contractor moved the temporary barrier from stage I to stage II to increase the dead load in stage II and correct the difference in elevation. The construction was delayed due to the misalignment of bolt hole in the stiffener plates. The contractor solved the problem by welding intermediate diaphragms near the mid span connected between Stage I and Stage II instead of connecting by high strength bolts. After summarize the solution from the surveys and using the finite element analysis, they recommended six construction techniques for staged bridge construction. First three methods involve the connection of the two stages with the cross-frame before the deck placement in stage II. They stated that these methods can prevent the twisting and the lateral movement of the girder in stage II. Other methods recommended maintaining the independence between two stages while the second deck is cast by not installing the cross frame. They also stated that the structures can be torsionally stiffened by using one of following: a balanced deck placement, two pour sequencing in the stage II or the addition of the lateral members on the stage II structure. Swett and Swett et al concluded that all methods are suitable to reduce the differential elevation between two stages provided that the appropriate one is chosen in each project.

AASHTO (2002) disclosed the rotation of the girder due to the skewed piers and abutments. The girder rotation will be normal to the pier or abutment in skewed support bridge. This rotation displaces the top flange transversely from the bottom flange and causes the web to be out-of-pump. The special consideration required the design of cross-frame connection to accommodate the increased stress caused by differential deflection between the adjacent girders.

Norton (2001) and Norton et al. (2003) monitored vertical, lateral deflection and rotation of a seventy-four meter, fifty-five degrees skewed steel plate girder bridge. They compared the measurement results to three-dimensional finite element modeling results. They noted that measured vertical displacements were close to the predicted. However, the lateral movements were not in a good agreement and the rotation of the girders was about 50% lower than the predicted.

2.3 Parameters Affecting Girder Deflections during Deck Construction

As mentioned in the previous section, designers and erectors have had problems related to the non-composite girder deflection during the bridge construction. The primary conclusion is that the predicted non-composite dead load girder deflection is significantly different that the observed field behavior. Researchers identified several parameters as primary parameters as major effects of the bridge behavior: bridge configuration, diaphragm and cross-frame, skew angle, stay-in-place metal deck form and etc. A summary of the past research studies related to parameters included in this study is presented in the subsequent section.

2.3.1 Diaphragms and Cross-Frames

Hilton (1972) monitored deflections of the steel bridge girders during the construction to study factors affecting the deflections during the construction. He measured the deflections from two of the simple span bridges to compare with the theoretical analysis. He concluded that girders of the bridge obviously behave as a unit more than independent girder due to the interconnecting diaphragm action.

Elmir (1988) investigated the spatial interconnection behavior between the cross-frames and end bent diaphragms with the main girders of simply support multi-girder skewed steel bridges. The method of analysis used was the three-dimensional finite element method. . He created models with different parameters such as skewed angle varied from non-skewed to sixty degrees and types of cross frame varied from actual, heavier and lighter. As a result, Elmir concluded that vertical deflection behavior of bridges is not dependent on the cross frame section. However, the lateral movement decreased when the cross frame cross section was increased.

Khaloo and Mirzbozorg (2003) studied the effects of the arrangement of internal transverse diaphragms on the behavior of bridges. They created three-dimensional finite element models of the simply supported bridge models with different skew angles and arrangements of the diaphragms as followed: no diaphragm, diaphragms parallel to the supporting lines, and diaphragms perpendicular to the longitudinal girder line. To compare the results between each model, they used the distribution factor that is the ratio between bending moment in the composite action and bending moment due to one line of wheel load. As a result, the arrangement of the transverse diaphragm has a significant effect to the load distribution factor related to the skew angle. In general, the distribution factor of the exterior girder decreased when the skew angle is increased for all arrangements of the cross-frames. They concluded that the deck with internal diaphragms perpendicular to the longitudinal girder line is the best arrangement for load distribution in skew bridge.

Huang et al. (2004) studied the effects of transverse diaphragms and diaphragm stiffness on the bridge behavior using three-dimensional finite element modeling techniques. From three case studies: stiffness calculated from the real cross frame, half of the stiffness from the calculated stiffness and no stiffness, they concluded that diaphragm stiffness has significant effects to the bridge behavior. The diaphragm acted to distribute the load by transferring load to the adjacent girder. In addition, because of the load transfer, the strain at the pinned end of the middle girders was reduced. When the stiffness of the diaphragms was reduced, the strain at the pinned of the exterior girder was decreased due to less transferring of the load.

Whisenhunt (2004) used ANSYS finite element code to create three-dimensional finite element models to investigate the effect of the type of cross-frame to the deflection behavior of the steel plate girder bridge. From the results of bridge models with X-type and

K-type of cross-frames, he concluded that different configuration of cross-frame has no significant effect to the bridge behavior.

2.3.2 Skew Angle

Gupta and Kumar (1983) studied the effect of skew angle to the bridge behavior. By creating the analytical bridge models with different skew angles from non skewed to forty degrees. They concluded as follows: the skewed angle up to thirty degree has slightly effect to the girder deflection, while the deflection increased rapidly when the skew angle was above thirty degrees; The maximum bending moment was significantly different between non skewed and ten degrees skewed angle bridge while the difference between ten degrees and forty degrees bridge was very small. They also stated that the designer does not have to be concerned about the skew angle up to thirty degrees but a careful analysis is needed for bridges with a skew above 30 degrees.

Bishara and Elmir (1990) performed a parametric study to investigate the effects of the bridge skew angle. They created three-dimensional finite element models with skew angles of zero, twenty, forty and sixty degrees to investigate the internal forces in the cross-frames. From the investigation in skewed bridge, they concluded that members of intermediate cross-frame might develop compressive or tensile forces depending on the load condition. They also stated that the effect of the skew less than 20 degrees can be neglected and the change in cross section of cross-frame member has no effect on the vertical deflection behavior of the bridges.

Ebeido and Kennedy (1996) derived the ASSHTO design method for the skewed bridge by using results from three-dimensional finite element models. To investigate the effects of the skew angle, they compare the analytical modeling results between non-skewed and forty-five degrees skewed bridge. The results showed that the moment at the intermediate support and the moment along the span decreased significantly with the increase of skew angle. They concluded that the effects of skew angle became greater when the angle was higher than thirty degrees.

Khaloo and Mirzbozorg (2003) created three-dimensional finite element models varying the skew angle of a simply supported bridge; zero, thirty, forty-five and sixty-degree bridge were considered. They concluded that the skewed angle is the most significant factor affecting the load distribution in their parametric study. The analytical result from the modeling showed that load distribution factor normally decreased when the skew angle increased. The result also showed that the bridges were not sensitive to the skew angle up to thirty degrees but became highly sensitive when the skew angle was over thirty degrees.

2.3.3 Stay-in-Place Metal Deck Form (SIP)

To understand the lateral bracing behavior of the stay-in-place deck forms (SIP), Jetann et al. (2002) measured the bracing behavior of the deck form with the existing and an improved connection between the girders and the forms. The research was conducted by testing the panel in a constructed frame with the different aspect ratio. They presented the ultimate shear strength and shear rigidity of the SIP with standard and improved connection between the girder flange and SIP forms.

Egilmez et al. (2003) continued Jetann's work by further investigation the lateral bracing behavior of the SIP forms. In addition, he developed an improved connection between the deck form and the steel girder flange. They conducted the laboratory experiments and compared the results to the finite element analysis result. This comparison was used to develop the modeling method for the shear strength of the different connections metal deck form. They modeled two girders braced with deck forms and compared the results to the results from Jetann's work (2002). Egilmez presented his results, recommended an improved connection for better shear strength, and provided detail of his finite element modeling method which produced accurate results.

Whisenhunt (2004) studied the effect of SIP form to the bridge behavior. He created the three-dimensional finite element models of five steel plate girder bridges with and without SIP. The results showed that SIP form had a significant effect to the deflection behavior of the skewed bridges but no effect to the non-skewed bridges. He explained that SIP form has no effect to the non-skewed bridge because there is no significant out-of-plane girder rotation when the deflection occurred. However, skewed bridge developed the out-of-plane girder rotation. The rotated top flange of girders activated the force transmission behavior of SIP form. Whisenhunt recommended that SIP form should be included in skewed bridge model for more accurate results.

2.4 Bridge Modeling

For the better understanding of bridge behavior, several research programs related to the bridge modeling were conducted. Two types of bridge modeling methods are the majority of this topic that have been developed, two-dimensional grillage modeling method and three-dimensional finite element modeling method. A summary of the previous work related to the bridge modeling is presented in the following sections.

2.4.1 2-Dimensional Grillage Model

To study the effect of skewed angles to the bridge behavior, Gupta and Kumar (1983) created five analytical models and compared with field measured results. They used the combination of two separate programs to simulate the grid framework and plate behavior to create grillage bridge models. Each grillage model has three simple beam elements as longitudinal girders and another six beam elements as transverse girders. Concrete slabs were created by using plate elements. After using stiffness method combined with the finite element to analyze the grid framework, they found that the results from the models had a good agreement with the result from the testing.

Swett (1998) and Swett et al. (2000) started their modeling methods by creating two-dimensional grillage models to develop bridge construction techniques. SAP 90 and SAP 2000 were used to create the two-dimensional models. By following the modeling methods of Heins (1975), they used frame element for the longitudinal girder; diaphragm member was transform into an equivalent plate and modeled as frame element. However, they noted that the grillage model could not capture the warping torsional stiffness exhibited by both the girders individually.

Bangash (1999) mentioned the grillage analysis of bridges in his book. He explained that the bridge deck is represented by the grillage of interconnected beam under loads. The concrete deck is modeled as a plane or three-dimensional grid work of discrete interconnected beam. The three-dimension girder can be transferred to two-dimension by using direct stiffness method or flexibility method.

To study the behavior of the skewed bridge and evaluate the adequacy of modeling method, Norton (2001) and Norton at al. (2003) monitored seventy-two meter, skewed steel plate girder bridge to compared the results with the analytical modeling results. Two levels of models were created; two-dimensional grillage model by using STADD and three-dimensional finite element model by using SAP 2000. For two-dimensional modeling, the girder was modeled as a frame element. Cross frames were also transformed into frame elements. Progress of concrete during the construction was divided into four steps modeled as four uniform load stages. Exterior girders were also subjected to out of plane moments simulating the load from the overhangs. They also include loads from the deck screeding machine in the models by applying point loads to the girders. For the boundary conditions, vertical, lateral and longitudinal displacements were restrained as a pinned support in one side and only vertical and lateral displacement were restrained in the other side as roller support.

2.4.2 Three-dimensional Finite Element Modeling

Bishara and Elmir (1990) studied the interaction between cross-frames and girders. ADINA finite element code was used to create a model of a one hundred thirty seven foot, simply supported steel bridge with varying skew angles (no skew, twenty, forty and sixty degrees). They used beam elements for girder sections and cross frames. Discretized triangle plate elements were used for concrete slab. The plate elements were connected to the beam elements using rigid link elements. They gave the key conclusions as follow; most cross-frame members developed internal tensile and compressive force; higher skewed angle produced higher maximum force in the cross frames; skewed effect can be neglected when the angle less than 20 degrees; larger cross-frame members produced larger internal force and cross-frame members does not affect to the vertical deflection of the bridge girder.

To verify the wheel load distribution factor of steel bridge girder presented by AASHTO and NCHRO, Fredrick et al. (1997) developed four of the finite element modeling methods. He compared the modeling results to the results from AASHTO and NCHRP method. They chose 56x30 feet simple span bridge super structure supported by four of W36x160 steel girders spaced every 8 feet for their study case. SAP 90 program was used to create first three modeling techniques. For the first modeling method, they created concrete slab by using quadrilateral shell elements (five degrees of freedom) and steel girders by using space frame members. The centroid of each girder was at the same location of the centroid of the concrete slab. Second, same types of element were used for the slab deck and girders but the location of the frame member was changed. Shell elements were eccentrically connected to the frame elements by using rigid link elements. Third, both girder webs and concrete slab were created by using shell elements, while frame elements were used for the girder flange that connected to the shell element by rigid link element frame element. ICES-STRUDL II program was used to create the fourth type of the modeling. Isotropic solid elements (eight degrees of freedom) were used for the concrete slab, while steel girder web and flange were created by shell element. As a result, they noted that the results from the first and the fourth modeling techniques correlated well with the result from NCHRP equation, while the results from AASHTO equation was conservative compared to the analytical results.

Frederick et at. (1999) created parametric studies which emphasis the effects of the span length and girder spacing compared to the load distribution factor between AASHTO, NCHRP calculation methods and the analytical results from three-dimensional finite element models. SAP 90 program was used to create the finite element model in their study. They used shell elements for the concrete slab and space-frame members for the steel girders. As a result, they concluded that in the shorter span length and spacing, AASHTO calculation method is less conservative compared to NCHRP calculation method but become more conservative when length and spacing of the girder increase.

Swett (1998) and Swett et al. (2000) used SAP 2000 program to create three-dimensional analytical models to develop new bridge construction techniques. Frame elements were used to model the top and bottom flange of the steel plate girder, while shell element were used for web of the girder. Diaphragm and lateral members were modeled by connecting the frame elements to the nodes at the flange-web interface. The modeling method was verified by comparing the results with experiment testing results. They noted that the results from the modeling were in the acceptable range compared with the experimental results.

Samaan et al. (2002) studied the effects of the length, number of lanes, number of girders, and number of cross bracings of the steel bridges on the stresses, maximum reaction and shear in the bridge subjected to various loading by using three-dimensional finite element modeling. ABQUS software program was used to create three-dimensional finite element model of the continuous span bridge. They used four-node shell elements for concrete slab, steel web, steel bottom flange and end bent diaphragm, while beam element was used to create top steel flange, cross-frame bracing and top chord. They connected shell elements as the concrete slab to the beam elements as top steel flange by using multi-point constrains option of the program. Vertical and lateral displacement were restrained at the bottom end nodes of the web as supports on both sides of girders, whereas all displacements were restrained at the bottom end node of the web in the middle hinge of the continuous span bridge. The modeling method was verified by comparing the result with the results from experiment. Kennedy noted that the results from modeling had a good agreement with the experimental results.

Wang and Helwig (2003) studied the effects of cross-frame and diaphragm to skewed bridge behavior. To establish the calculation method for stiffness and strength of the cross frame for bridge structures, they created three-dimensional finite element models with different characteristics and compared the moment from the models to the moment from their equation. Eight-node shell elements were used to create cross section and stiffener of the girder. As a result, Helwig and Wang were able to prove that their derived equations had a good agreement with the result from the modeling.

Norton (2003) created three-dimensional finite element model using shell elements for the girder web and concrete deck whereas frame elements were used for the girder flange, stiffener and cross frame. The cross-frames were connected to the girder web rigidly and boundary conditions were assigned similar to her two-dimensional model. To represent the real bridge structure, shell elements that represented the concrete deck were connected to the frame elements, which represented the girder top flanges, using rigid link elements. As a result, vertical displacement from three-dimensional model was 0.3-10 percent higher than the field data compared to 25-38 percent from two-dimensional modeling results and the average ratio of predicted of the three-dimensional to measured is only 1.2.

To develop understanding of the transverse load distribution of the highly skewed bridge, Chajes et al. (2004) created three-dimensional bridge models to compare with testing results and results from AASHTO method. Two-span, continuous, slab-on-steel composite high way bridge with sixty-degree angle was the case study of this research. Strains at the various locations along the girders were measure from the field to compare with the analytical results from ANSYS modeling. To create the models, they used four-node, three-dimensional elastic shell elements with six degrees of freedom for the concrete deck. The girder was modeled using two-node three-dimensional elastic beam elements with six degrees of freedom at each node. The cross-frames were modeled as a beam element. For supports at the end of the girder, they used pinned support in one side and a roller in the other side, while pinned supports were also used in the middle as the intermediate pier support. Composite action between girders and slab deck were represented by using constraint equations between degrees of freedom of shell and beam elements. The overhang and parapet were not included in the model. As a result, they stated that the three-dimensional finite element models were in a good agreement with the experimental testing results that proved to be very informative in understanding and predicting the behaviors of highly skewed bridges.

Padur (2004) created the bridge modeler, Graphical Use Interface (GUI), to create three-dimensional finite element models in SAP90 program of straight concrete-deck-on-steel-stringer bridges. By using Microsoft Virtual Basic create input files, the stringers are constructed with of shell elements for the girder web, and frame elements for the girder top and bottom flange. Frame elements were also used to model the cross-frame bracing. The concrete deck was created using two layers of shell elements connected by rigid link elements. The concrete deck was connected to the girders using rigid link elements. The end supports were modeled as vertical springs and restrained the lateral movement. His modeling technique was verified by comparing with the experimental results. The modeling results had a good agreement with the experimental result.

Whisenhunt (2004) developed the analysis finite element modeling method to predict the non-composite dead load deflection during the bridge construction. ANSYS program was used to create the three-dimensional finite element models. SHELL93 elements were used for the steel plate girders and stiffener plates and three-dimensional (LINK8) truss elements were used for the intermediate cross frames. Stay-in-place metal deck forms were also included in the model by using LINK8 elements to create an in-plane truss that was connected to the nodes on the girder flanges coupling all of the translational DOF's (global x, y, and z directions) between the edges of the top flanges. The models were verified by comparing the results with the results from the field measurement. Whisenhunt concluded that the result from his modeling method has a good agreement with the result from the experiment.

Choo et al. (2005) studied the effect of environmental, material and deck placement to the continuous, skewed, high performance steel bridge behavior during the construction. They measured the deflection of the bridge during the construction compared to the three-dimensional finite element models. SAP 2000 was used to create the models in this study. The bridge model consists of the girder and wet concrete modeled by shell element (three or four nodes shell element). They explained the reason for using shell element because of the better concrete load distribution. The shell element was defined as a thin plate which neglected shear transverse deformation. Rigid link element was used to connect between girders and the deck to maintain the compatibility. The frame element created as rigid link was established by using the large modulus and material density was assigned which would not affect to the over all of the performance of the bridge model. For the boundary condition, the comparison between pin and roller support as bridge abutments was conducted. As a result, roller support appeared to be more accurate during the construction compared to the result from the field measurement, while the pin support provided more accurate prediction at the completed deck placement. However, from the investigation, there was no significant difference between both modeling methods.

By recording the deflection related to the progression of pouring sequence, Choo et al. (2005) studied the effect of composite action between steel bar and concrete. The finite element models were conducted by changing the concrete modulus of concrete related to time, and then compare the results to the deflections from the field data. As a result, Choo concluded that the development of composite action between concrete and steel bar during the construction has a small beneficial to the model only three percents.

Chung and Sotelino (2005) developed finite element models for composite steel bridge girder to improve the concrete material model. By using ABAQUS software program to create composite steel bridge system, they used three-node beam Timoshenko elements (B32 element) as the steel girder section connected to shear-flexible eight nodes shell elements (S8R elements) as the concrete deck. To assure the full composite action between girders and concrete deck, rigid links (ABAQUS MPC) were used to connect from the centroid of the girder section to the centroid of concrete deck. Shell element was created using layer approached to more accurately represent concrete behaviors such as crack propagation, dowel action, and tension stiffening. To verify the modeling methods, the modeling results was compared to the results from experimental testing. The predicted and measured vertical deflections were very close, within nine percent difference.

2.5 Need for Research

AASHTO 2002 guideline for design for construction states, “If the differential deflection is significant, the webs of the girders will rotate in a transverse direction. The webs could either be erected in the vertical position and rotate out-of-plump after the dead load is applied, or be erected out of plump and rotate to a vertical position after the dead load applied." From this statement, it obviously shows the inconsistent of the bridge design that cause several construction problems as mentioned in this chapter.

Since the bridge construction problems were investigated for many years, several past research were conducted in an effort to develop a better understanding of the bridge behavior. Many parameters, such as cross frame, skew angle, stay-in-place form and etc., were found to be factors affecting the bridge behavior. The analysis by creating the bridge model is one of the major topics that has been researched related to the bridge behavior study. Several modeling methods, such as two-dimensional grillage modeling method and three-dimensional finite element modeling method were created. Research has shown that three-dimensional finite element modeling method is the most successful modeling method. Results from finite element modeling analysis helped engineers to have a better understanding of the bridge behavior. In addition, engineers may use the finite element analysis to predict the bridge behavior more accurately.

However, three-dimensional finite element analysis of a complete bridge structure can be complex and time-consuming. Due to the complex nature of the finite element analysis code, this method has not gained widespread used for the design and analysis in the bridge industry. Therefore, there is an obvious need to develop a simplified modeling method that can be implemented using commonly available commercial software.

Chapter 3 – Field Measurement Procedure and Results

3.1 Overview

Field measurement of the vertical non-composite dead load deflections during the concrete deck placement of the steel plate girder bridges as conducted as a part of this research. Measurements from several bridge structures of varying geometry were necessary to obtain a correlation between the analytical modeling results and the measured field deflection data. Therefore, the bridge structures included in this study have varying girder length, girder spacing, skew angle, cross frame configuration and stay-in-place deck form properties. A summary of parameter characteristics are included in this chapter.

Vertical deflections from previous research by Whisenhunt (2004) and recent research by Fisher (2005) are included in this study. In addition, another field measurement was taken at the Wilmington Street Bridge in this research. Deflections of each bridge were typically recorded at quarter, mid and three quarter points along five of the steel plate girders. Bearing settlement was also measured and subtracted from the deflection values obtained along the girder lengths. The example of locations of the measured point is shown in Figure 3-1. The bridge descriptions and the summary of the field measured results are included herein.

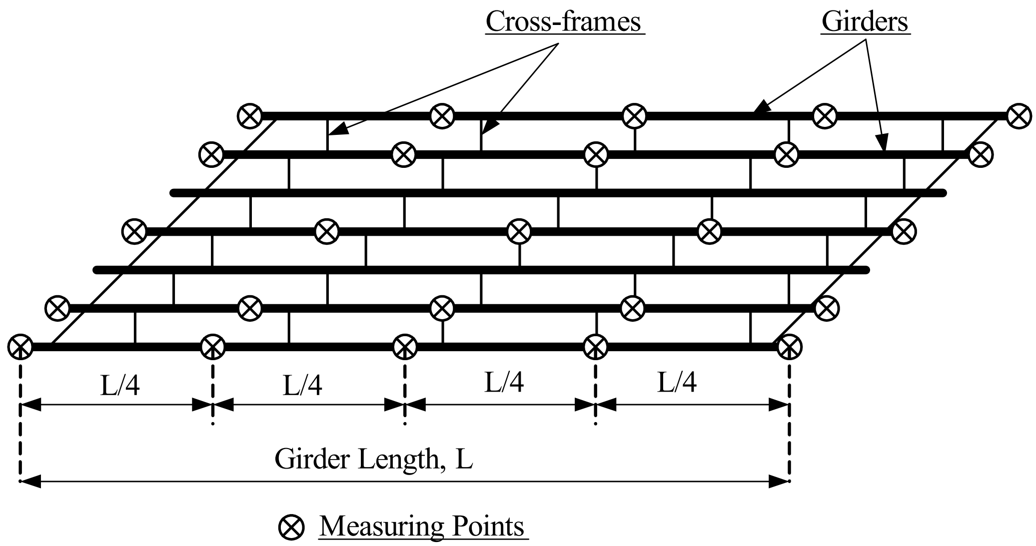


Figure 3-1 Typical Deflection Measurement Locations of a Single Span Bridge

3.2 Parameter Characteristics

In bridge systems, there are several parameters directly related to steel plate girder that affect the vertical deflection behavior. These parameters include girder length, spacing between girders, girder depth, and cross section properties. Concrete deck properties such as deck thickness and density of concrete can also have significant effects to the deflection behavior of bridges. In addition, there are some parameters revealed recently that may influence to the bridge behavior, for example, the cross-frame and diaphragm configuration, skew angle, and the stay-in-place metal deck form properties. A discussion of these secondary parameters is included herein.

3.2.1 Cross-Frames and Diaphragms

The intermediate and end-bent cross-frames are the important components that affecting the vertical deflection of the bridge. The main propose of having cross-frames in the bridge system is to prevent the lateral buckling of the girder and prevent the out-of-plane rotation of girder during construction. However, it was reported by several previous researchers that the cross-frame and diaphragm can affect to the composite load distribution. Load can transfer from girder to the adjacent girder due to the connection of cross-frame. It was concluded that the bridge system likely behaves as a unit structure (Hilton 1972). Therefore, the type and the stiffness of the cross-frame is believed to be a factor affecting the vertical deflection behavior of the bridge system.

Two types of cross-frames have been found that used in the bridge structures, X type, K type cross frames as illustrated in the Figure 3.1. The term cross frame is used for both intermediate cross frames within the span and diaphragms at the supports. Cross frames are typically connected by high strength bolts or welding to a connector plate, which is welded to girder web and flange. Normally, angle are used for the intermediate cross frame, and a combination of channel and structure tee sections are used for the end bent diaphragms as shown in Figure 3-2.

a) K Type Cross-Frame

b) X Type Cross-Frame

Figure 3-2 K-Type and X-Type of Cross Frame

3.2.2 Skew Angle

The effects of skew angle on bridge behavior have been investigated for many years. It has been shown from previous research of Gupta and Kumar (1983) and Bishara and Elmir (1990) that a bridge skew angle more than twenty degrees has significant effects to the load distribution along the girder. In addition, the vertical deflection behavior in a skewed bridge is different from the non-skewed bridge, which leads to the unpredictable deflection under the non-composite dead load. However, the traditional calculation method, single girder line model, for the non-composite deflection does not account for the effects of the skew angle. To illustrated this behavior, example plot of the measured and predicted mid-span deflections for the Wilmington Street Bridge is presented in Figure 3-3. The difference between the prediction and the measured is apparent.

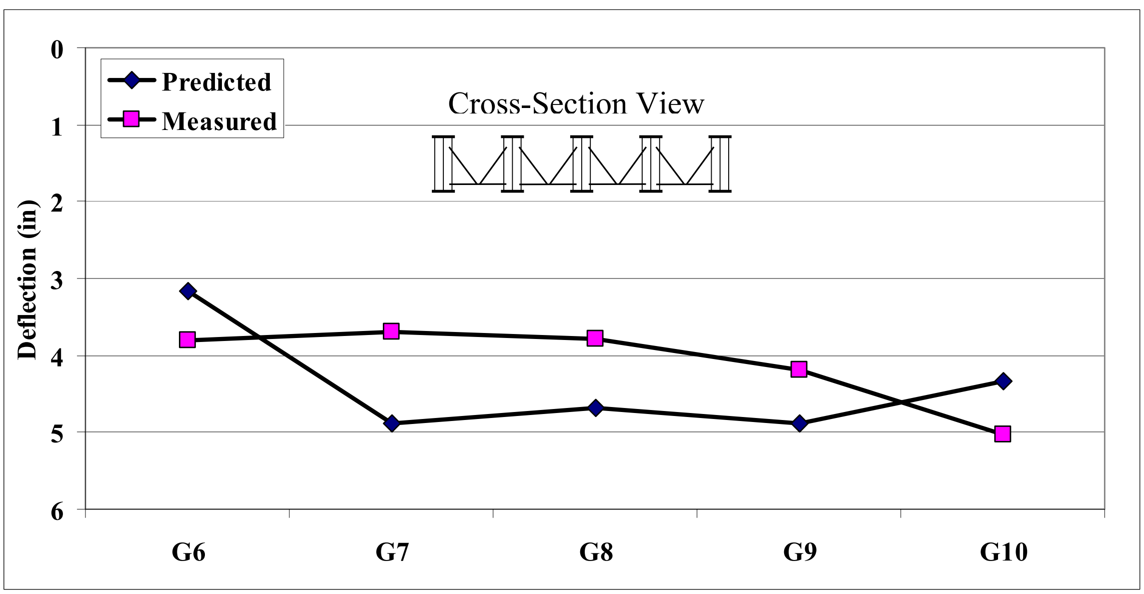


Figure 3-3 Comparison of Measured vs. Predicted Mid-span Deflections for Wilmington Street Bridge

Using the single line model, the predicted deflection was calculated for each girder loaded by proportional tributary width of the concrete deck. As shown in the Figure 3-3, the Wilmington Street Bridge, which has a skew of sixty-two degree, deflected different from the single line model prediction. A comparison between the measured results and predicted results from single line model of each single span bridge is presented in chapter 5. In an effort to accurately verify the bridge modeling techniques, four of the single span bridges with a range of skew angles had been included in this thesis.

3.2.3 Stay-in-Place Metal Deck Form

Stay-in-Place Metal Deck Forms (SIP) are used to be support the wet concrete during construction and may be designed to transmit in-plane forces by acting as a diaphragm system spanning between girders. The forms are modeled as shear diaphragms that provide warping resistant to the top flange of the girder. However, in the bridge industry, SIP forms are not typically included in the calculation for the buckling resistance. The design of the SIP forms and their connections to the girders is usually the responsibility of the contractor. The connections between SIP form and angle allow the contractors to adjust the elevation of the SIP to account for the change in the thickness of the top flange of the girder or the differential camber elevation between adjacent girders. Figure 3-4 shows a typical connection between the SIP and the top flange of the girder.

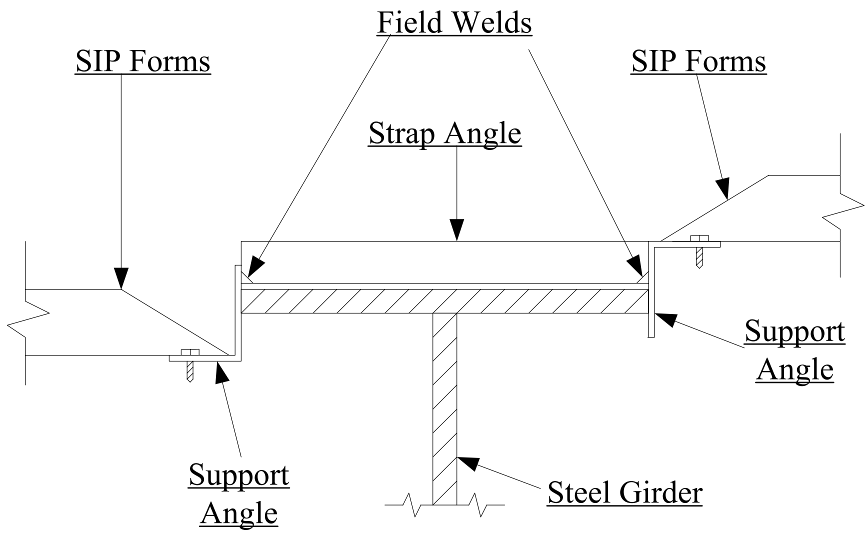


Figure 3-4 SIP Eccentric Connection

However, previous research reported by Eglimez (2003) and Whisenhunt (2004) reported that the diaphragm strength of the SIP forms affects the bridge behavior. Whisenhunt (2004) reported that shear stiffness of SIP affects the vertical deflection behavior of the skewed bridge. It is also believed by the author that SIP forms help the bridge system resist the shear and moment due to the differential vertical deflections between the girders.

From the field observations, three different sections of the SIP steel deck forms were used in the measured bridges. The SIP forms had width of 24 inches and 32 inches with gage of 16, 20, and 24. Additional, the details and example figures of the SIP forms are presented in Appendix F.

3.3 Bridge Description

3.3.1 General

In this thesis, monitoring results from four bridge by Whisenhunt (2004) and Fisher (2005) are presented and used in the analytical comparison to verify the modeling techniques in chapter 5. These four bridges are named as followed; Eno River Bridge, US 29 Bridge, Bridge 8, and Bridge 10. One additional bridge, Wilmington Street Bridge, was monitored as a part of this research study and was included in the modeling and comparison. A summary of the bridge descriptions and characteristics are presented in Table 3-1.

Table 3-1 Summary of Bridge Measured

Bridge	Type	Span Length, ft	Number of Girders	Girder Spacing, ft	Nominal Skew Angle, deg	Equivalent Skew Angle, deg
Eno River	Single Span	236	5	9.6	90	0
US 29	Single Span	124	7	7.8	46	44
Bridge 8	Single Span	153	6	11.3	60	30
Bridge 10	Continuous Span	300	4	9.5	147	57
Wilmington St.	Single Span	149.5	5	8.3	152	62

3.3.2 Eno River Bridge (NC 157 over Eno River, Project # U-2102)

The Eno River Bridge is located in Durham, North Carolina. It is a five-girder, simple span, staged construction steel plate girder bridge. High Performance Steel HPS 70W as used to fabricate the girders in this project. X-type by L 5x5x5/16 steel angle and K-type by MC 12x31 and L 5x5x5/16 steel cross-frames were used for the intermediate and end bent bracing system. Respectively, the field measured concrete unit weight was 116.8 pound/ft^3

The reported results from the field measurement were the vertical deflection for each girder at the quarter points along the girder length. The duration of the concrete placement was also recorded approximately ten hours. Figure 3-5 shows the plan view of the Eno River Bridge.

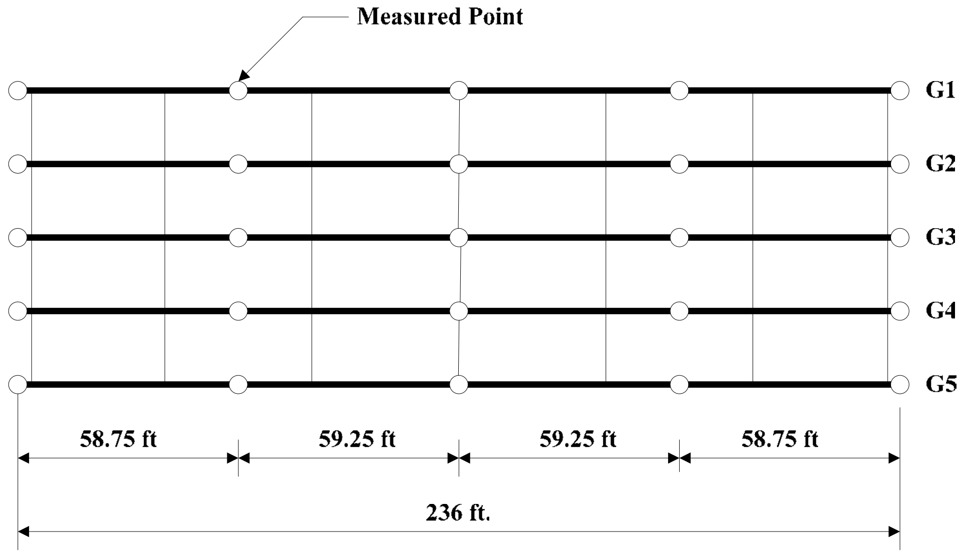


Figure 3-5 Plan View of Eno River Bridge

3.3.3 US 29 Bridge (US29 over NC 150, Project # R-0984B)

US 29 Bridge is a seven-girder, simple span, single staged construction, steel plate girder bridge located on the southbound lane of US 29 crossing over NC 150 near Reidsville, North Carolina. All the structure steel members were fabricated using AASHTO M270 Grade 50 steel. K-type by L 3x3x5/16 steel angle and K-type by C 15x33.9 and WT 4x9 were used for the intermediate and end bent bracing. Respectively, the field measures concrete unit weight was approximately 117 pound/ft$^{3.}$

Since the US 29 Bridge has seven girders, the field measurement reported the vertical deflections for two exterior, two adjacent exterior and the middle girders. The quarter and three quarter points were recorded along the longitudinal girders. However, the middle point of each girder could not be monitored because of the traffic under the bridge. The measured point was conducted at about five feet away from the actual middle point of each girder instead. In additional, because of the concerns about the partial composite action between the girders and concrete which could possibly change the expected bridge deflection; a concrete retarder was used to control the setting time of the concrete in this project. The construction lasted approximately six hours. Figure 3-6 shows the plan view of the US 29 bridge.

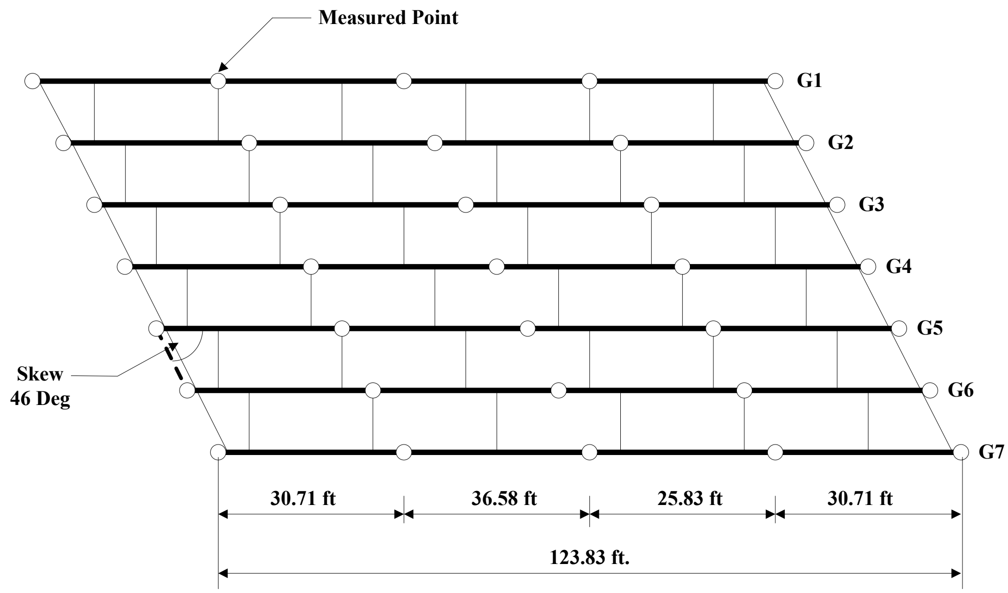


Figure 3-6 Plan View of US 29 Bridge

3.3.4 Bridge 8 (East Bound Bridge on US 64 Bypass over Smithfield Road # R-2547)

Bridge 8 is a six-girder, simple span, single staged construction steel plate girder bridge located on the eastbound lane of US 64 Bypass over Smithfield road, North Carolina. AASHTO M270 Grade 50 was used to fabricate the steel members. For the lateral bracing system, X-type by L3x3x3/8 and K-type by C 15x33.9 and WT 4x14 were used for the intermediate and end bent cross-frames. The nominal concrete unit weight provided by the bridge plan is 150 pound/ft^3.

Vertical deflections were measured for all six girders at the quarter and three quarter points along the longitudinal girders. Again, because of the roadway under the bridge, the middle measured points were recorded approximately sixteen feet away from the actual mid points. Concrete deck placement lasted approximately five hours. Figure 3-7 shows the plan view of the Bridge 8.

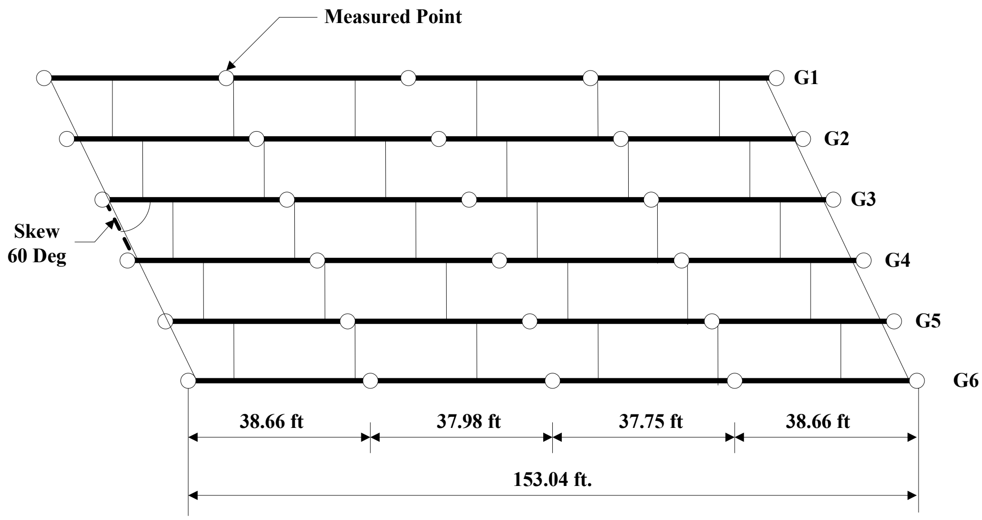


Figure 3-7 Plan View of Bridge 8

3.3.5 Bridge 10 (Knightdale-Eagle Rock Road over US 64 Bypass # R-2547)

Bridge 10 is a four-girder, continuous span, single staged construction steel plate girder bridge located on the Knightdale-Eagle Rock Road over US 64 Bypass, North Carolina. The first span is one hundred and fifty five feet long measured from the abutment to the intermediate support and one hundred and forty five feet for the second span from the intermediate support to another abutment. The steel structures were fabricated with AASHTO M270 Grade 50 steel. The lateral bracing system as constructed of X type by WT 4x12 and K type by MC 18x42.7 and WT 4x12 for the intermediate and end diaphragms. Respectively, the nominal concrete unit weight noted in the bridge plan is 150 pound/ft^3. A plan view of Bridge 10 is shown in the Figure 3-8.

Vertical deflections were measured at four points along each girder; four-tenth and seven-tenth of the first span, and two-tenth and six-tenth of the second span. Since Bridge 8 is a long bridge, pour sequence was divided in two. First, the concrete deck placement was started in the second span at eighty feet away from the abutment and was continued to the end of the second span. Second, the concrete placement started from the abutment in the first span and continued to the beginning point of the first pour. The first pour lasted approximately two hours and the second pour lasted approximately seven hours.

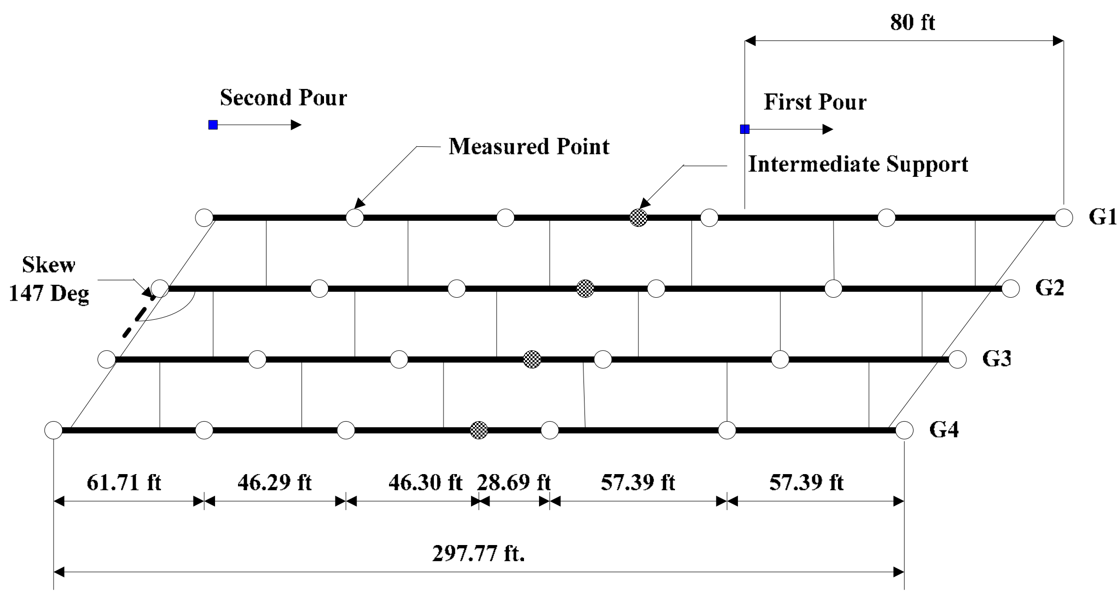


Figure 3-8 Plan View of Bridge 10

3.3.6 Wilmington Street Bridge (South Wilmington Street Bridge # B-3257)

Wilmington Street Bridge is a five-girder, simple span, two-stage construction steel plate girder bridge located on the Wilmington Street near downtown Raleigh, North Carolina. The girders were fabricated using AASHTO M270 grade 50. K-type by 3x3x5/16 and K-type by WT 4x12 and C 15x 50 were used for the intermediate and end bent cross frames. Respectively, the measured concrete unit weight measured was 118 pound/ft^3.

Vertical deflections of the girders were measure at the quarter and three quarter points along the span of each girder. For the deflection in the mid point of span, the points at six tenth of the each girder was measured instead because of a conflict with the railroad tracks. The concrete placement lasted approximately five hours. A plan view of the Wilmington Street Bridge is shown in the Figure 3-9.

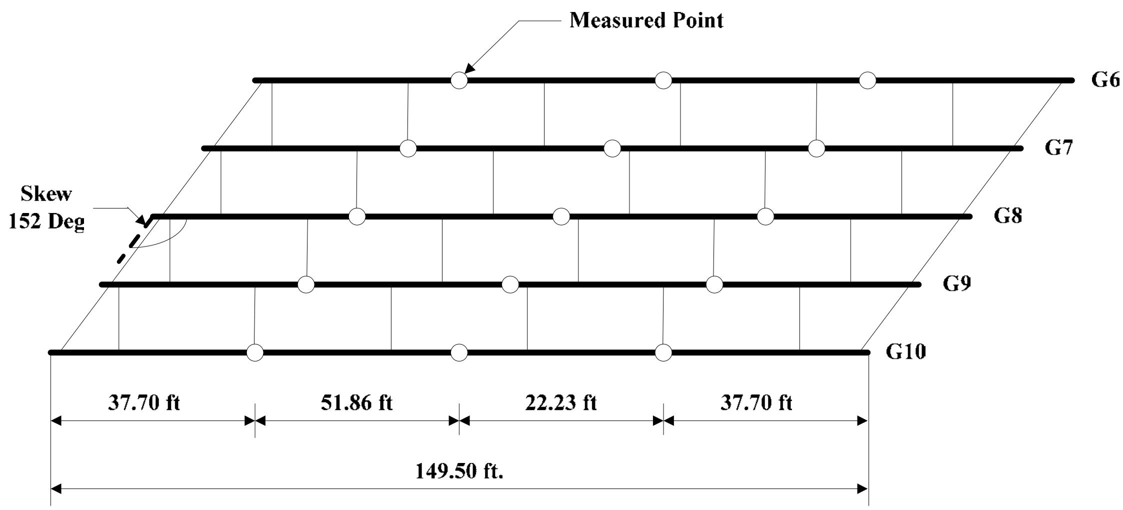


Figure 3-9 Plan View of Wilmington Street Bridge

3.4 Measurement Methods

To obtain the vertical girder deflections, two methods of the field measurement were utilized: one method uses string potentiometer connected to a power source and a multimeter and the other uses tell-tail. Whisenhunt (2004) and Fisher (2005) used string potentiometers to record the vertical non-composite deflections in their studies, whereas the second method was used to monitor the vertical deflections in Wilmington Street Bridge. Details of both methods are presented in the following sections.

3.4.1 Potentiometer Method

Whisenhunt (2004) and Fisher (2005) used the string potentiometer to measure the vertical deflection in their studies. To measure the vertical movement of each girder, thin steel wires were attached to the bottom flanges of the girders in the desired locations. Two C-clamps were used to attach the steel angles with the bottom flanges of the girders in order to hang the thin tension wires as shown in the Figure 3-10.

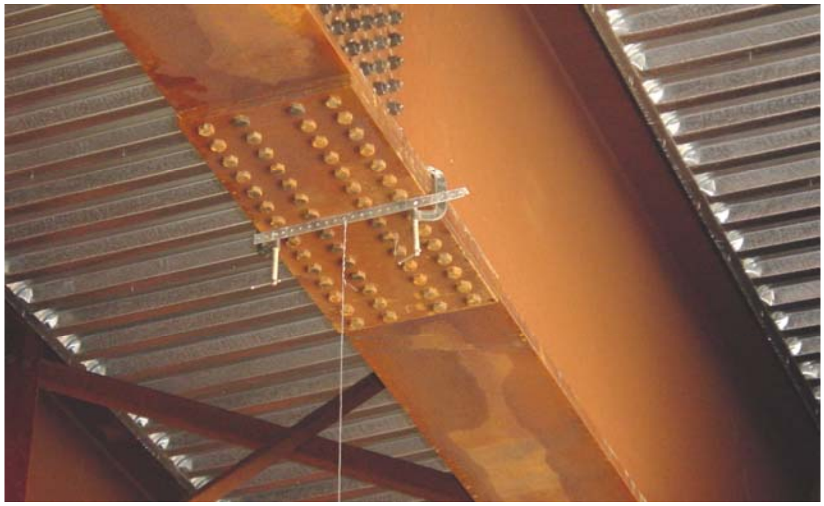


Figure 3-10 C-Clamps and Steel Angle Connected to the Bottom Flange of the Girder

Potentiometer was put on the ground and attached to the steel wire to capture the vertical movement. Small Weight was also attached to the wire to keep the string stretched and constant tension in the wires as shown in the Figure 3-11.

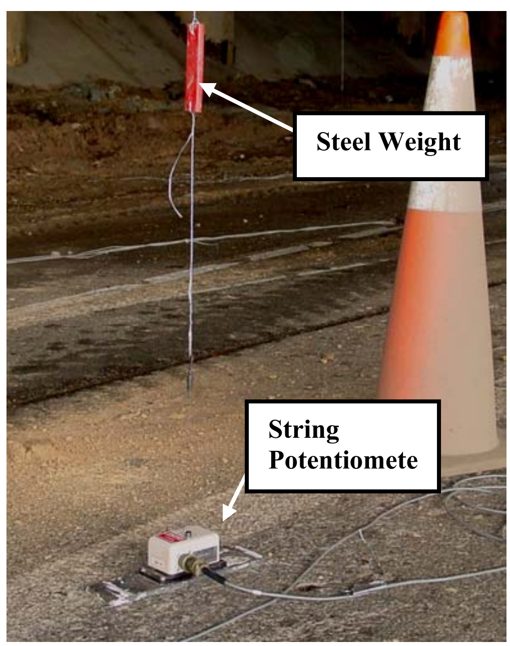


Figure 3-11 String Potentiometer

The potentiometers were wired to a switch and balance unit and connected to the power supply. The calibrated multimeter was connected to the system to capture the differential voltage. The actual deflection was calculated using calibration factors from the differential voltage that was recorded during the bridge deck placement.

Dial gages were used to measure the vertical movements of the girder end support bearings at each end-bent (see Figure 3-12). The net vertical deflection of the girder was calculated by subtracting the deflection at the end support from the deflection value at the measured point.

Figure 3-12 Dial Gage Placed at End Support of Girder

Whisenhunt also discussed the possible source of the error in his measurement procedure. Because of the sensitivity of the micrometer and potentiometer, the reading can be affected by the vibration of wind due to gusts. However, he concluded that these errors were small and not significant with respect to the measured values.

3.4.2 *Tell-Tail Method*

Wilmington Street Bridge deflections were measured using the Tell-Tail method. Two C-clamps and one angle were used to attach the wires from the bottom flanges of the girders at the desired location as shown in Figure 3-13.

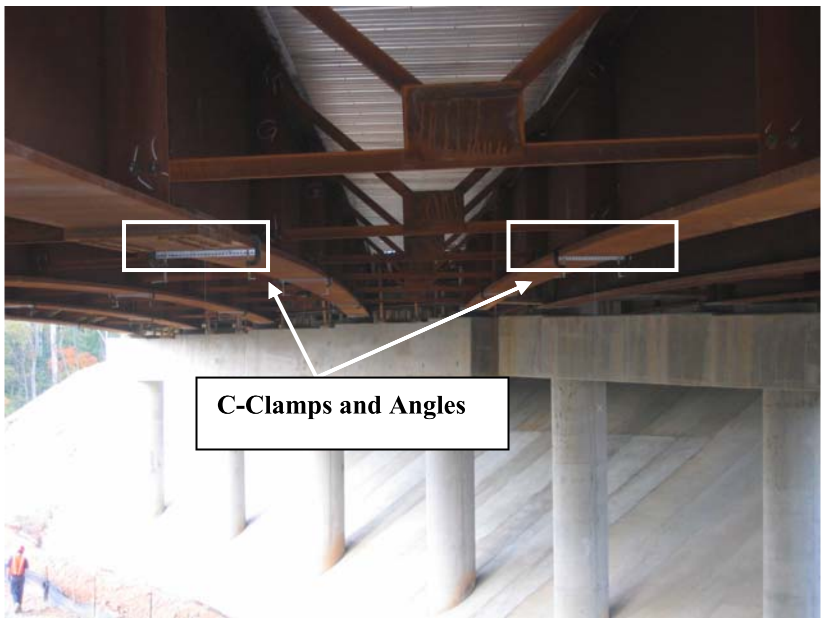
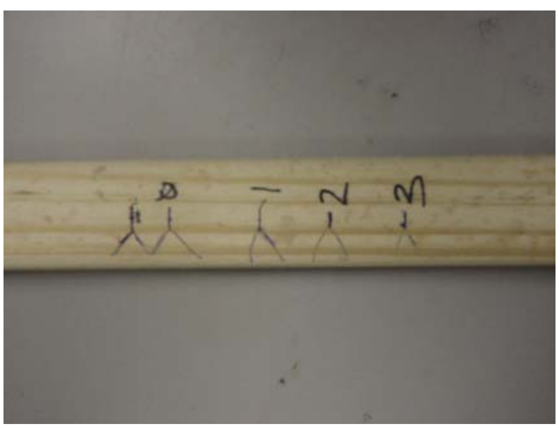


Figure 3-13 C-Clamps and Angles attached to Bottom Flanges of Wilmington Street Bridge

Small steel weights were used to keep constant tension in the wires and to produce an elevation marker. Wooden stakes were driven into the ground adjacent to each of the hanging steel weights. The inertial elevation of the hanging weights before the start of the concrete placement was marked on the wood stake. This mark was utilized as the zero deflection reference line for the other measurements. Additional marks were recorded each the wet concrete reached to the quarter point, mid point, three quarter point of the longitudinal span.

At last, the wood stakes were marked when the concrete reached to the end of the span. A tape measure was used to measure the differential elevation marked on the wooden stakes. Figure 3-14 shows the picture of a wooden stake during the installation process and the wood stakes after the bridge construction was completed.

a) Wooden Stake Installation



b) Scale marked during the Construction

c) Front View of the Wooden Stake

Figure 3-14 Wooden Stake Used in Tell-Tail Method

Wilmington Street Bridge has steel pot bearing at the end supports, which do not allows the vertical movement (See Figure 3-15). Therefore, the vertical deflection at the support was neglected.

Figure 3-15 Pot Bearing of Wilmington Street Bridge

3.5 Measured Results

Table 3-2 contains the vertical deflection at the mid-point of each girder from each bridge. To compare the results, Labels A-F was assigned to the girders. Label A was assigned to the girder in the very left side related to the direction of the survey line, and next label for the adjacent girder and so on. Since Bridge 10 has two concrete pours, the results are presented the vertical deflection separately in term of deflections from first concrete pour (Bridge 10 A) and second concrete pour (Bridge 10B).

Table 3-2 Summary of Mid-Span Non-Composite Measured Deflections (inches)

Bridge	Eno River	US 29	Bridge 8	Bridge 10 A	Bridge 10 B	Wilmington St.
Girder A	6.94	4.33	2.89	1.29	2.07	3.80
Girder B	6.41	4.05	3.14	1.10	1.64	3.70
Girder C	5.88	3.84	3.17	1.21	1.66	3.78
Girder D	5.14	4.18	3.26	1.36	1.64	4.19
Girder E	4.89	4.55	3.30	NA	NA	5.04
Girder F	NA	NA	3.24	NA	NA	NA

The summary results from Table 3-2 were plotted and presented to show the vertical deflection shape of the cross section in the Figure 3-16.

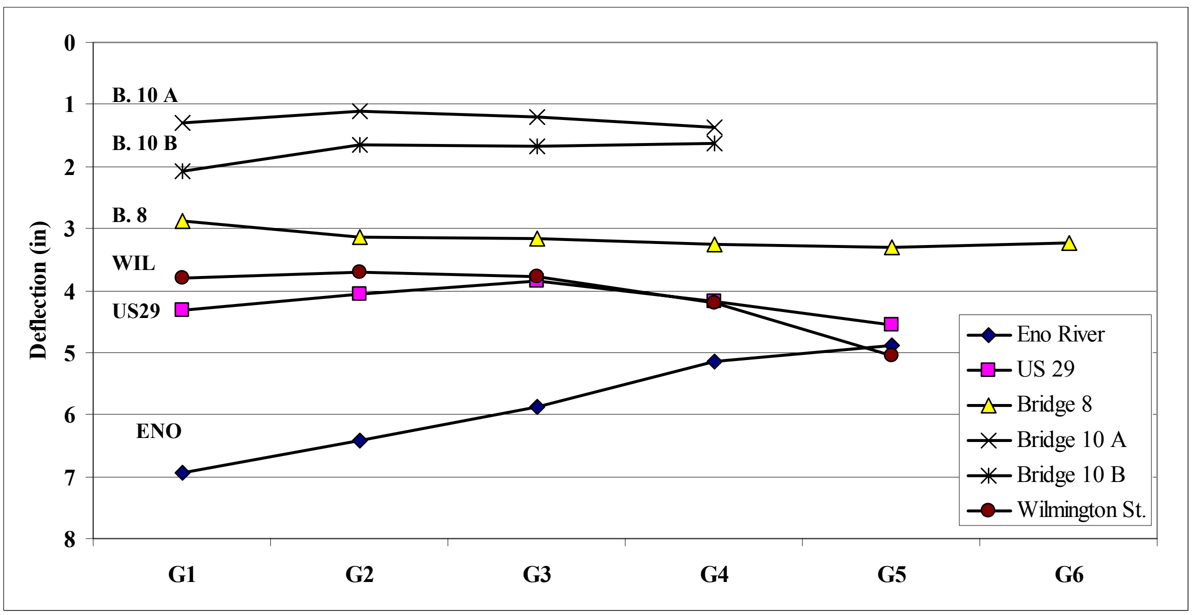


Figure 3-16 Plot of Mid-Span Non-Composite Measured Deflections

The Eno River Bridge and the Wilmington Steel Bridge have an unequal over hang, which obviously affect the vertical deflection behavior. The exterior girder on the side that has a greater overhang has a greater vertical load on the girder. Therefore, the deflection shape varies approximately linearly across the entire width of the bridge from one side to the other side. In addition, the Wilmington Street Bridge is a very high skew bridge (Skew = 62 degree). The vertical deflection shape is similar to an “inverted bowl shape” instead of linear.

Bridge 8, Bridge 10 and US 29 Bridge, have symmetric overhangs thus was also subjected to the balance load during the construction. However, the deflection shapes of Bridge 10 and US 29 are an “Inverted V-shape”, whereas the deflection shape of Bridge 8 is “V-Shape”. The reason for the different of the deflection shapes will be discussed later. Tables containing all of the measured field deflection data and all the deflection plots for all bridges measured are located in Appendix A-E.

3.6 Summary

There are several parameters affecting the bridge’s vertical deflection behavior, such as girder length, girder spacing, and size of the girder and the concrete deck properties. In addition, it was revealed that cross frame and diaphragms configurations, skew angle and stay-in-place metal deck forms have a significant effect to the bridge behavior.

As shown in the results, the vertical deflection behavior of different bridges with different characteristics causes varying deflected shapes. Details of the common vertical deflection prediction methods and analytical modeling techniques are presented in chapter 4. In addition, comparisons of the analytical and field measured values are presented in chapter 5.

Chapter 4 –Modeling Techniques and Results

4.1 Introduction

As part of this research, SAP 2000 was used to create analytical models of five steel plate girder bridges. The primary propose of the modeling is to create a new simplified modeling method which is easier and faster than finite element modeling methods to predict the non composite dead load deflections. The developed modeling methods were verified by comparing the results with the measured results as presented in chapter 5.

Four simplified modeling methods created by SAP2000 version 9 were developed in this study. In the effort to overview the model development and explain each component of the model, the contents in this chapter is separated in seven main sections; types of modeling method, steel plate girders, cross frames and diaphragms, stay-in-place forms, composite action, load calculation and result.

4.2 General

Structural Analysis Program (SAP) is a finite element analysis program developed by the Computer and Structure Inc. (CSI). CSI has been developed several version of SAP and it has become one of the oldest finite element programs currently on the market. By using the interface command, SAP is also one of the simplest structure analytical programs that have been used from the engineer all over the world. SAP 2000 is the latest release of the SAP series of computer program. In this thesis, SAP2000 version 9 was used to create all bridge models.

SAP2000 version 9 accommodate user to work easier and faster with new enhanced graphical user interface, such as object creation by point, line, area extrusions into line, area and solid, convenient unit conversion, allowing properties assignments during on screen object creation , and etc. In addition, SAP 2000 version 9 was chosen to be used in this study because of the new function of the shell element allowed the user create orthotropic shell element. By using different modification factors multiply to the stiffness of the shell element, the user can create the shell element that has different stiffness in different direction. This function was used to model the SIP forms that will be discuss later.

To create bridge models, all material properties used to create models in this thesis were linear elastic. All modeled steel members, the elastic modulus was 29000 ksi and the passion ratio was equal to 0.3. The yield strength of the steel member was assigned according to the real yield strength of the member in the bridge plan. Since the vertical deflections of the bridge structure were recorded due to the vertical dead load of the concrete deck only, the modification factor for the dead load in SAP 2000 was set equal to zero.

The remaining sections in this chapter will outline the modeling techniques developed and each component of the model. Four techniques, two-dimensional grillage model, three dimensional model, three-dimensional with frame SIP, three-dimensional with shell SIP, will be discussed first, then the components of models will be in the following sections. These components included steel plate girder, cross frame, stay-in-place form, load calculation, and the composite action between concrete deck and steel plate girders. The results from the models will also be presented at the end of this chapter.

4.3 Types of Model

Four simplified modeling methods were created in this study. The development of the modeling methods started from the basic two-dimensional grillage model. Each steel plate girder was modeled as single girder frame element. The cross-frame members were transferred to the simulated beam modeled as a single frame element. Figure 4-1 shows the sample of the two-dimensional grillage model of Eno River Bridge. The method used to transfer the cross frame to the frame element will be discussed later.

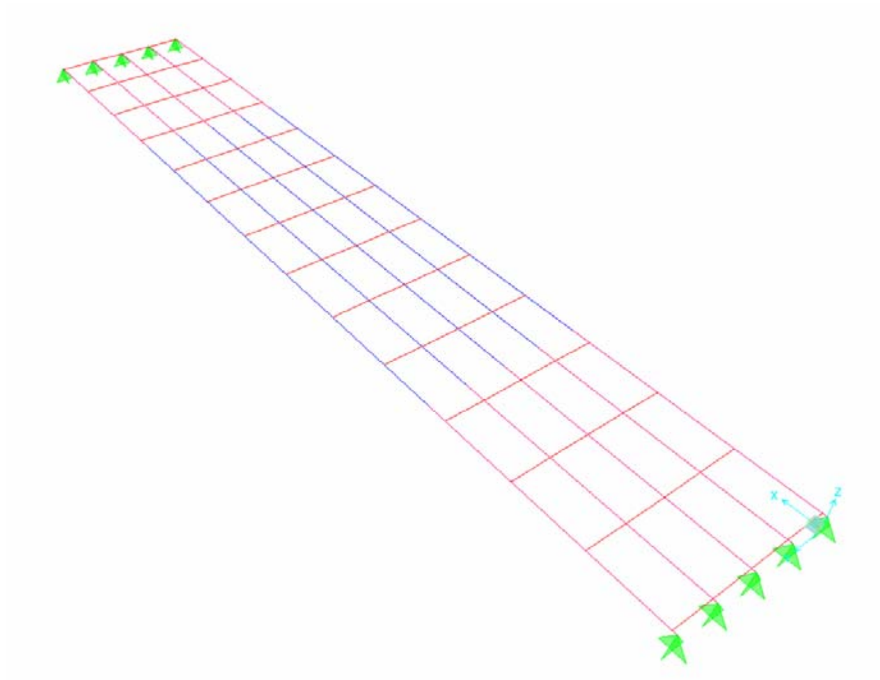


Figure 4-1 Two-Dimensional Grillage Model of Eno River Bridge

The three-dimensional analytical model was developed in the next step. Instead of using a single frame element simulated as the entire cross frame, the three-dimensional cross frame was created. The rigid links were connected to the frame elements to simulate the three-dimensional cross frame connected to the girders as shown in Figure 4-2.

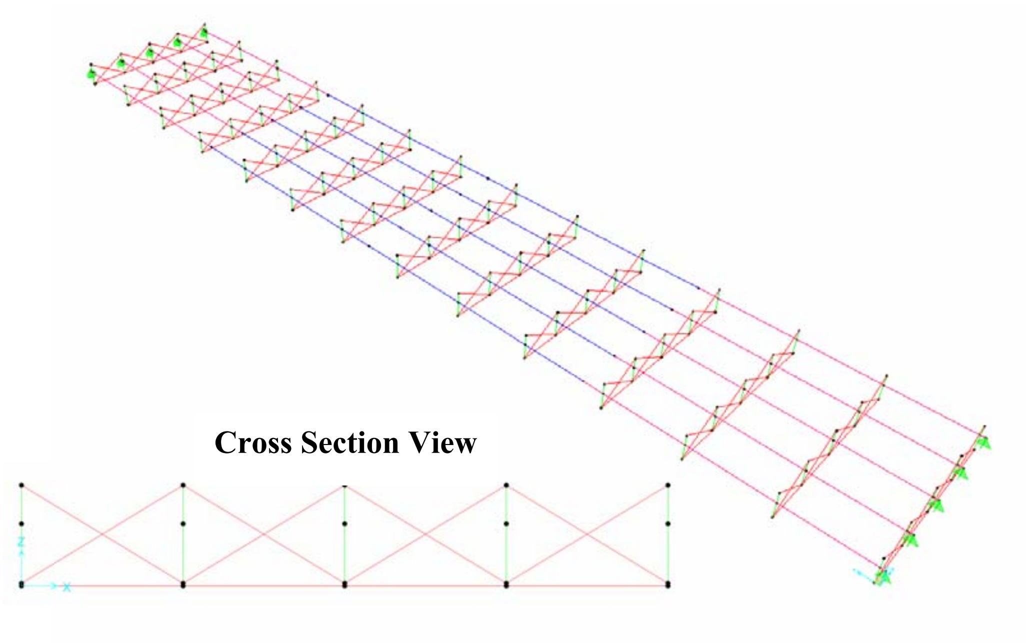


Figure 4-2 Three-Dimensional Model of Eno River Bridge

Since it was revealed that the stay-in-place form has a significant effect to the vertical deflection bridge behavior (Whisenhunt 2004), SIP forms were included in the third step of the modeling method. To represent the transferring load ability of the SIP form, tension-compression frame elements were used to create the simulated SIP forms. To represent the load transferring between girders and SIP forms, the frame elements as SIP form were connected to the girders by using rigid link elements as shown in Figure 4-3.

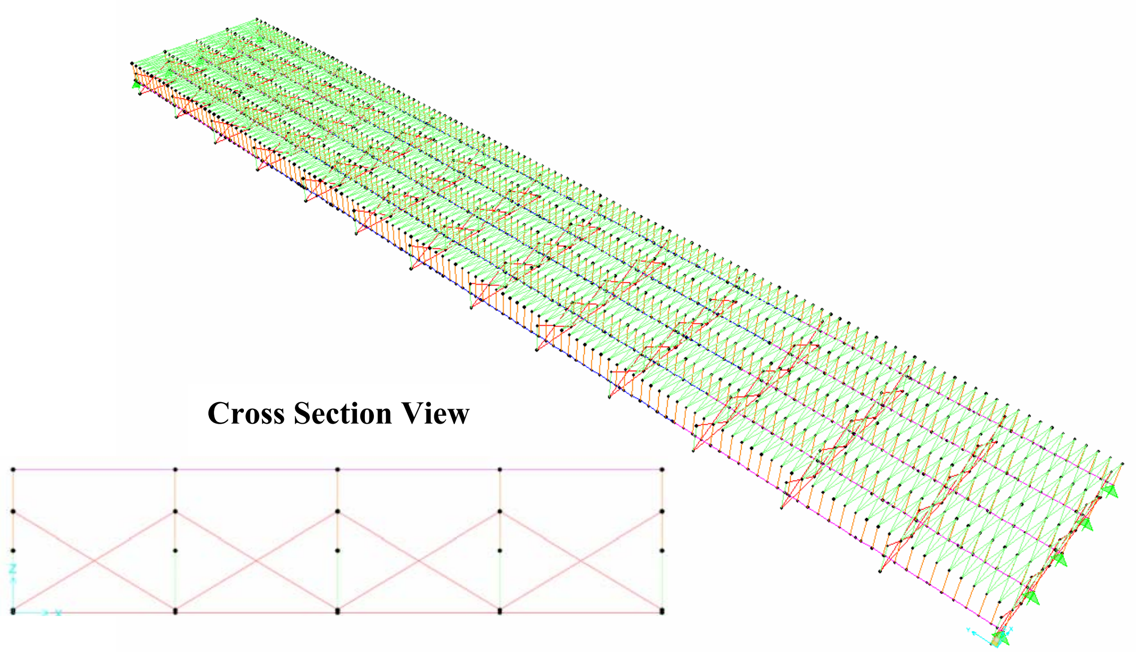


Figure 4-3 Three-Dimensional with SIP Frame Element Model of Eno River Bridge

The last step of the modeling method was developed by creating shell elements as the SIP forms. The compression and tension-transferring load ability composite with flexural behavior of the SIP forms were calculated and transferred to the shell element properties. Again, the shell element was connected to the rigid link element to represent the composite action between SIP forms and girders. Figure 4-4 shows the picture of the three-dimensional model included shell elements as SIP forms.

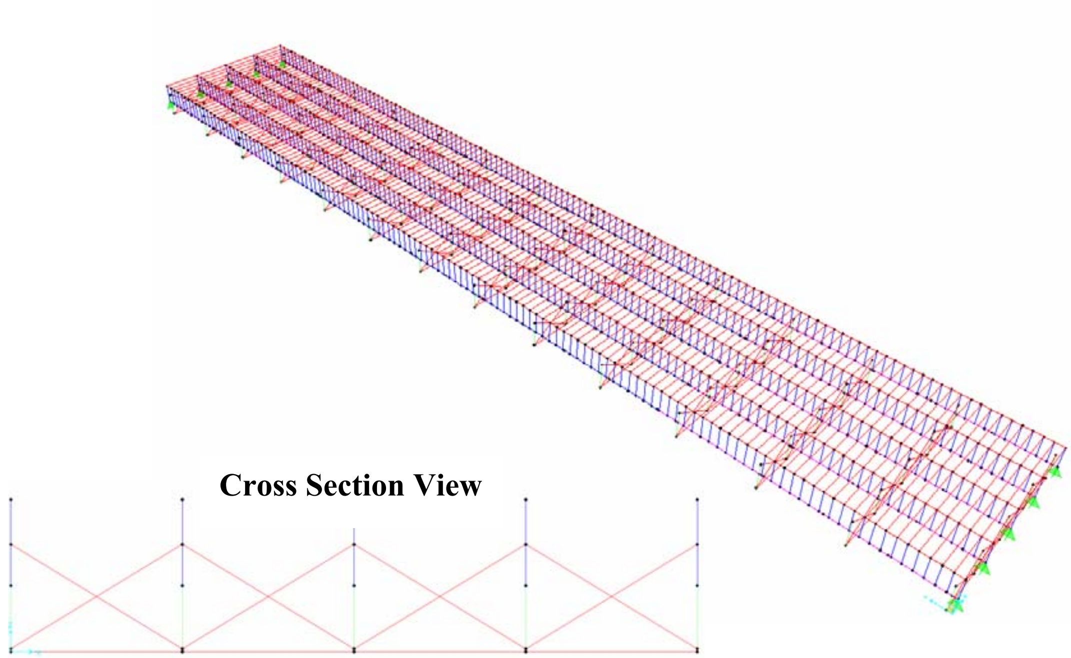


Figure 4-4 Three-Dimensional with SIP Shell Element Model of Eno River Bridge

4.4 Model's Component

4.4.1 Steel Plate Girders

The steel plate girder was created by using nodes to outline the length of the girder. By assigning nodes at both end-support points, the reference points of the edge of the girder were made. Since most of the steel plate girders were designed to increase the cross section at the mid location along span in order to increase the moment capacity; nodes need to be assigned at the change-section point. In addition, SAP 2000 Version 9 is able to report the deflections only at nodes; nodes also need to be assigned at the same location as measured points. Frame elements were used to connect each node together as the girder. (see Figure 4-5) Girder sections obtained from the bridge plan were assigned to the frame elements. By assuming that the frame element was laid out at the location of the centroid of the real steel plate girder sections, the location of the frame element will be used to refer to the locations of the cross frame and length of the rigid link elements in the following section.

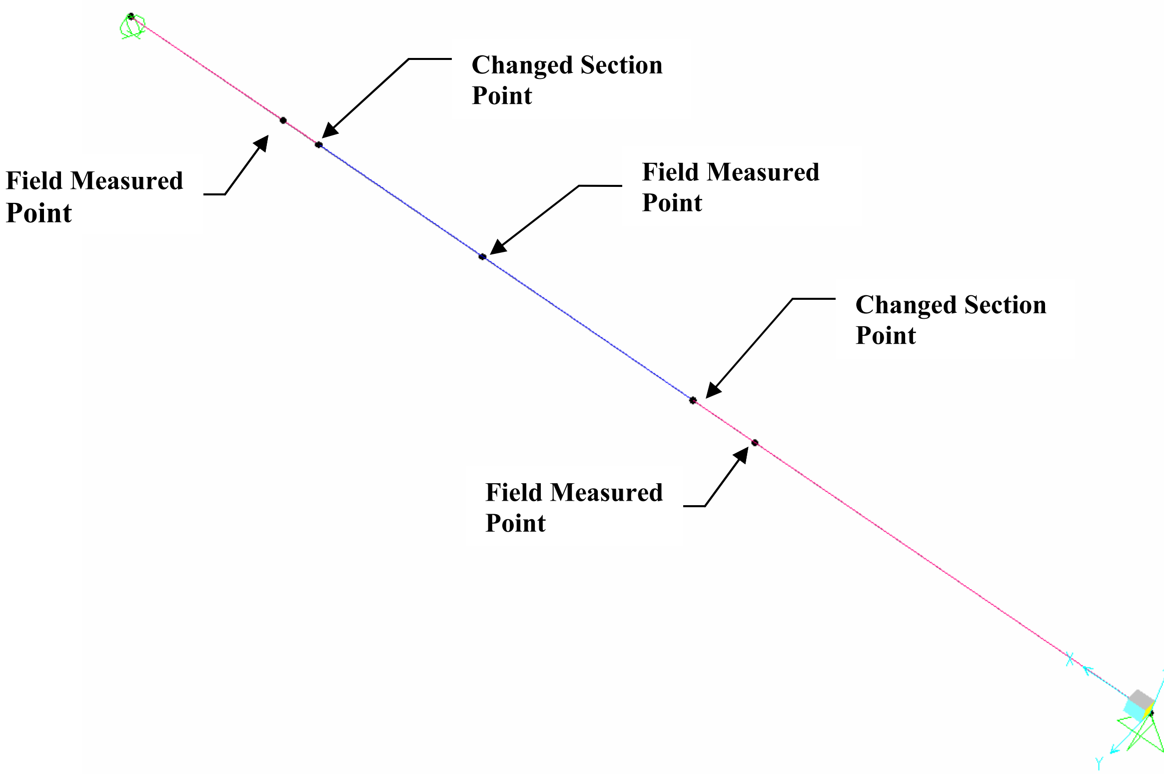


Figure 4-5 Single Girder Model

This modeling method was developed to be used for any girder configuration. To verify this modeling technique, the simple span girder was given the constant section for entire the model and subjected to uniformly distribution loads in order to compare the vertical deflection with the results from the hand analysis based on beam theory. In addition, steel plate girder SAP model that has an unequal cross section along the span was also verified by comparing the results with the results from the ANSYS finite element model. By following the modeling approach presented by Whisenhunt (2004), ANSYS finite element single girder was created to verify the results from SAP models.

Ratios were used to compare the results from the analytical models to the results from the SAP models. As a result, the ratio between SAP2000 without shear deformation and the hand analysis based on beam theory was approximately one. (approximately zero percent different). For the cross section changed along span of the girder, the ratio between result from SAP 2000 model and ANSYS finite element model was 1.012 (approximately 2% different). Therefore, it was concluded that the accuracy of the girder modeling technique used in SAP 2000 was adequate to use for the bridge models.

For the boundary condition at end-support nodes, all displacements (Direction X, Y, and Z) were restrained at one end as a pinned support. Only vertical and lateral displacements (Direction X and Z) were restrained at the other end as a roller support.

Since the displacement in the Y direction at the end-support of bridges is not ideally released as a roller support. The sensitivity study between pinned and roller support was created to ensure the reasonable of the modeling approach. Non-skewed and skewed bridge models were created with pinned-pinned support and pinned-roller support. As a result, there is no difference in the vertical deflections between two of the study cases in both bridge models.

4.4.2 Cross Frames & Diaphragms

Intermediate cross frames and end bent diaphragms were modeled by using frame elements. The frame elements as cross frame were rigidly connected to the girder at the same location of the cross-frame laid out in the bridge plan. Two types of the modeling techniques were created to model the cross frame, two-dimensional simulated beam and three-dimensional cross frame, as explained in the following sections.

4.4.2.1 Two-Dimensional Simulated Beam

For the first developed modeling technique, the entire cross frame was transferred to a single beam element connected to bridge girders as shown in Figure 4-6. To calculate the cross section of the simulated beam element, SAP analytical truss model was created to compare with the hand analysis. By using the same geometry of the cross frame, truss model was created by restrained both nodes on the left sides and symmetrically loaded on nodes on the right side. The vertical deflection of the node on the right side was recorded and used to be the reference deflection value for the hand analysis.

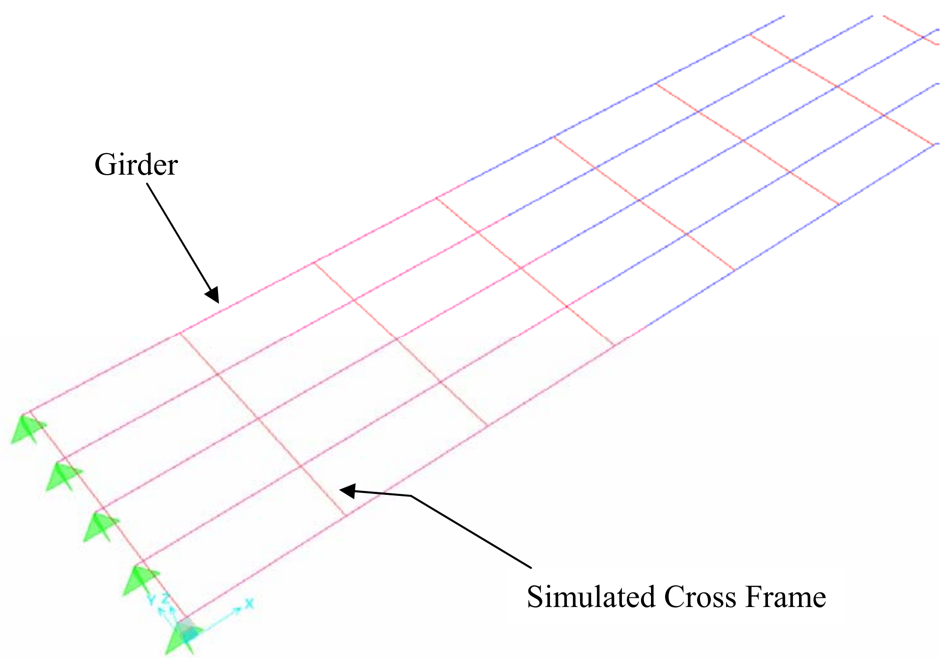
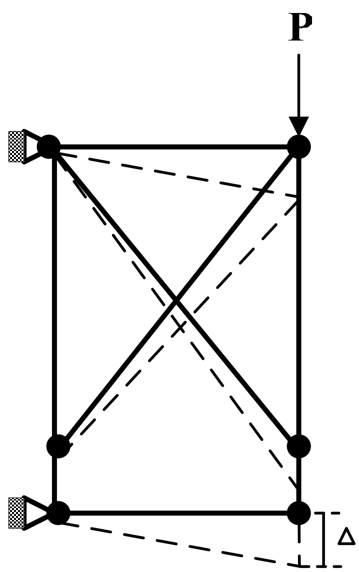
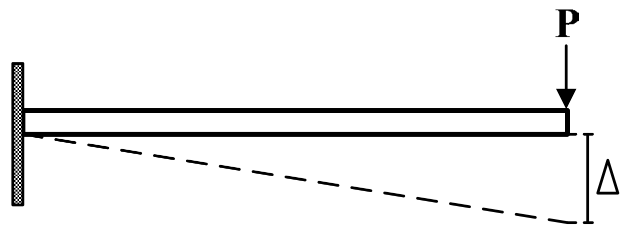


Figure 4-6 SAP, Simulated Beam as Cross Frames

For simulated beam hand analysis, a fixed-end simulated beam was subjected to the same load condition to compare the vertical deflection with the results from the truss models. The length of the simulated beam was set equal to the width of the real cross frame. (Figure 4-7) The analysis based on beam theory was made in order to get the cross section of the beam. Cross section of the simulated beam obtained by adjusting the cross section until the analysis gave the same deflection.



a) Cross-Frame Model, SAP



b.) Simulated Cross Beam, Beam Theory

Figure 4-7 Simulated Beam Element Compared with SAP Cross Frame Analysis

4.4.2.2 Three-Dimensional Simulated Cross Frame

Frame elements were used to create the three-dimensional simulated cross-frame. The cross frames were typically consisted of L-shape steel angles, whereas the end bent diaphragms were consisted of C or MC shape and WT shape steel members. In order to represent the model behavior similar to the real bridge structure, the same cross sections of the cross-frame members were used to create the model. To represent the force and moment transferring behavior of the cross-frame, rigid link elements were used to connect the frame element to the girder (see Figure 4-8). The detail of the rigid link was explained as followed.

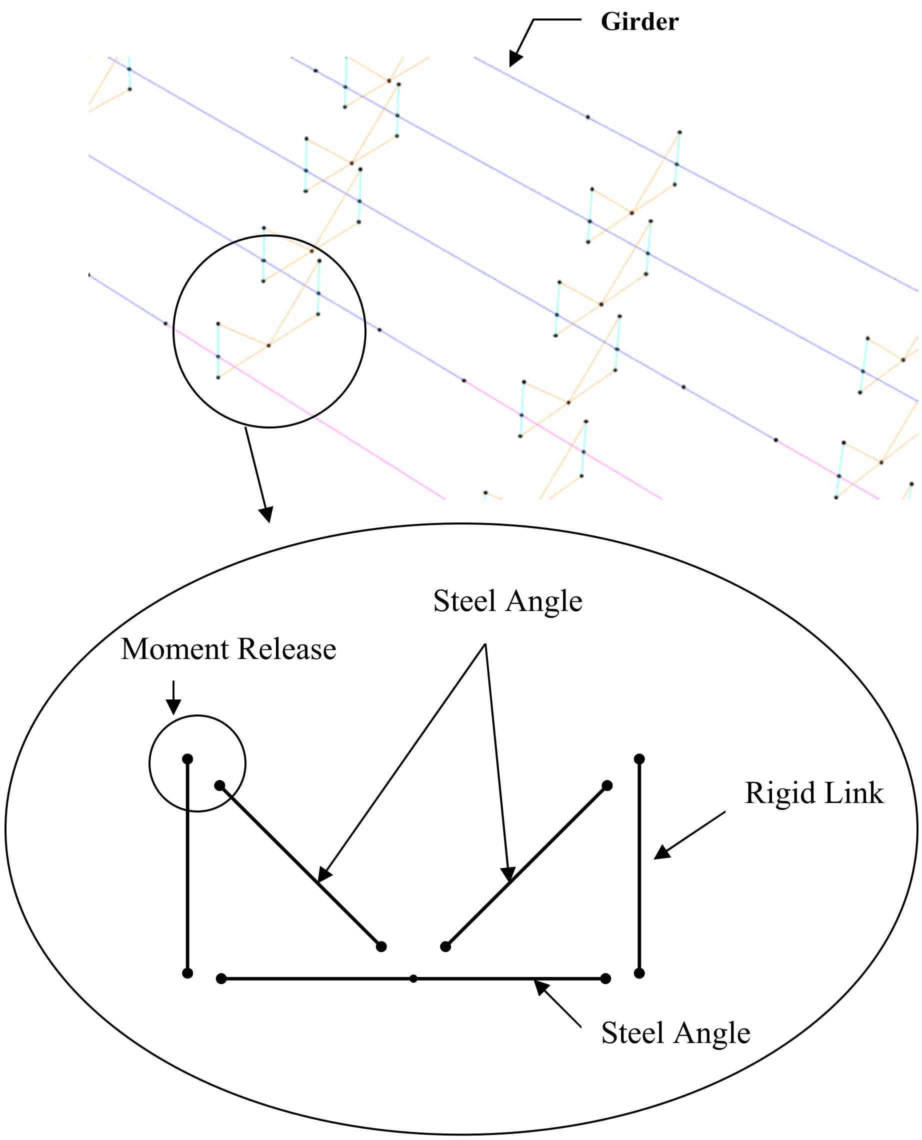


Figure 4-8 SAP, Simulated Cross-Frame

4.4.2.3 Rigid Link Element 1 (RL1)

Frame element was used to create the rigid link element. By assigning the large flexure rigidity to the element, fame element was assumed to be ideally rigid and transfer forces from element to adjacent element perfectly. Flexure rigidity or the product between elastic modulus and moment of inertia of the member is one of the most significant factors affecting the vertical deflection behavior of the bridge model. It was believed by the author that increasing the stiffness of the rigid link element improves the stiffness of the model.

Regarding the previous research recommendations (Jetann, 2002), large elastic modulus of the member was chosen to be the variable factor of creating the rigid link element instead of varying the cross-section of the member. Jetann explained that large section of the rigid link might over improved the lateral bracing length and increased stiffness of the structures. However, SAP 2000 program has a limitation for the difference of the bending stiffness of the adjacent elements. Too large of a different stiffness between the elements can cause an analytical error. In this study, the flexural rigidity of rigid link element was larger than the flexural rigidity of cross-frame member approximately fifty times. This approach was verified by a sensitivity study that will be discussed later in this chapter.

Length of the rigid link was creating regarding to the length from centroid of the girder to the center of the connection between girder and cross frame as shown in the Figure 4-9. Since there are two kinds of rigid link element in this study, the rigid link connected between cross frames and girders was named “Rigid Link 1” (RL1). “Rigid Link 2” (RL2) that used to connect between girders and SIP form will be discussed in the following section.

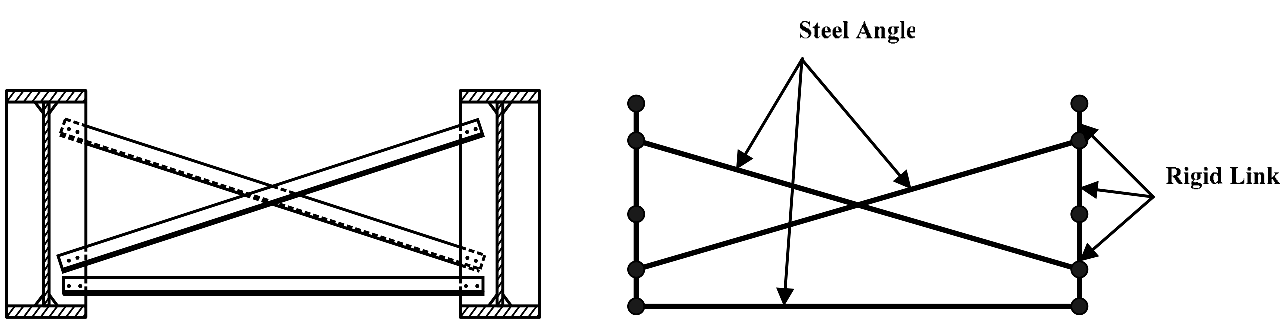
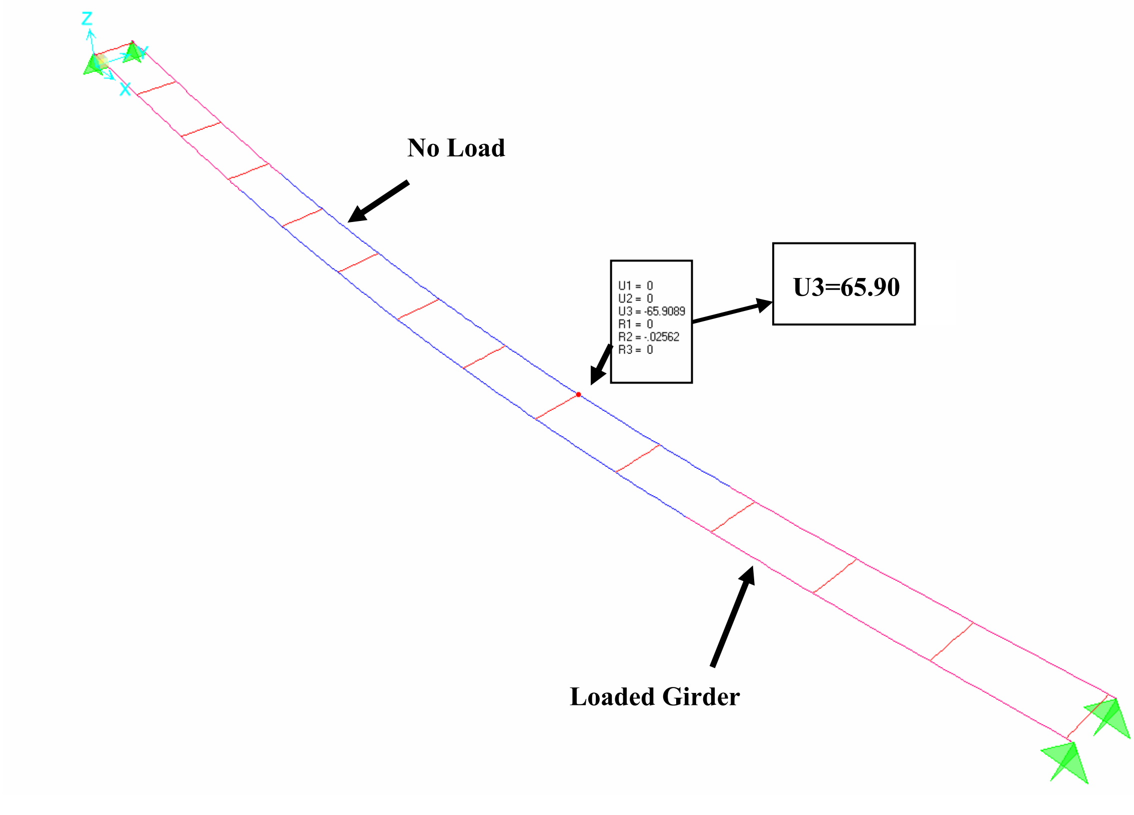


Figure 4-9 Simulated Cross Frame Compared with Actual Cross Frame

4.4.2.4 Load Sharing Ability

To verify cross-frame modeling method, twin girders were created to check the load sharing ability of the models. One of the girder was subjected to uniformly distribution loads, whereas no load on the other girder. This was done to observe the load distributing ability of the cross-frames and verify the deflected shape of the model. One kip uniformly distribution load was applied girder used in Eno River Bridge. Figure 4-10 shows the deflection shape of the twin girders connected by simulated beams.



*Unit is in inch

Figure 4-10 Load Sharing Ability of Simulated Beam as Cross Frame

Figure 4-11 shows the deflection shape of the twin girders connected by three-dimensional simulated cross frames.

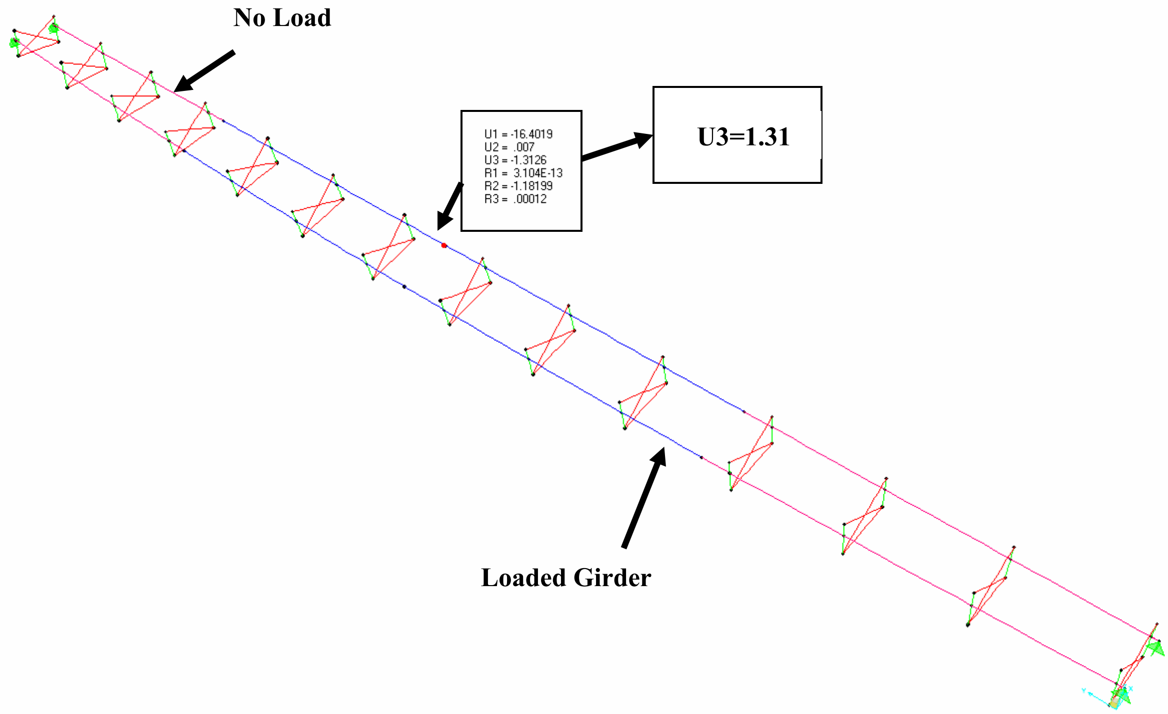


Figure 4-11 Load Sharing Ability of Three-Dimensional Simulated Cross Frame

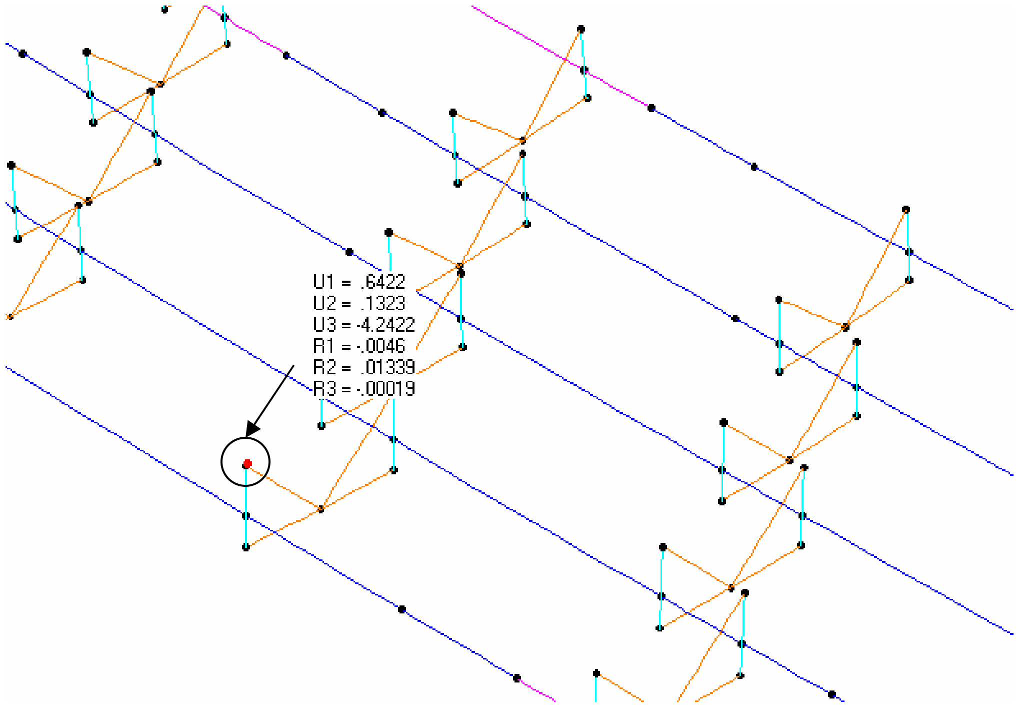
As a result, both of the cross-frame modeling methods were able to transfer vertical load to the adjacent girder. Once these modeling techniques were verified and deemed appropriate, the entire bridge systems were created.

4.4.3 Stay-in-Place Metal Deck Form

4.4.3.1 General

Whisenhunt (2004) revealed a significant effect of the stay-in-place form to the vertical deflection behavior of the bridge structure. As a result from his study, Whisenhunt concluded from his models that including the SIP form to the skewed bridge models improved the vertical deflection value became similar to the measured results. Based on the study by Helwig (2002), reasonable strength and stiffness of SIP form was selected without conducting additional laboratory. This study provided the basis that was used to develop the shear properties to be used in the SAP models. First, the models were developed by using tension-compression only members that connect the top flanges of the girders. It was later discovered that the SIP form systems largely behaves as tension-compression composite with flexure behavior member. Details of the development of the SIP properties used in this study and their effect on the behavior of bridge models will be later discussed.

The deflection behavior predicted by SAP models of skewed bridges was found to be markedly different from the measured. Figure 4-12 is a picture of skewed bridge model with deflections on a node on the top of one of the rigid link elements. According to the figure, the plate girders of skewed bridges tend to rotate in addition to the downward deflection. U1 is represented the lateral deflection, whereas U3 represented the vertical deflection at the node. The lateral deflection on the node showed that the girders of the skewed bridge were rotated. This differs from the behavior of the non-skewed bridge model that was found only deflect downward with small or no rotation. This rotation of the girder cross-sections would provide the mechanism necessary to activate the SIP forms as force distributing elements within the SAP models.



Note* unit is in inch

Figure 4-12 Displacement of Skewed Bridge Model

4.4.3.2 Importance of Stay-in-Place Metal Deck Form

Vertical load from weight of the SIP form was included to the design dead loads applied to the models in order to predict the vertical deflection. However, the force transferring behavior of the SIP was not included in the steel bridge design. In an effort to investigate the effect of SIP form to the vertical deflection behavior of the bridge models, the comparison of the non-skewed (Eno River Bridge) and skewed (Wilmington Street Bridge) bridge with and without SIP was conducted.

The vertical deflections from SAP models at the mid-span of non-skewed bridge were plotted in the Figure 4-13. It shows that the SIP form has no significant effect to the vertical deflection behavior of the non-skewed bridge. Since the girder of the non-skewed bridge has no rotation and deflects downward only, the force transferring behavior of the SIP form does not account to the model.

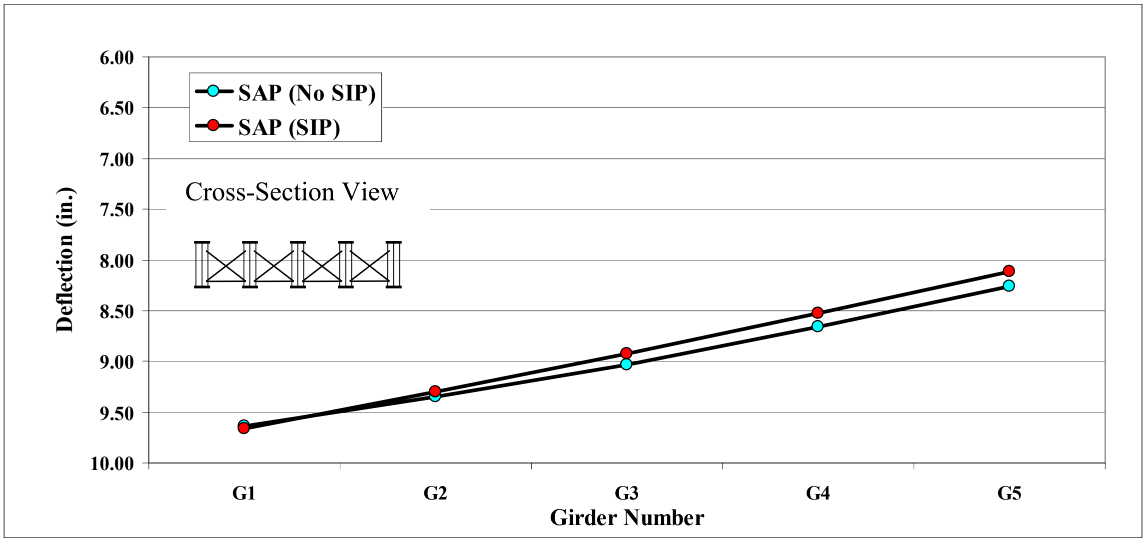


Figure 4-13 Non-skewed Bridge, Vertical Deflections from SAP Models with and without SIP Forms at Mid Span

The effect of including SIP to the bridge model is significant in skewed bridge. Figure 4-14 shows the mid-span deflections predicted by SAP of the Wilmington Street Bridge model with and without SIP form. The difference in deflection behavior of the two models is apparent. The opposite trend of the deflected shape is obviously seen. Without SIP, the girder of the bridge deflected followed the trend of the vertical load. Since Wilmington Street Bridge has an unequal over hang, the results from the model without SIP form showed that the exterior girders in one side deflect less than the other one. In addition, the mid girder deflected more due to more proportion tributary width of the concrete deck. When SIP forms were included in the model, the deflection shape of the model with SIP form is inverted compared with the results from the model without SIP.

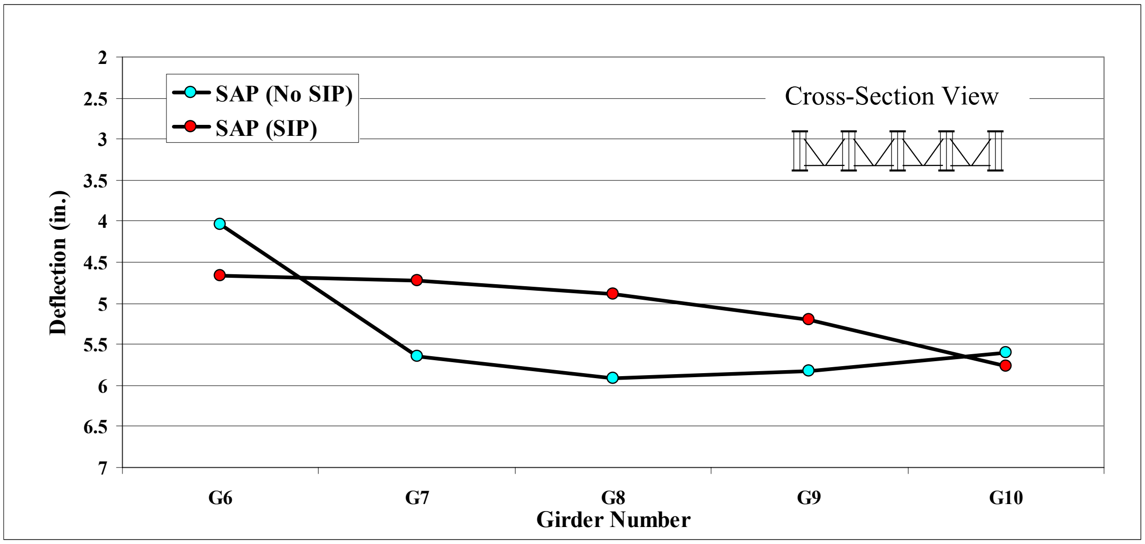


Figure 4-14 Skewed Bridge, Vertical Deflections from SAP Models with and without SIP Forms at Mid Span

4.4.3.3 Stay-in-Place Modeling Method

Whisenhunt (2005) concluded in his research that SIP form is a significant factor affected to the bridge behavior and need to be included in the bridge modeling. To include SIP forms in the bridge models, two modeling method were created in this study; Tension-Compression frame element and Tension-Compression and Bending Shell Element. Both methods will be explained in the following sections.

4.4.3.3.1 Tension-Compression Frame Element

By following the approach presented by Whisenhunt (2004), frame elements were used to represent the tension and compression behavior of the SIP form. In this study, the frame element was connected to the rigid link connected to the girder as shown in Figure 4-15.

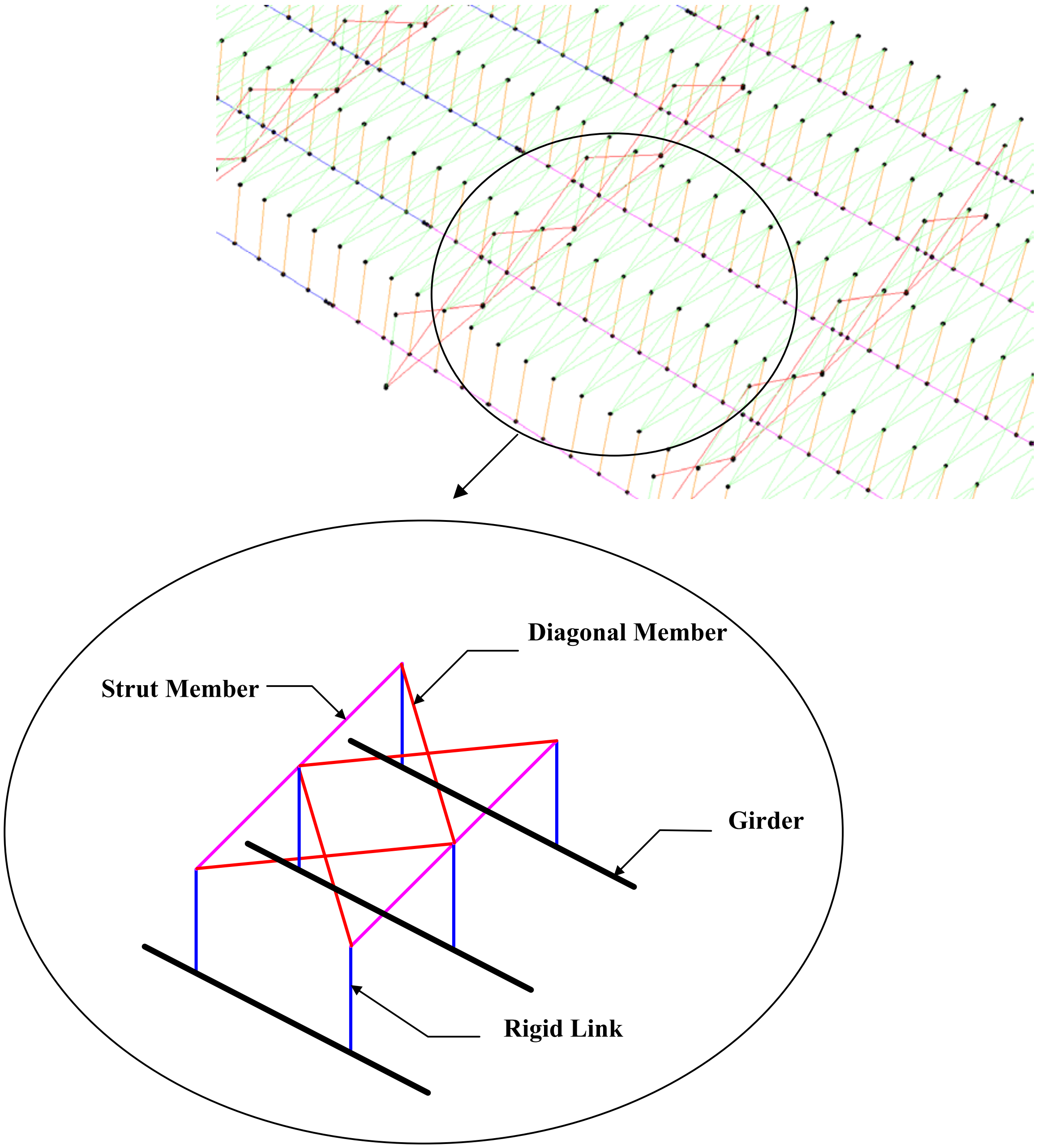


Figure 4-15 Frame Elements as SIP Forms

The section properties of strut and diagonal members were assigned to the frame element by the result of the calculation that Whisenhunt (2004) presented in his study.

4.4.3.3.2 Tension-Compression composite with Bending Shell Element

Shell element was used to represent the tension, compression and bending behavior of the SIP form. Shell element was connected to the rigid link element to transfer forces among each girder. (Detail of rigid link connected between girders and SIP forms included in section 4.4.3.3.2.4). The width of each shell element was set to equal to the cover width of the SIP form, while the length of SIP form was assumed equal to the girder spacing of bridge structure as shown in Figure 4-16.

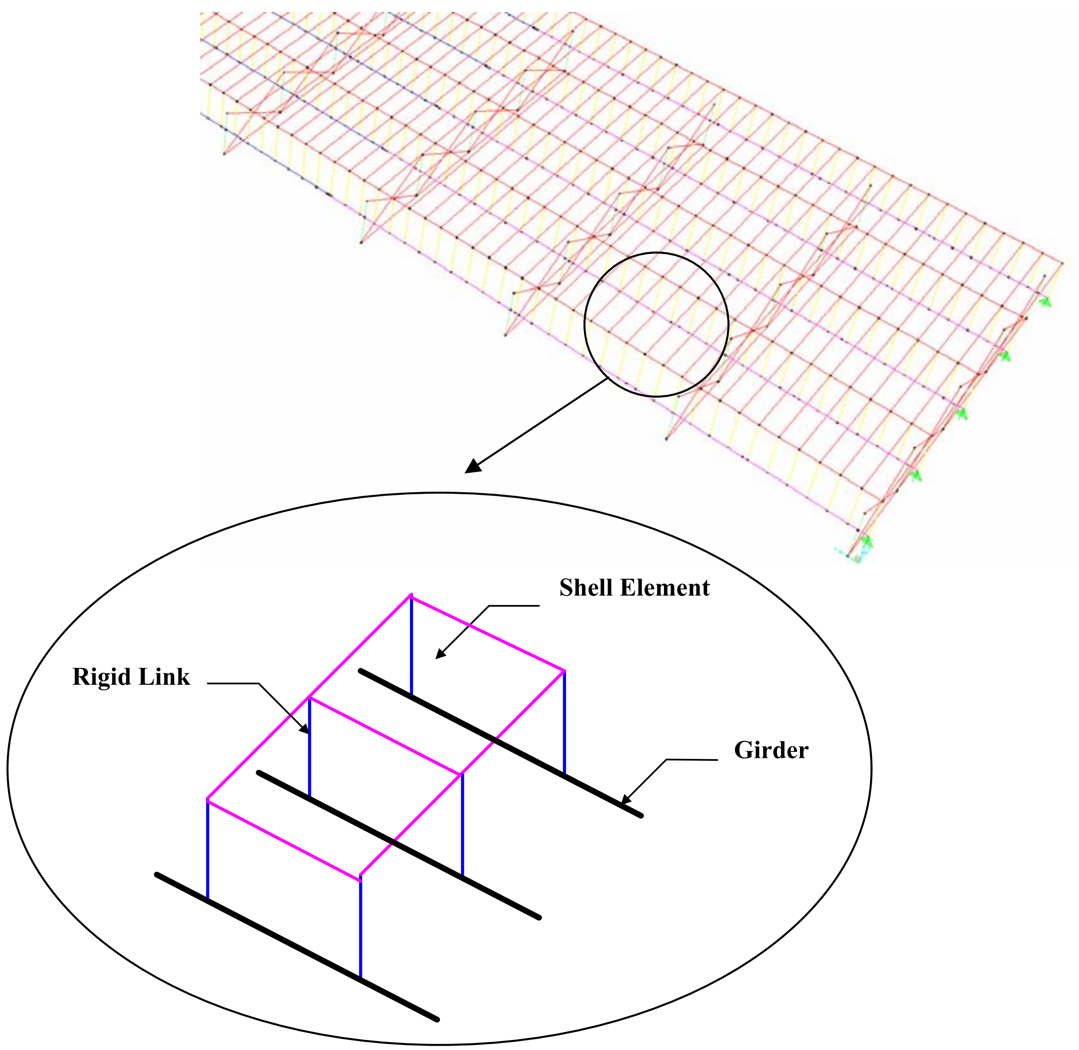


Figure 4-16 Shell Elements as SIP Forms

Shell element in SAP 2000 separates the calculation for stiffness into two categories; axial plus shear stiffness and bending stiffness. Therefore, two kinds of thickness need to be assigned. Thickness (th) is required to calculate axial and shear stiffness and thickness of bending (thb) is required for bending stiffness calculation. The sample of thickness calculation was performed in Appendix F.

The real SIP forms are a non-isotropic plate. To represent behavior of the SIP form more accurately, SIP forms properties were divided into three components, axial stiffness, shear stiffness and bending stiffness in order to obtain the different section properties of the orthotropic shell element.

4.4.3.3.2.1 Axial Stiffness of Stay-in-Place Form

SAP 2000 assigned the local axis to the shell element compared with the SIP form as shown in the Figure 4-17.

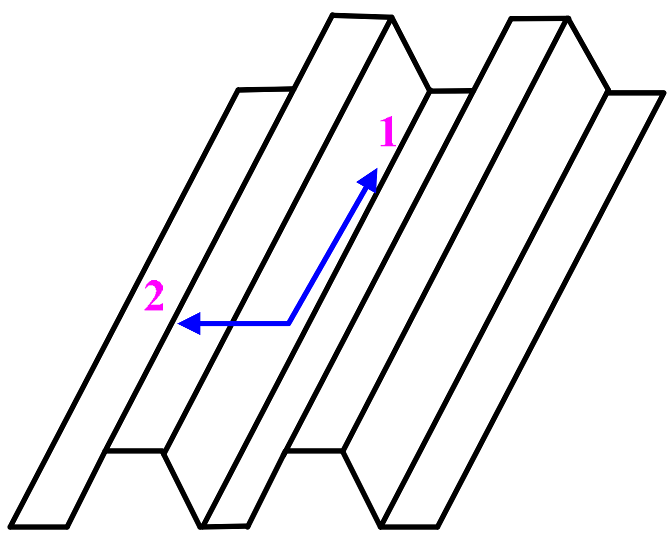


Figure 4-17 SAP Local Axis Direction 1-2 Compared with SIP Form

The axial stiffness of the SIP form model in the direction 1-1 was calculated by using the modification factor in direction 1-1 (f_{11}) multiply to the axial stiffness calculated from thickness (th) of the shell element. Therefore, factor f_{11} was a proportion between the thickness of the shell element (th) and the real thickness of SIP form. For the direction 2-2, three-dimensional SIP form analytical model was created in SAP 2000 to compare with the shell element analytical model. The simulated SIP form model was subjected to the point load to record the deflection in the direction 2-2. By applying the same load condition, the elastic modulus of the shell element was varied to receive the same deflection. Figure 4-18 shows the pictures of the simulated SIP form analytical model compared with the shell element analytical model subjected to the applied point loads. The proportion between elastic modulus of shell element and elastic modulus of SIP form was used as the modification factor in direction2-2 (f_{22}).

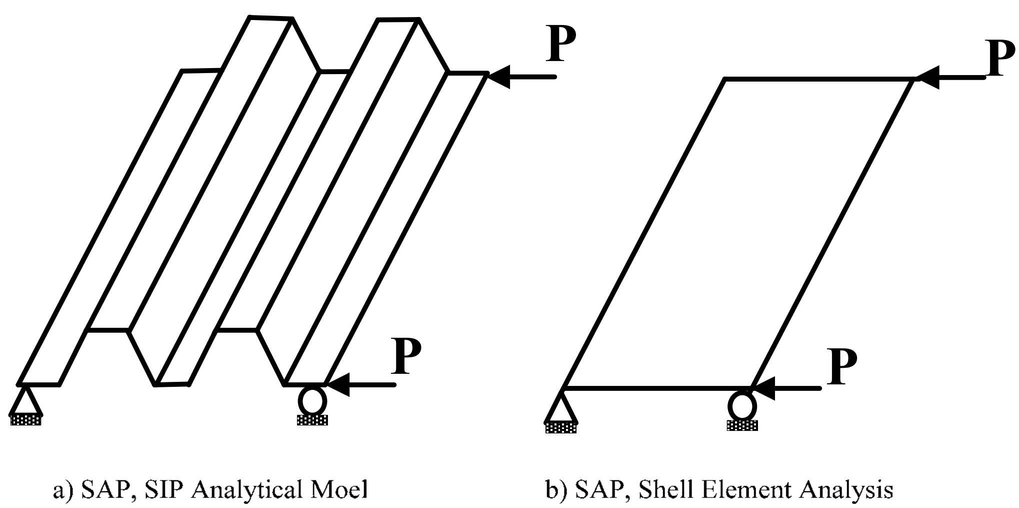


Figure 4-18 SAP Models of Simulated SIP form and Shell Element under Applied Load

4.4.3.3.2.2 Shear Stiffness of SIP form

Jetann et al. (2002) performed experimental tests that measured the diaphragm shear strength of SIP form systems that utilized several different connection details. A value of shear stiffness (G'=11 kips/inch) was measured for the SIP form system with the SIP connection detail similar to what has been found using by the bridges in this study and was used as the basis for the shear properties in the SAP models. The shear stiffness was used in hand analysis to obtain the reference vertical deflection.

By using SAP to 2000 create a shell element in the truss frame and apply the vertical load as shown in the Figure 4-19, the appropriate shear modulus was assigned to the element to obtain the vertical deflection similar to the deflection from hand analysis. The proportion between the shear modulus obtained from SAP 2000 analysis and the shear modulus of the regular steel plate was used to be modification factor for shear stiffness (f_{12}).

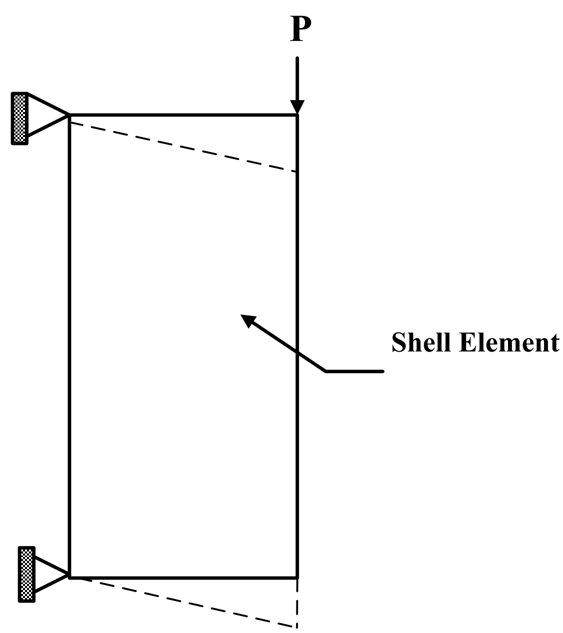


Figure 4-19 SAP, Shell Element Analysis for f_{12}

4.4.3.3.2.3 Bending Stiffness of SIP form

Figure 4-20 shows the direction of m_{11}, m_{22} and m_{12} of the orthotropic shell element. Since m11 is calculated based on the cross section of the shell element similar to what we used for the calculation of the thickness of bending, the modification factor m11 is set equal to one. For m22 and m12, the bending stiffness in direction 2-2 and torsional stiffness is varied on the thickness of member to the third (t^3), so the proportion to the third of the thickness of bending (thb) and the real thickness of SIP was used in the modeling.

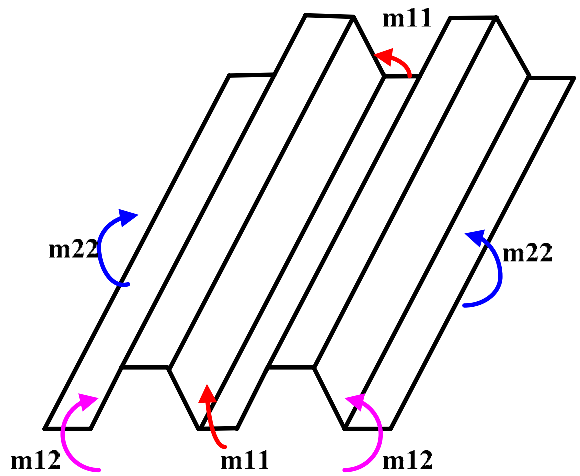


Figure 4-20 SAP, Moment Direction

The sample of the calculation and summary tables of SIP form properties used in the modeling are presented in Appendix G.

4.4.3.3.2.4 Rigid Link Element 2 (RL2)

The rigid link elements were also used to connect the shell element to girders. This connection was named as "RL2". Because of the limitation of the different bending stiffness of the elements connected to each other in SAP 2000, different stiffness from RL1 was assign to the RL2 used to connect girders and SIP form in order to prevent the calculation analytical error. Similar approach to RL1 but the flexural rigidity compared to girder instead of member of cross frame, the flexural rigidity of RL2 larger than girder's about fifty times was used in this study. Figure 4-21 shows the location of RL1 and RL 2 in the bridge model.

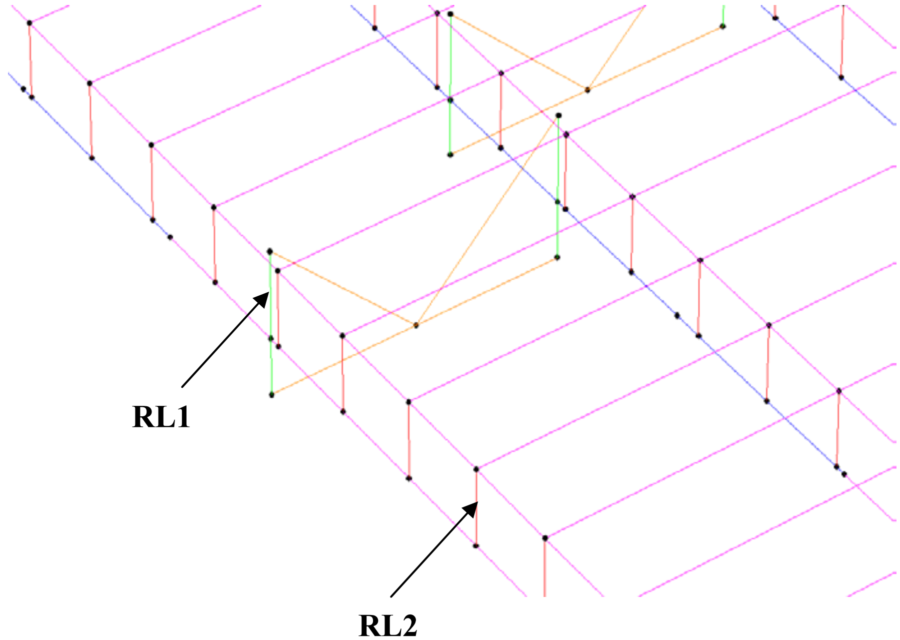


Figure 4-21 Location of RL1 and RL2

4.5 Composite Action

Since concrete deck placement in Bridge 10 had two stages. It was believed that the composite action between concrete deck and girders in the first pour affected the vertical deflection behavior of the bridge girders in the second pour. To represent the composite action between the concrete deck and girders in SAP modeling, plate girders were subjected to the vertical loads in two steps. First, the girders were loaded by wet concrete weight only on the first pour location. Second, the additional moment of inertial due to the cross section area from concrete deck were added to the girders on the first pour location. Then, the girders were subjected to the vertical load only on the second pour position. The vertical deflections from stage one and stage two loading were recorded and super imposed in order to obtain the total vertical deflections of the bridge models.

4.6 Load Calculation and Application

Calculation of the dead load due to the weight of concrete deck was performed base on the field measurement of the thickness of the concrete bridge deck. These dimensions included the depth of the concrete slab between each girder and the overhang width for each exterior girder. Loads were calculated for each interior girder based on the slab thicknesses and tributary widths equal to the girder spacing obtained from the construction plans. The tributary width for the exterior girders included the overhang width and one-half of the typical girder spacing. Steel bars, SIP forms, screeding machine and Labor were not included in the non-composite dead load. These loads were applied to the girders as line loads loaded on the top of the frame elements in SAP models.

4.7 Results

4.7.1 Summary of SAP Deflections

4.7.1.1 Different SAP Modeling Results of Eno River Bridge

Table 4-1 contains the summary of the mid-span deflection predicted by SAP with different modeling methods of Eno River Bridge. Appendix A contains all of the relevant results, pictures and graphs for Eno River Bridge.

Table 4-1 Summary of Mid-Span SAP Deflections of Eno River Bridge (inches.)

Girder	2D	3D (No SIP)	3D (Frame SIP)	3D (Shell SIP)
G1	8.99	9.64	9.66	9.66
G2	8.99	9.35	9.30	9.30
G3	8.99	9.03	8.92	8.92
G4	8.99	8.66	8.53	8.53
G5	8.99	8.26	8.11	8.11

The deflections in Table 4-1 have been plotted to show the deflected shapes of mid-span cross-section of each modeling method. (see Figure 4-22).

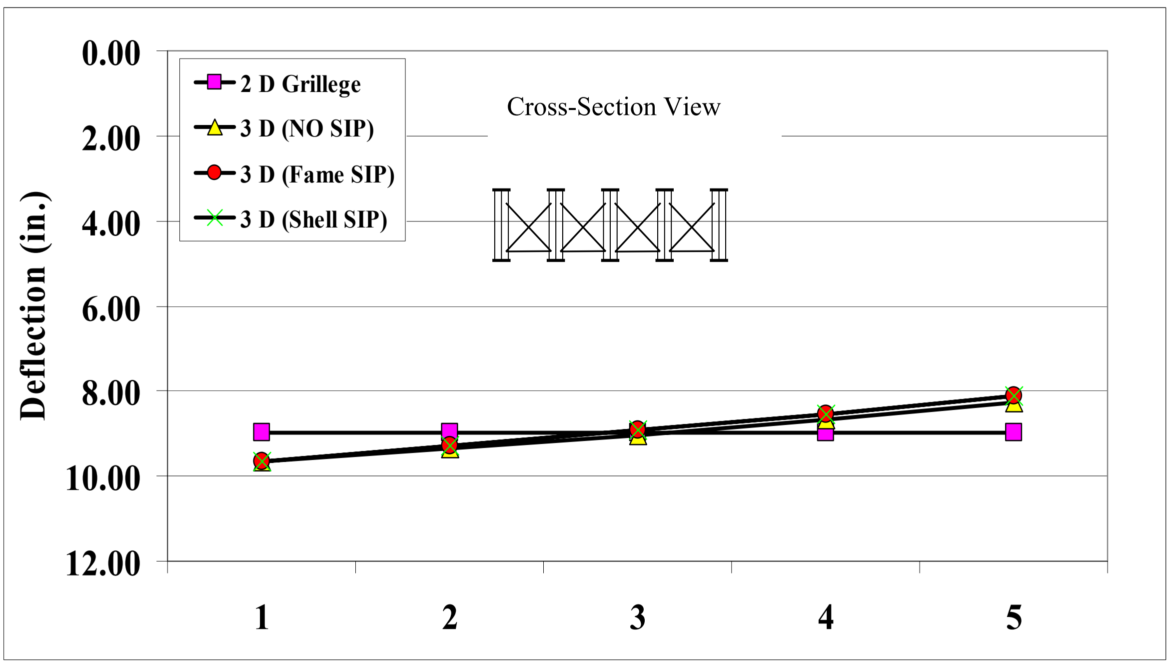


Figure 4-22 Plot of Mid-Span SAP Deflections of Eno River Bridge

The SAP prediction with different modeling methods plotted in Figure 4-22 indicates that the cross-sectional deflected shape of Eno River Bridge linear across its cross-section with the largest deflection occurring at exterior girder one from three of the modeling methods. However, deflected shape from two-dimensional grillage modeling method is flat along the cross section.

4.7.1.2 Different SAP Modeling Results of US29

Table 4-2 contains a summary of the quarter-span deflection predicted by SAP with different modeling methods of US29. Appendix B contains all of the relevant results, pictures and graphs for US29.

Table 4-2 Summary of Mid-Span SAP Deflections of US29 (inch.)

Girder	3D (No SIP)	3D (Frame SIP)	3D (Shell SIP)
G1	5.02	5.16	5.38
G2	5.68	5.65	5.54
G3	5.78	5.78	5.64
G4	5.68	5.68	5.50
G5	35.02	5.13	5.35

The deflections in Table 4-2 have been plotted to show the deflected shapes of mid-span cross-section of each modeling method. (see Figure 4-23).

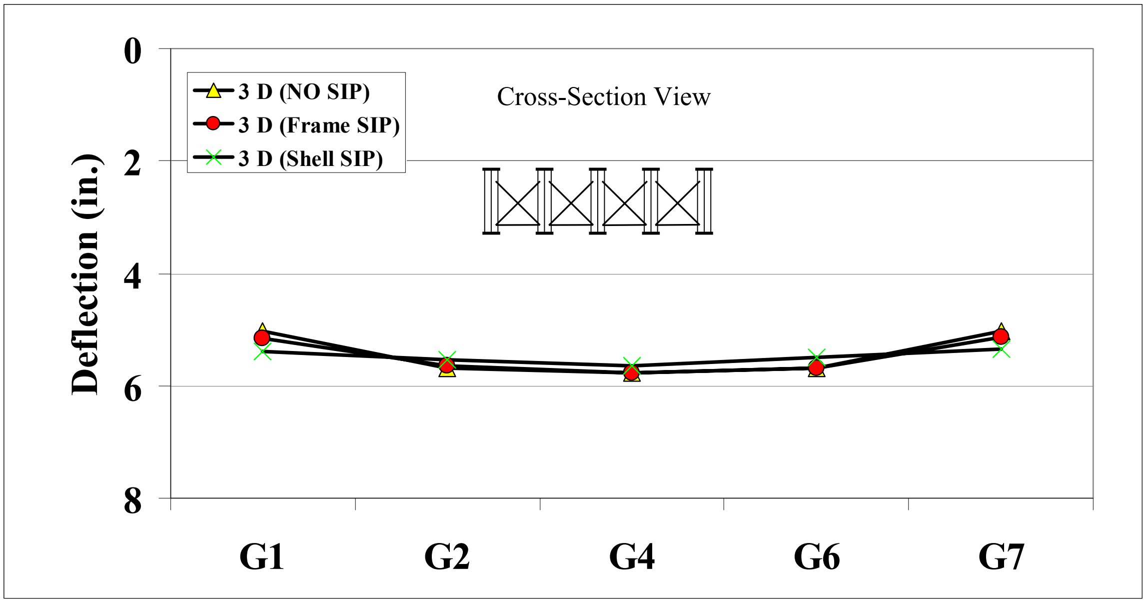


Figure 4-23 Plot of Mid-Span SAP Deflections of US29

4.7.1.3 SAP Three-Dimensional Models with No SIP Forms

Table 4-3 contains of the summary of the mid-span deflection predicted by SAP of each bridge. SIP forms were still not included in the model. The other relevant results and pictures of each model are summary in Appendix A-E.

Table 4-3 Summary of Mid-Span SAP Deflections (NO SIP) of each Bridge (inch.)

Bridge	Eno River	US 29	Bridge 8	Bridge 10 A	Bridge 10 B	Wilmington St.
G1	9.64	5.02	4.18	1.89	0.94	4.04
G2	9.35	5.68	4.73	2.09	1.05	5.64
G3	9.03	5.78	4.90	2.10	1.05	5.91
G4	8.66	5.68	4.90	1.91	0.93	5.83
G5	8.26	5.02	4.73	NA	NA	5.60
G6	NA	NA	4.18	NA	NA	NA

The deflections in Table 4-3 have been plotted to show the deflected shapes of mid-span cross-section of each bridge model (see Figure 4-23).

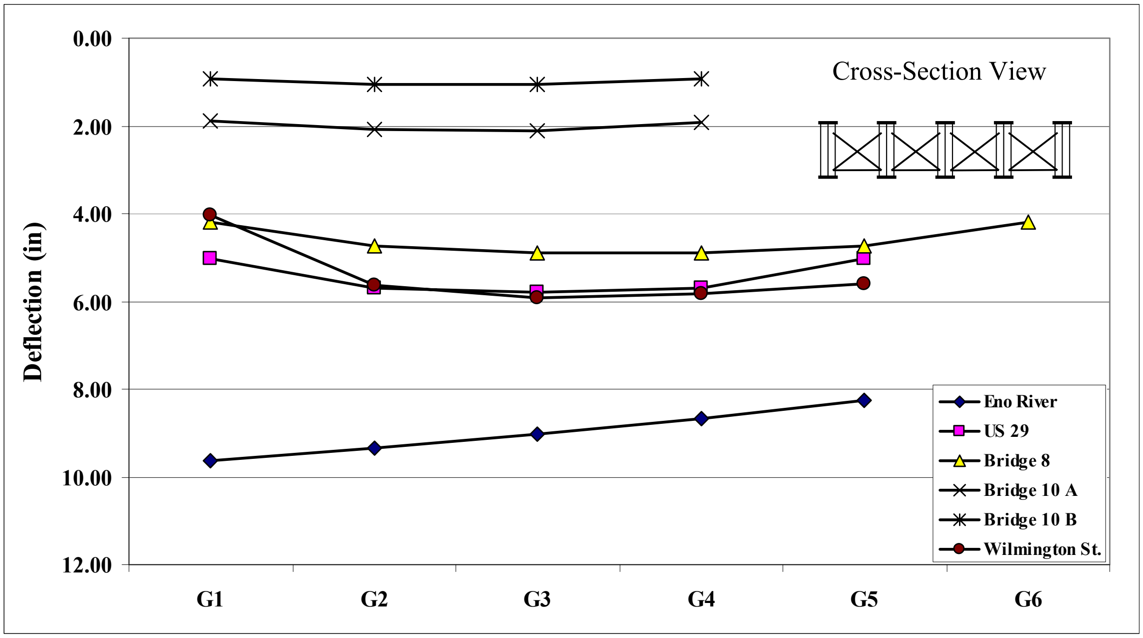


Figure 4-24 Plot of Mid-Span SAP Deflections (SIP Form not Included)

The SAP deflections plotted in Figure 4-24 indicate that the cross-sectional deflected shape is similar for each of the skewed bridges. It is apparent in the figure that the exterior girders deflected the least amount and the middle interior girders deflected the most. The other two girders next to the mid girder have almost the same amount of the vertical deflection value as the mid girder due to the same amount of the vertical loads. The mid-span deflected shape for the non-skewed bridge (Eno River) linear across its cross-section with the largest deflection occurring at exterior girder one.

4.7.1.4 SAP Three-Dimensional Models with Shell Elements as SIP Forms

Table 4-4 contains a summary of the mid span deflection predicted by SAP of each bridge. SIP forms were included in each model. Again, the relevant results and pictures of each model were included in the Appendix A-E.

Table 4-4 Summary of Mid-Span SAP Deflections (SIP) of each Bridge (inch.)

Bridge	Eno River	US 29	Bridge 8	Bridge 10 A	Bridge 10 B	Wilmington St.
G1	9.66	5.38	4.25	1.88	1.08	4.44
G2	9.30	5.54	4.31	2.01	1.16	4.46
G3	8.92	5.64	4.35	2.02	1.16	4.59
G4	8.53	5.5	4.35	1.92	1.05	4.84
G5	8.11	5.35	4.31	NA	NA	5.32
G6	NA	NA	4.26	NA	NA	NA

The deflections in Table 4-4have been plotted to show the deflected shapes of mid-span cross-section of each bridge model (see Figure 4-25).

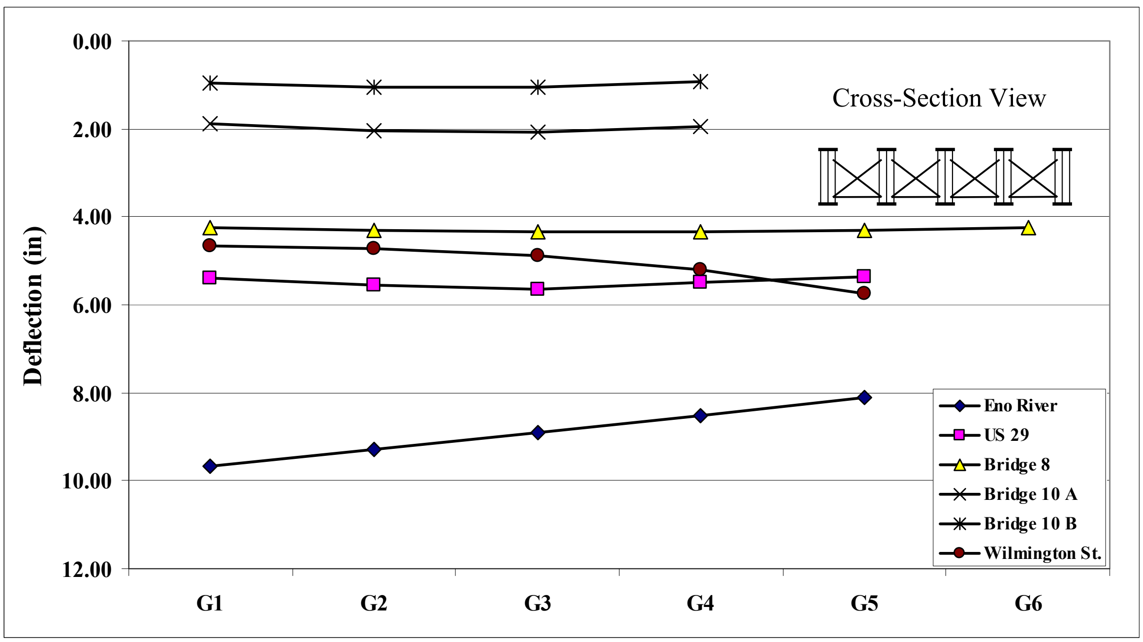


Figure 4-25 Plot of Mid-Span SAP Deflections (SIP Form Included)

According to the Figure 4-25 and as previously discussed in section 4.4.3.2, vertical deflection behavior of the bridge models were affected by the presence of the SIP Forms. The large effect might be seen in Wilmington Street Bridge. Instead of having a "bowl" deflection shape, the deflection shape was flatted and the maximum vertical deflection occurred at the exterior girder. For Bridge 8, Bridge 10 and US 29, the presence of the SIP forms flatted the deflection shapes by reducing the maximum deflections of the exterior girders and increased the deflection of the interior girders. However, the deflected shape for the non-skewed Eno River bridge model was not affected by the presence of the SIP forms. A comparison of each bridge and the verification of the modeling methods will be discussed in Chapter 5.

4.7.1.5 Effect of Composite Action to Modeling Results of Bridge 10

Since Bridge 10 had two steps of the concrete deck placement as mention in section 3.3.5, the composite action between the girder and concrete deck was accounted to the vertical behavior of the bridge structure. Table 4-5 and Table 4-6 contain the mid-span deflection predicted by SAP of Bridge 10 with composite action.

Table 4-5 Summary of Mid-span SAP Deflections of Bridge 10 Span A (inch.)

Point	SAP			SAP (SIP)		
	Pour 1	Pour 2	Total	Pour 1	Pour 2	Total
G1	-0.73	2.57	1.84	-0.74	2.62	1.88
G2	-0.78	2.83	2.05	-0.75	2.76	2.01
G3	-0.77	2.83	2.06	-0.75	2.77	2.02
G4	-0.71	2.58	1.87	-0.72	2.64	1.92

Table 4-6 Summary of Mid-span SAP Deflections of Bridge 10 Span B (inch.)

Point	SAP			SAP (SIP)		
	Pour 1	Pour 2	Total	Pour 1	Pour 2	Total
G1	1.55	-0.45	1.10	1.54	-0.46	1.08
G2	1.67	-0.48	1.19	1.63	-0.47	1.16
G3	1.66	-0.49	1.17	1.62	-0.46	1.16
G4	1.50	-0.46	1.04	1.53	-0.48	1.05

The comparison between the result with and without the composite action will be plotted and discussed in Chapter 5.

4.7.2 A Sensitivity Study of the Bending Stiffness of Rigid Link Element (RL1 & RL2)

Stiffness of the rigid link element is an important parameter in a bridge model that has a significant effect to the vertical deflection behavior. Since large different between bending stiffness of the adjacent frame elements can cause the modeling analytical error in SAP2000, two elastic modulus values need to be assigned to the rigid link elements (RL1 and RL2).The first type of rigid link element (RL1) is the rigid link connected between girders and cross-frame members. Second type (RL2) is the rigid link element connected between girders and SIP forms. To conduct a sensitivity of the bending stiffness of rigid link elements affected to the vertical deflection of the bridge model, two case studies were made by applying the different elastic modulus values to the rigid link elements then compared the vertical deflections. The first case study is the sensitivity of bending stiffness of RL1. The different bending stiffness compared to the stiffness of the cross-frame element was applied to the RL1. Figure 4-26 shows the plot of the deflection of middle girder of the Eno River Bridge with different stiffness of RL1.

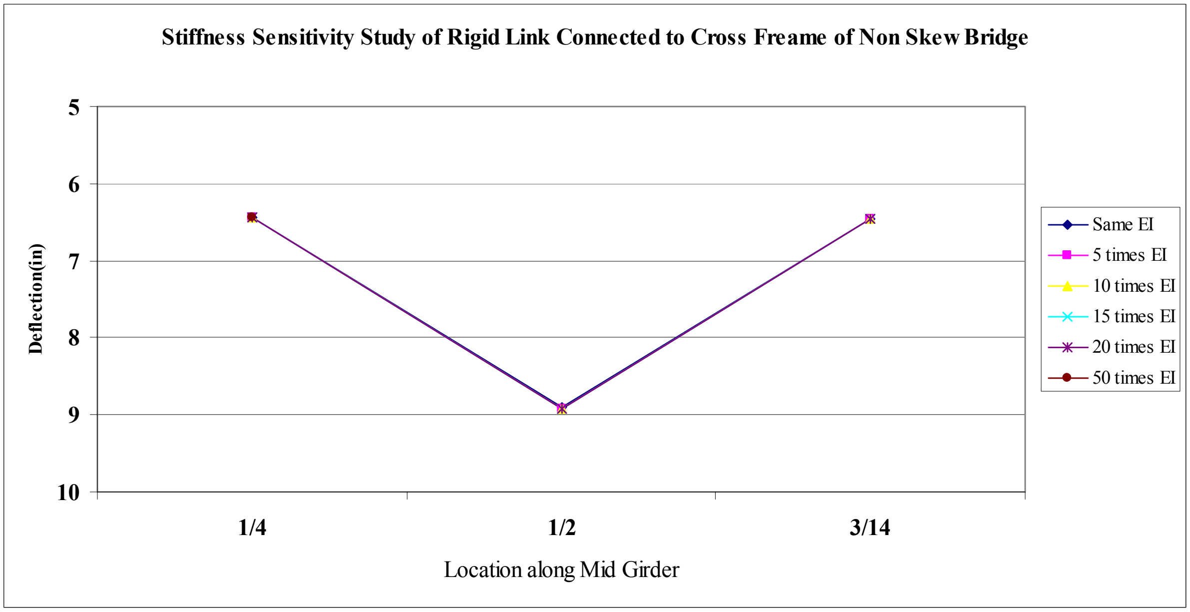


Figure 4-26 Plot, Deflection of Middle Girder with Different Bending Stiffness of RL 1 (Non-Skewed Bridge)

Figure 4-27 shows the plot of the deflection of the middle girder of the high skewed Bridge, Wilmington Street Bridge.

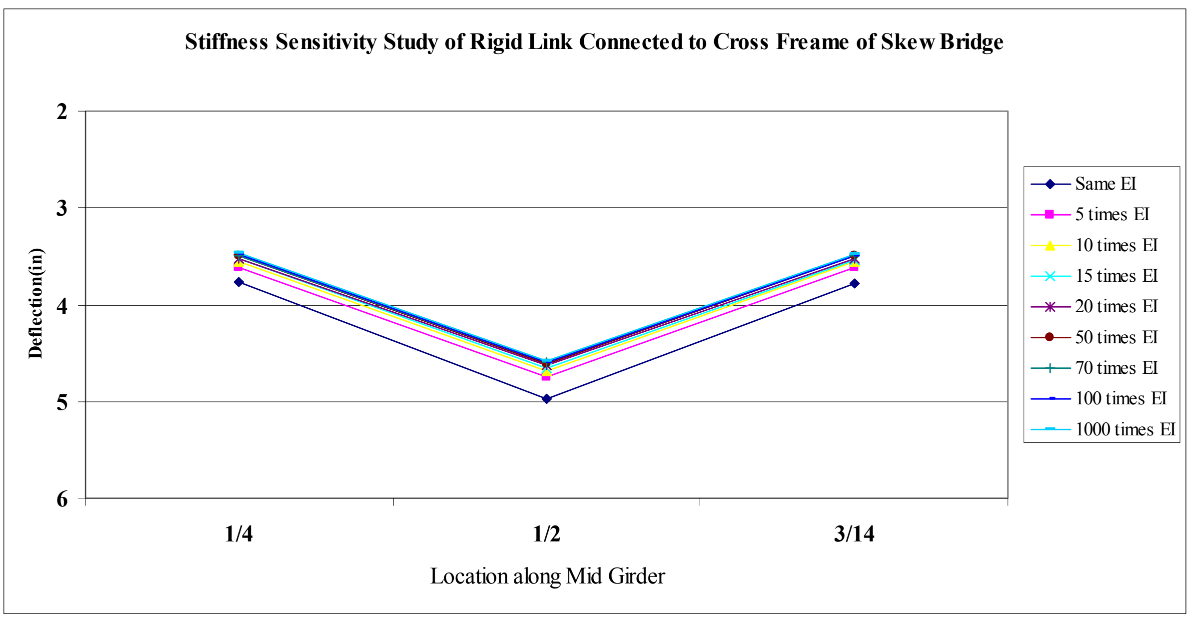


Figure 4-27 Plot, Deflection of Middle Girder with Different Bending Stiffness of RL 1 (Skewed Bridge)

According to Figure 4-26 and Figure 4-27, it obviously shows the difference between the vertical deflection behavior of non-skewed and skewed bridge affecting by the different stiffness of RL1. For the non-skewed bridge, there is no difference in the vertical deflection behavior with the different stiffness of RL1. For the skewed bridge, greater stiffness makes the model stiffer. It is believed by the author that the load distribution among each girder connected by cross frames becomes significant factor affecting to the deflection behavior in the skewed bridge. When the vertical loads try to distribute from girder to girder through the cross frames, greater stiffness in rigid link element makes the model stiffer and deflects less. In addition, stiffness of the rigid link approximately seventy times over the stiffness of the cross-frame member has no significant effect to the vertical deflection.

In the second case study, the effect of the different bending stiffness of RL 2 compared to the girder was investigated. Figure 4-28 shows the deflection of the non-skewed bridge with different bending stiffness of RL2 compared to the stiffness of girder and Figure 4-29 shows the deflection of the skewed bridge with different bending stiffness of RL2 compared to the bending stiffness of the bridge girder.

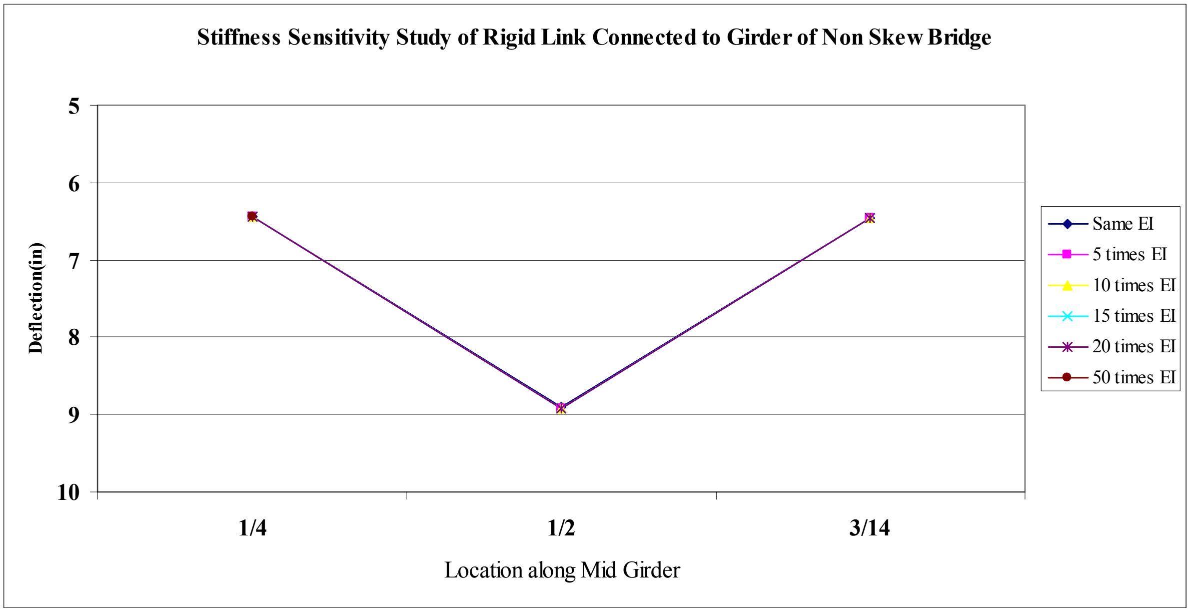


Figure 4-28 Plot, Deflection of Middle Girder with Different Bending Stiffness of RL 2 (Non-Skewed Bridge)

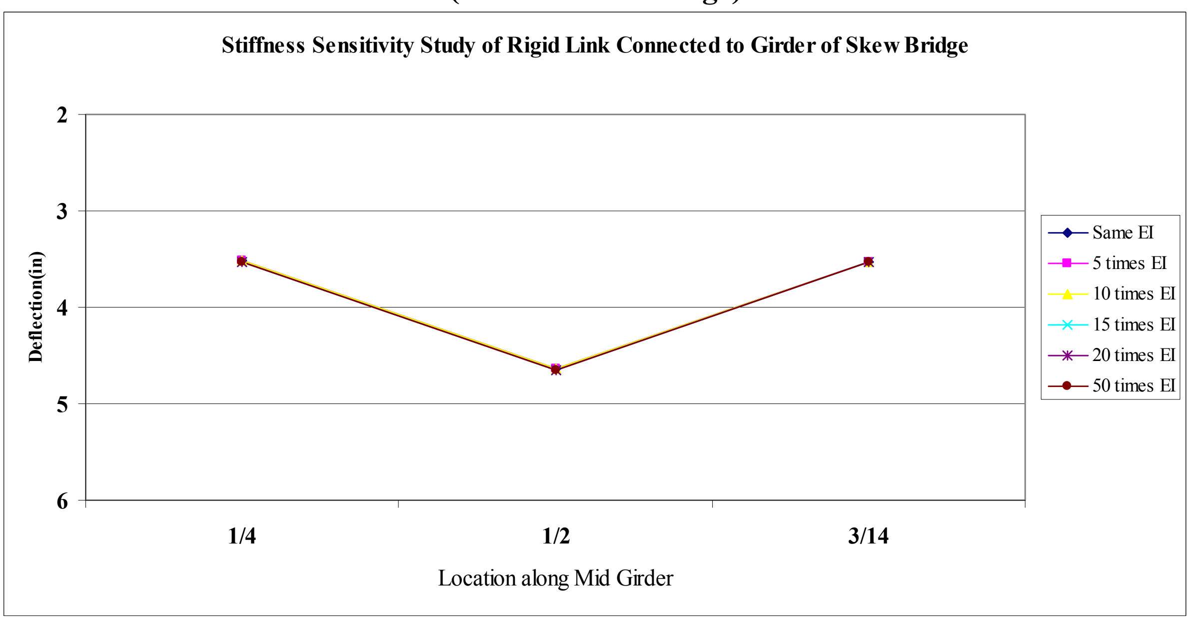


Figure 4-29 Plot, Deflection of Middle Girder with Different Bending Stiffness of RL 1 (Skewed Bridge)

It can be seen in Figure 4-28 and Figure 4-29 that the bending stiffness of the RL2 in non-skewed or skewed bridge has no effect to the vertical deflection of the bridge model. The stiffness of rigid link defines the amount of composite action. RL2 is used to connect between girders and SIP forms. In this case, the composite action between girder and the deck form is very small and can be negligible. The forces from girders to the deck forms are shared by numerous of rigid link elements. The internal forces in each rigid link become insignificant and have no effect from the different bending stiffness.

4.8 Overview and Summary

The steps involved in the development of the simplified modeling method by using SAP 2000 have been outlined and discussed. The SAP model was creates systematically by starting at the single girder model. To verify the modeling method, results from single girder models were compared with the classical hand calculations based on the beam theory and results from ANSYS finite element models. Next, the multi girders were created and connected to each other by cross frames and diaphragms. Two types of modeling methods for cross frame system were created. Load sharing ability study was conducted to ensure both modeling methods. Then, other components of the bridge structures were included in the models, such as boundary condition and SIP form.

It is apparent that SIP forms have a significant effect to the vertical deflection behavior of the bridge structure. A summary of the mid-span deflections predicted by SAP two-dimensional and three-dimensional models with and without SIP form was also included in this chapter.

Comparison and analysis of the SAP deflections to the field measured non-composite deflections and ANSYS deflections was performed to investigate how well the modeled bridge behavior correlates with what was observed in the field and ANSYS results. The detail of comparison and analysis is included in Chapter 5

Chapter 5 – Comparison/Analysis of Results

5.1 Overview

Whisenhunt (2004) compared the deflections between the predicted from single girder line models and field measurement results in his study. He concluded that the predicted results from the method commonly using to predict non-composite deflections in the bridge plan do not correlate with the bridge behavior occurred during the construction. The plots between predicted deflections and measured deflections from each bridge clearly indicate this problem.

In an effort to create a simplified modeling method to predict the non-composite dead load deflections of the bridge structure, SAP 2000 was used to create four different modeling methods. By comparing the results from each modeling method to the measured results of two sample bridges, non-skewed and skewed bridge, only two methods were chosen to create models for all of the single span bridges. The development of SAP models and a summary of deflections were discussed in detail in Chapter 4. ANSYS results from Whisenhunt (2004) and Fisher (2005) were included in this study in order to verify the modeling method in term of complexity and effectiveness. A comparison between SAP deflections, ANSYS deflections and field-measured deflections is also included herein.

One continuous span bridge model was also created in order to see the limitation of the simplified modeling method. By including the composite action between concrete deck and girders in the model, a comparison of the deflections from SAP, ANSYS and field measurement is also presented in this chapter.

5.2 SAP 2000 Single Girder Line Deflections vs. Field Measured Deflections

An investigation of the correlation of the predicted deflections in the bridge plans and measured deflections was conducted in order to indicate the problem as mentioned in chapter one more thoroughly. A single girder line model was created for the middle girder of each bridge by using the details provided in the bridge plan. Frame elements were used to create the steel plate girder connected by nodes placed at points where the cross-section geometry of the steel plate girder changed and at locations where measured results were recorded. As mentioned in chapter 3, since the measured deflections were recorded due to the weight of concrete slab and buildups, the girder was subjected to the vertical load due to the concrete deck only. The self weight of the frame element also must be set equal to zero. This modeling approach was verified by Whisenhunt (2004). He concluded that this modeling method is similar to the approach has been used to predict the non-composite dead load deflection in the bridge plan.

Figure 5-1 presents the comparisons between deflected shapes from single girder models and the measured deflected shapes of each single span bridge. Table 5-1 contains the summary of the average ratios between predicted deflections by single girder model and measured deflections.

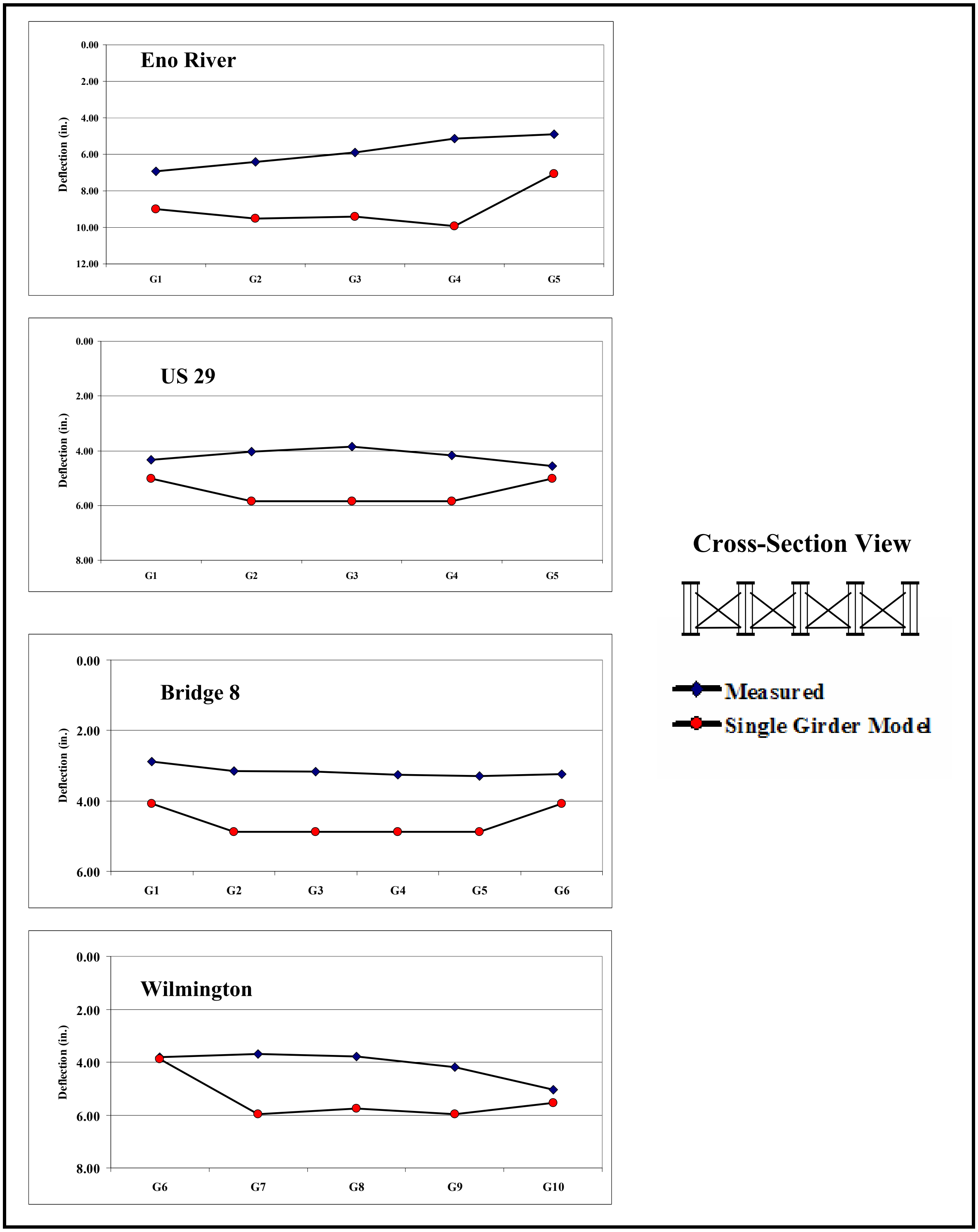


Figure 5-1 Single Girder Line Deflections vs. Measured Deflections at Mid Span

Table 5-1 Ratios of SAP 2000 Single Girder Deflections to Measured Deflections at Mid Span

Average Ratio between Predicted and Measured at Mid Span				
	Eno	**US 29**	**Bridge 8**	**Wilmington St.**
Exterior	**1.37**	**1.13**	**1.10**	**1.06**
Interior	**1.67**	**1.45**	**1.15**	**1.52**

As illustrated in Figure 5-1, the SAP predicted deflections for the middle girder are larger than the measured deflections. In addition, the deflected shapes of the predicted are markedly different from the measured. “Bowl Shape” is apparent in predicted deflection shapes of every bridge model, whereas “Inverted Bowl” shape were apparent from the field measured in two of the skewed bridges, US29 and Wilmington Street Bridge. From those results, it can be concluded that the single girder model is not effective in prediction the girder deflection in skewed bridges.

From the ratios in Table 5-1, it apparent that the prediction using single girder line for interior girder is less than using for the exterior girder. It can be explained that the interior girders connected to adjacent girders by cross frames in both sides, whereas only one side for the exterior girder. The effect of not having cross frame as a transferring load distribution factor affects the interior girder more than exterior girder.

As an effort to investigate the predicted deflections by single girder model compared to measured results, it can be concluded that the predicted method generally used to create the camber table in the bridge plan do not correlate well with the bridge behavior that was observed in the field. Regardless of the effects of skew angle, cross-frame stiffness and stay-in-place form, the single girder modeling methods tend to over predict the non-composite dead load deflections. Therefore, three-dimensional simplified models included cross frame and SIP forms were created in this study. A comparison between deflections from developed models in this study, deflection from ANSYS finite element models and field-measured deflections will be in the following sections.

5.3 Verification of Modeling Methods

In the process of modeling development, four different modeling methods with different complexity were created; two-dimensional grillage model, three-dimensional model without SIP, three dimensional model with frame SIP forms, and three-dimensional model with shell SIP forms. In an effort to verify the modeling methods, measured deflections from Eno River Bridge as non-skewed bridge and US 29 as skewed bridge were used to compare the modeling results.

5.3.1 Eno River Bridge Verification

Figure 5-2 is a plot of deflection shapes of different modeling methods compared to the measured deflection shape at 1/4 span of Eno River Bridge (Non Skewed Bridge).

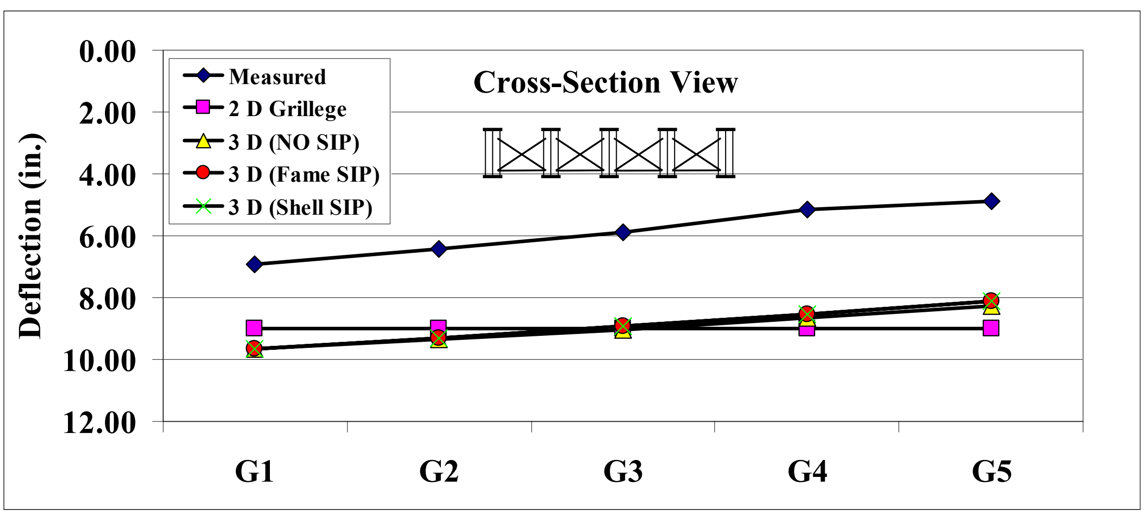


Figure 5-2 SAP, Deflection Shapes Compared with Measured at Mid Span of Eno River Bridge

According to Figure 5-2, there is a good agreement between deflection shapes of three types of three-dimensional modeling methods and deflected shape from field measurement. The deflected shapes were linear from one exterior girder to another one. However, the deflected shape from two-dimensional grillage model by using single simulated beams as the entire cross frames had a flat deflected shape. By using the two-dimensional grillage model, the predicted deflections in each girder at the same cross section were equal. Due to the over predicted bending stiffness of the real cross frames, using simulated beams as cross frame made the entire bridge model became over stiff and acted as a unit plate. This indicates that a simulated single beam is not an appropriate modeling method for the entire cross frame.

5.3.2 US 29 Verification

Figure 5-3 is a plot of deflection shapes of different modeling methods compared to the measured deflection shape at 1/4 span of US 29 (Skewed Bridge).

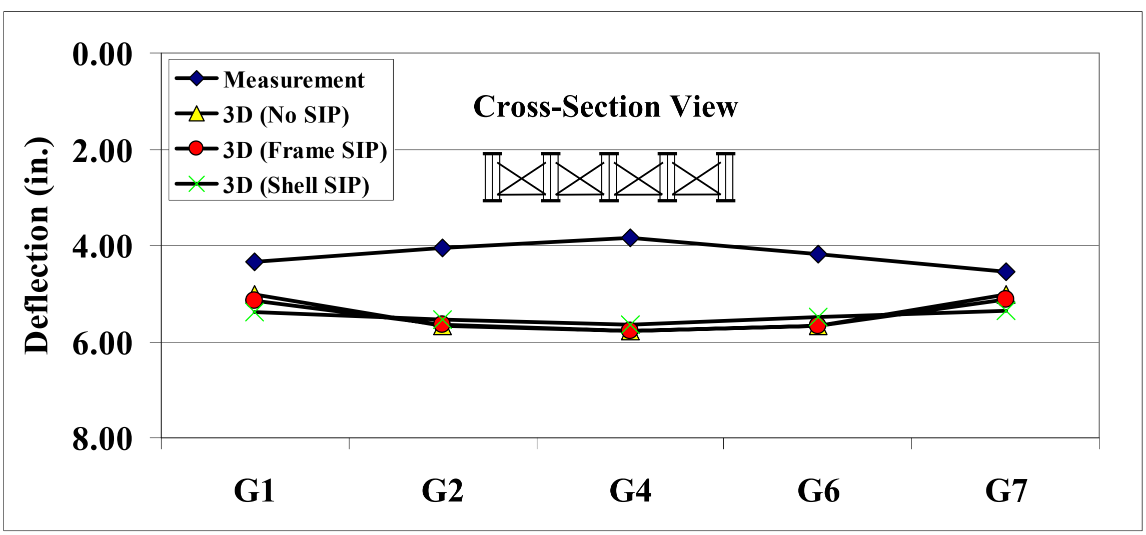


Figure 5-3 SAP, Deflection Shapes Compared with Measured at Mid Span of US 29

According to Figure 5-3, the deflection shapes of the skewed bridge models are slightly different from each other. The deflection shapes all three modeling methods gave the same shapes those are “Bowl” shape. However, the deflection shapes from three-dimensional model included shell SIP seem to be the most effective modeling method. Although the deflection shape from model included shell element as SIP forms did not correlate perfectly with the measured deflection shape, the deflections of exterior girders were decreased and minimized the maximum deflections of interior girders which likely similar to the measured.

From the sample verification of modeling methods in non-skewed and skewed bridge, three dimensional without SIP and three-dimensional with shell SIP modeling methods seem to be the most effective methods in terms of the simplest and the most accurate, respectively. Therefore, only these two methods were used to create the SAP models for all bridges. The summary of the comparison between deflections from SAP models, ANSYS models and field measurement will be in the following sections.

5.4 SAP 2000 Deflections V.S. ANSYS Deflections and Field Measurement Deflections

5.4.1 General

Several bridge parameters and approaches, discussed in chapter 4, were explored in the development of the SAP simplified bridge model. Four of the modeling methods were created to model the entire bridge structures in this study. However, after the verification of modeling results, only two modeling methods, three-dimensional model without SIP and three-dimensional model with shell SIP, have been selected. The following sections will cover comparison between the field-measured deflections, deflections predicted by ANSYS finite element model and deflections predicted by SAP models.

5.4.2 *SAP Three-Dimensional Model Deflections (NO SIP) V.S. Measured Deflections & ANSYS (NO SIP) Deflections*

Four single-span bridges were modeled using single girder frame elements as steel plate girders connected to the adjacent girders by three-dimensional simulated cross frames without including SIP forms. In order to compare and verify this modeling method, deflections from SAP models were compared to field measured results. In addition, deflections from ANSYS finite element models (without SIP) were also included in the comparison. The deflections predicted from these models had been plotted to compare the deflection shapes from the field measurements. The comparisons were made at 1/4 and mid span as shown in Figure 5-4 and Figure 5-5 Since the behavior of deflections at 3/4 along the span length predicted by SAP models and the field measurement deflections were similar to those of 1/4 span, the comparison at those locations were not included.

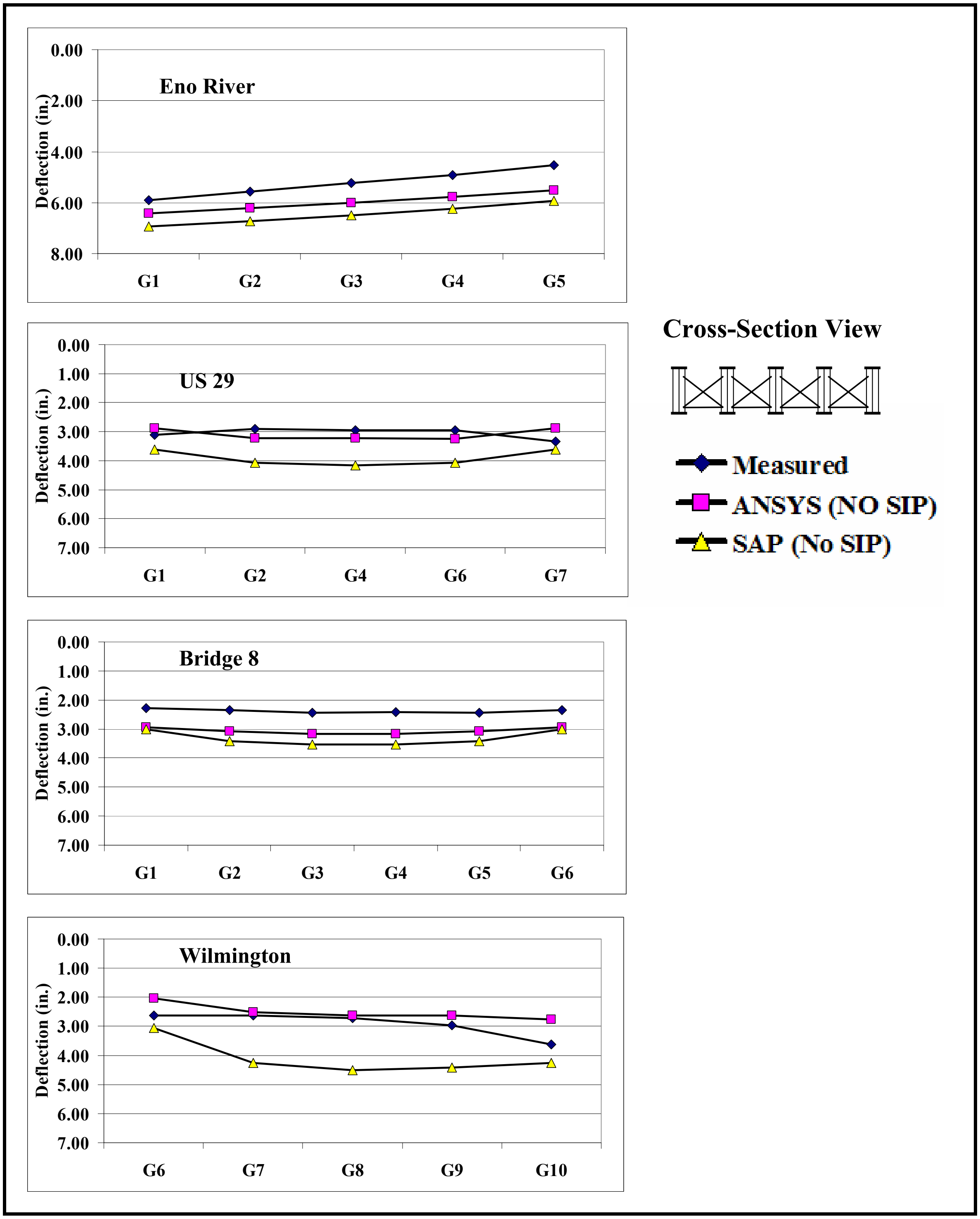


Figure 5-4 SAP Deflections (NO SIP) vs. Measured and ANSYS Deflections at 1/4 Span

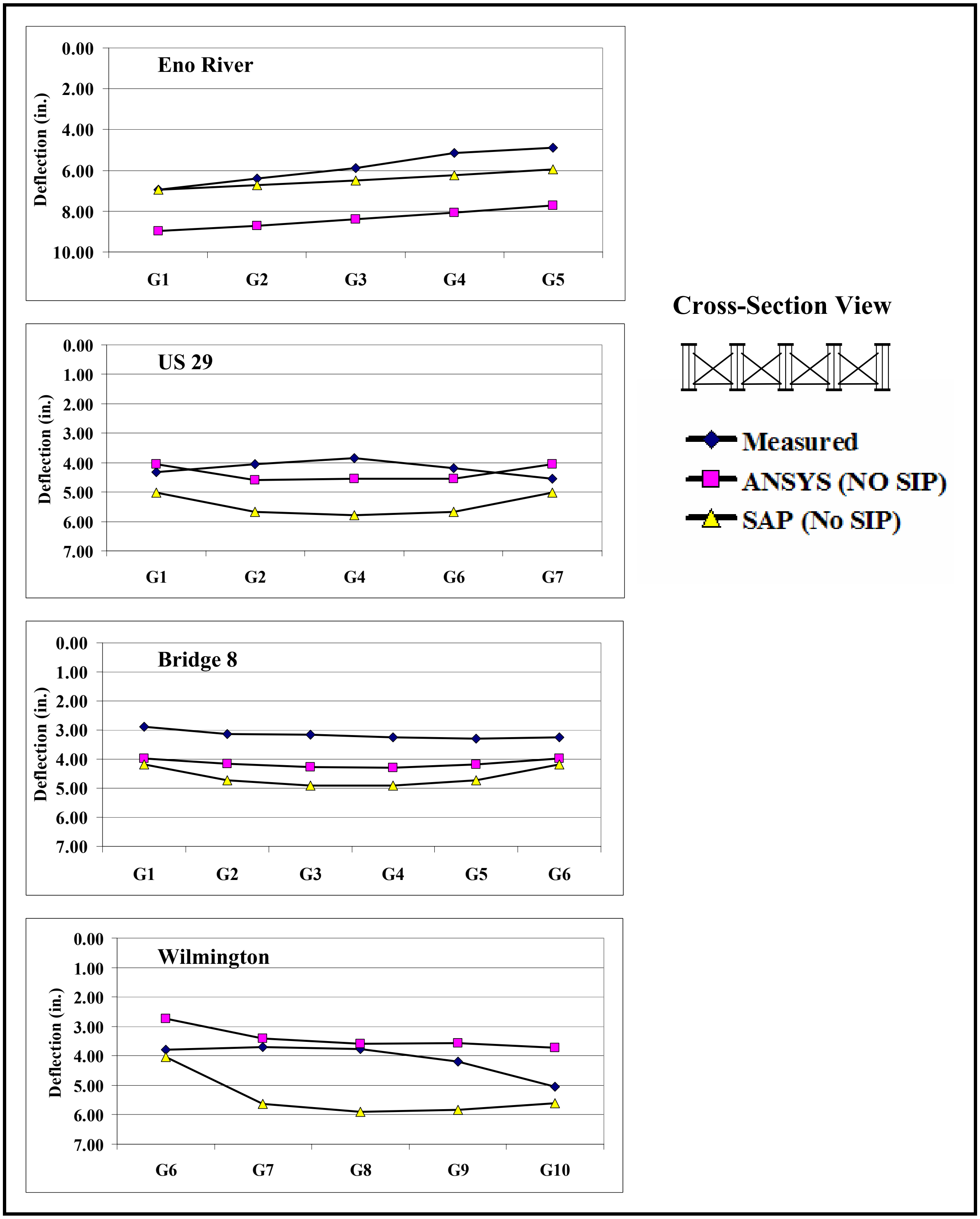


Figure 5-5 SAP Deflections (NO SIP) vs. Measured and ANSYS Deflections at Mid Span

According to Figure 5-4 and Figure 5-5, there is a good agreement between the cross sectional deflection shapes in non-skewed bridge (Eno River Bridge). This is an indication that SIP forms do not account on non-skewed bridge. SAP predictions for Eno River Bridge are larger than the measured result approximately thirty-four percent for both exterior and interior girders.

For skewed bridges, the predictions from SAP three-dimensional modeling without SIP forms were larger than the measured. Deflected shapes from three-dimensional without SIP modeling method have a poor correlation with the measured results. From US 29 and Wilmington Street comparison, deflected shapes from SAP modeling were “Bowl” shape, whereas the measured deflection shapes were “Inverted Bowl” shape. According to Table 5-2, the magnitude of the average ratios of exterior girders of US 29 is twelve percent same as the average ratios of exterior girder of Wilmington Street. For interior girders, the average ratio of US 29 is approximately forty-one percent and fifty-four percent for Wilmington Street Bridge. However, in Bridge 8 comparison, the deflection shape of SAP modeling was similar to the measured. The average ratio for exterior of Bridge 8 is thirty-eight percent for exterior girders and forty-eight percent for interior girders.

According to Table 5-3, results from SAP models seem to have a constant difference compared with ANSYS results. It is obviously seen from the Figure 5-4 and Figure 5-5 that the deflected shapes from SAP and ANSYS without SIP are similar. SAP modeling method predicted deflections of exterior girders better than interior girders compared to ANSYS prediction. The average ratios of exterior girder are lower than the average ratios of interior girder. (see Table 5-3, Eno River Bridge, Bridge 8 and Wilmington)

Table 5-2 Ratios of SAP 2000 (No SIP) to Field Measurement Deflections

Bridges	Location	Girder A	Girder B	Girder C	Girder D	Girder E	Girder F
Eno	1/4	1.17	1.21	1.24	1.27	1.31	NA
	1/2	**1.39**	**1.46**	**1.54**	**1.69**	**1.69**	**NA**
	3/4	1.15	1.19	1.23	1.25	1.30	NA
US 29	1/4	1.16	1.41	1.41	1.38	1.08	NA
	1/2	**1.16**	**1.40**	**1.50**	**1.36**	**1.10**	**NA**
	3/4	1.13	1.41	1.50	1.34	1.09	NA
Bridge 8	1/4	1.32	1.46	1.46	1.47	1.41	1.29
	1/2	**1.45**	**1.51**	**1.55**	**1.50**	**1.43**	**1.29**
	3/4	1.27	1.42	1.52	1.43	1.41	1.23
Wilmington St.	1/4	1.16	1.62	1.66	1.49	1.17	NA
	1/2	**1.06**	**1.52**	**1.56**	**1.39**	**1.11**	**NA**
	3/4	1.08	1.58	1.60	1.40	1.15	NA

Table 5-3 Ratios of SAP 2000 (No SIP) to ANSYS (No SIP) Deflections

Bridges	Location	Girder A	Girder B	Girder C	Girder D	Girder E	Girder F
Eno	1/4	1.08	1.08	1.08	1.08	1.08	NA
	1/2	**1.07**	**1.07**	**1.07**	**1.07**	**1.07**	**NA**
	3/4	1.08	1.08	1.08	1.08	1.08	NA
US 29	1/4	1.25	1.26	1.29	1.26	1.25	NA
	1/2	**1.24**	**1.24**	**1.27**	**1.25**	**1.24**	**NA**
	3/4	1.25	1.24	1.28	1.26	1.25	NA
Bridge 8	1/4	1.03	1.11	1.12	1.12	1.11	1.03
	1/2	**1.05**	**1.14**	**1.14**	**1.14**	**1.13**	**1.05**
	3/4	1.03	1.11	1.12	1.12	1.11	1.03
Wilmington St.	1/4	1.49	1.70	1.71	1.68	1.54	NA
	1/2	**1.47**	**1.65**	**1.64**	**1.64**	**1.51**	**NA**
	3/4	1.51	1.71	1.69	1.70	1.53	NA

5.4.3 SAP Three-Dimensional Model Deflections (Shell SIP) V.S. Measured Deflections & ANSYS (SIP) Deflections

Bridge models were developed by including SIP forms by using orthotropic shell element. This approach was done in an effort to develop simplified modeling method for skewed bridge structures. Details of the method used to represent the SIP forms were discussed in Chapter 4. Again, the comparison between SAP deflections and measured deflection were made to verify the modeling method. Deflections from ANSYS finite element models (with SIP) were also included to investigate how well the simplified modeling method can correlate with the real bridge behavior compared to the finite element modeling method. Figure 5-6 and Figure 5-7 contains plots with the SAP deflections including SIP forms versus measured and ANSYS (included SIP) deflections.

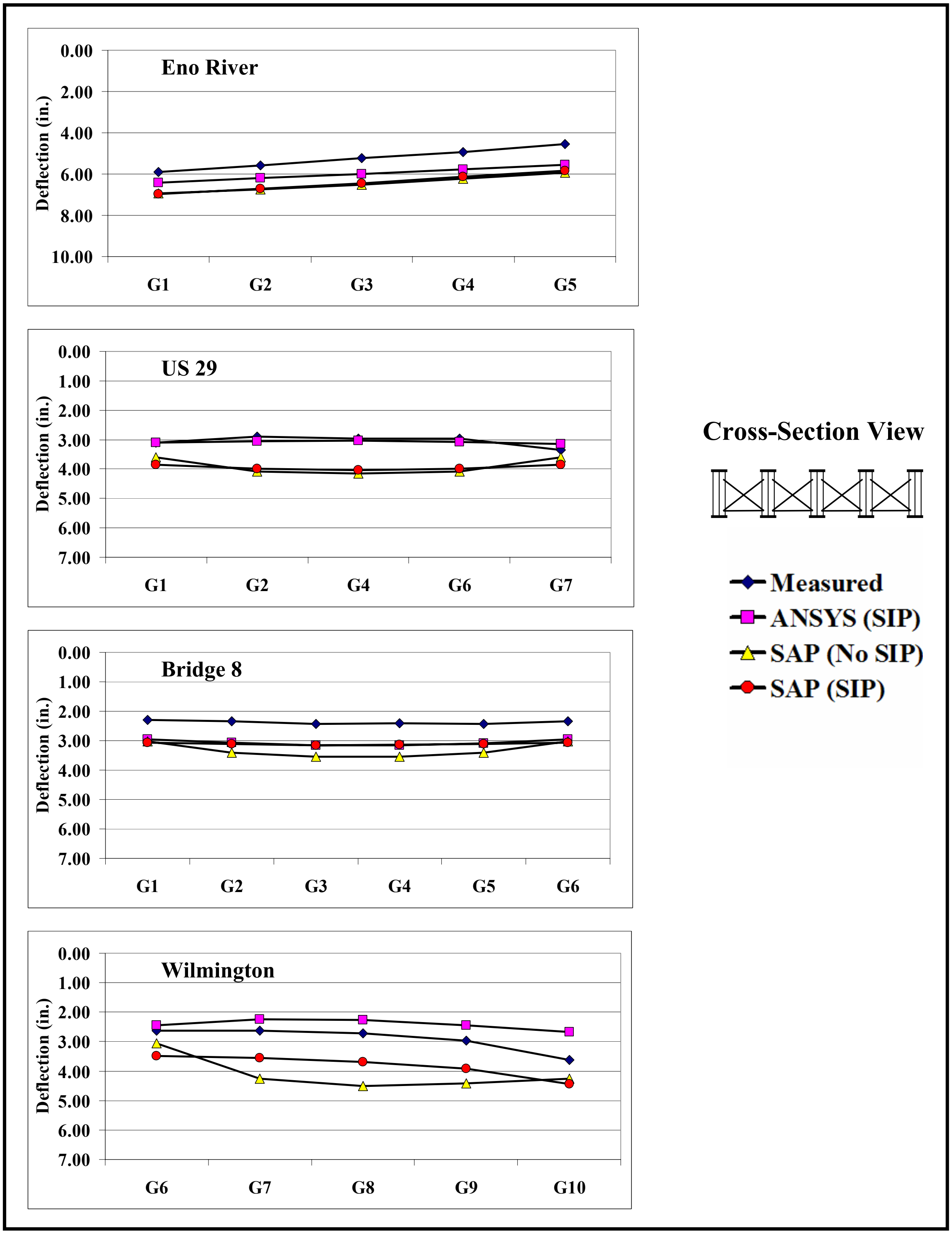


Figure 5-6 SAP Deflections (SIP) vs. Measured and ANSYS Deflections at 1/4 Span

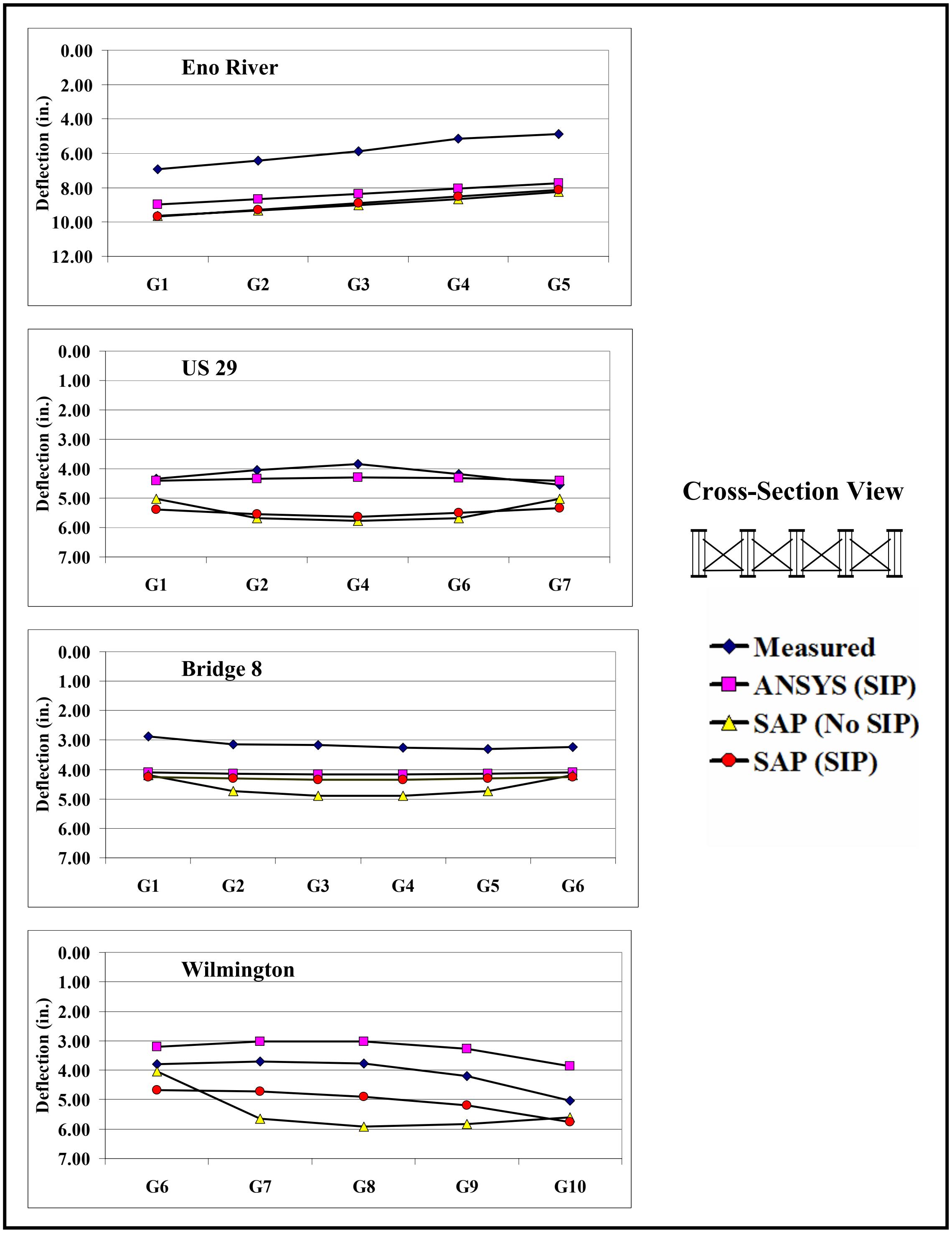


Figure 5-7 SAP Deflections (SIP) vs. Measured and ANSYS Deflections at Mid Span

According to Figure 5-6 and Figure 5-7, there is a good agreement in the deflected shapes predicted by SAP for the Eno River Bridge, Bridge 8 and Wilmington Street Bridge.

The deflected shape in Eno River Bridge is similar to the results from the model without SIP. It emphasizes that SIP forms are not active in non-skewed bridge. However, by including SIP forms in skewed bridge, it is obviously seen that the deflected shape in US 29 and Wilmington Street Bridge are changed. Instead of having a “Bowl” shape, the deflected shapes of Wilmington Street Bridge is “Inverted Bowl” shape same as the measured. For the US 29, the deflected shape is flatted and likely similar to the measured more than the results from the model not included SIP forms. Similar to US 29 model behavior, deflected shape of Bridge 8 is flatted by minimizing interior girder deflections and increasing the exterior girder deflections.

According to Table 5-4, deflections from the models included SIP forms seem to be more effective. The average ratios of Wilmington Street Bridge is approximately thirty percent for interior girders compared to sixty-eight percent different form results of the model without SIP. For Bridge 8, there is slightly effect of including SIP in the model to the exterior girder. The average difference is approximately thirty-three percent compared to thirty-eight percent from model not including SIP for the exterior girder. The effect of SIP form becomes greater in interior girders, from forty-six percent different decreases to thirty-one percent. Similar to what occur in Bridge 8, the deflections of interior girders of US 29 are more accurate to the measured result after included SIP form.

Compared to ANSYS results, SAP models by including SIP forms have a very good agreement in the deflected shapes in every bridge. The deflected shapes from SAP seem to shift down from deflected shapes from ANSYS. According to Table 5-5, the average ratios seem to be constant along each girder.

The effect of SIP forms in all skewed bridge is obviously seen from the modeling results. The results from SAP modeling indicated that SIP forms participate in the vertical load transferring in skewed bridge. However, SIP forms do not have any account on non-skewed bridge as illustrate in the deflections from Eno River Bridge. The deflections had no significant difference between deflections from with and without SIP forms.

Table 5-4 Ratios of SAP 2000 (Shell SIP) to Field Measurement Deflections

Bridges	Location	Girder A	Girder B	Girder C	Girder D	Girder E	Girder F
Eno	1/4	1.18	1.20	1.23	1.25	1.28	NA
	1/2	**1.39**	**1.45**	**1.52**	**1.66**	**1.66**	**NA**
	3/4	1.16	1.19	1.22	1.24	1.28	NA
US 29	1/4	1.24	1.37	1.37	1.35	1.15	NA
	1/2	**1.24**	**1.37**	**1.47**	**1.32**	**1.17**	**NA**
	3/4	1.21	1.38	1.46	1.29	1.16	NA
Bridge 8	1/4	1.34	1.33	1.30	1.30	1.28	1.31
	1/2	**1.47**	**1.37**	**1.37**	**1.34**	**1.31**	**1.31**
	3/4	1.30	1.29	1.34	1.26	1.29	1.25
Wilmington St.	1/4	1.32	1.35	1.36	1.32	1.23	NA
	1/2	**1.23**	**1.28**	**1.29**	**1.24**	**1.14**	**NA**
	3/4	1.25	1.32	1.33	1.25	1.17	NA

Table 5-5 Ratios of SAP 2000 (Shell SIP) to ANSYS (SIP) Deflections

Bridges	Location	Girder A	Girder B	Girder C	Girder D	Girder E	Girder F
Eno	1/4	1.09	1.08	1.08	1.06	1.05	NA
	1/2	**1.08**	**1.07**	**1.07**	**1.06**	**1.05**	**NA**
	3/4	1.08	1.08	1.08	1.07	1.06	NA
US 29	1/4	1.24	1.30	1.33	1.30	1.23	NA
	1/2	**1.22**	**1.28**	**1.31**	**1.27**	**1.22**	**NA**
	3/4	1.23	1.29	1.32	1.28	1.23	NA
Bridge 8	1/4	1.01	1.02	1.02	1.02	1.02	1.01
	1/2	**1.04**	**1.04**	**1.04**	**1.04**	**1.04**	**1.04**
	3/4	1.01	1.01	1.02	1.02	1.02	1.02
Wilmington St.	1/4	1.43	1.59	1.62	1.60	1.66	NA
	1/2	**1.46**	**1.57**	**1.61**	**1.58**	**1.49**	**NA**
	3/4	1.53	1.62	1.67	1.65	1.48	NA

5.5 Bridge 10 Modeling Results and Comparison

The comparison plots between SAP, ANSYS and measured deflections at different location of bridge 10 were presented in Figure 5-8. SAP model plotted in Figure 5-8 was developed with SIP forms but without the composite action. In order to compare the results based on the similar modeling approach, ANSYS results showed in the plot was also from the finite element model analysis without composite action.

Since Bridge 10 is a continuous span bridge, the vertical deflection behavior found in the field became more complicate and unpredictable. The deflected shapes in Span A are markedly different from ANSYS and measured shapes. The measured and ANSYS deflected shapes seems likely to be “Inverted Bowl” shapes, whereas the deflected shape from SAP predicted is “Bowl” shape. However, there is a good agreement between SAP and ANSYS in span B. The shapes from both predicted models are linear along the cross section but the shapes are still different from the measured.

The composite action was simulated in Bridge 10 as discussed in Chapter 4. Figure 5-9 represents the plot showing the effect of including composite action in the model compared to the measurement and ANSYS results in interior girder (Girder 2). According to Figure 5-9, SAP modeling method has a very good agreement with both ANSYS and measured deflected shapes in pour one. However, the composite action modeling method used in pour two made the model less stiff than ANSYS model and the real bridge structure. After the deflections were super imposed, the deflections from the SAP models are much larger than ANSYS and the measured results in Span A, whereas the deflection in Span B are smaller than the measured and almost equal to the result from ANSYS.

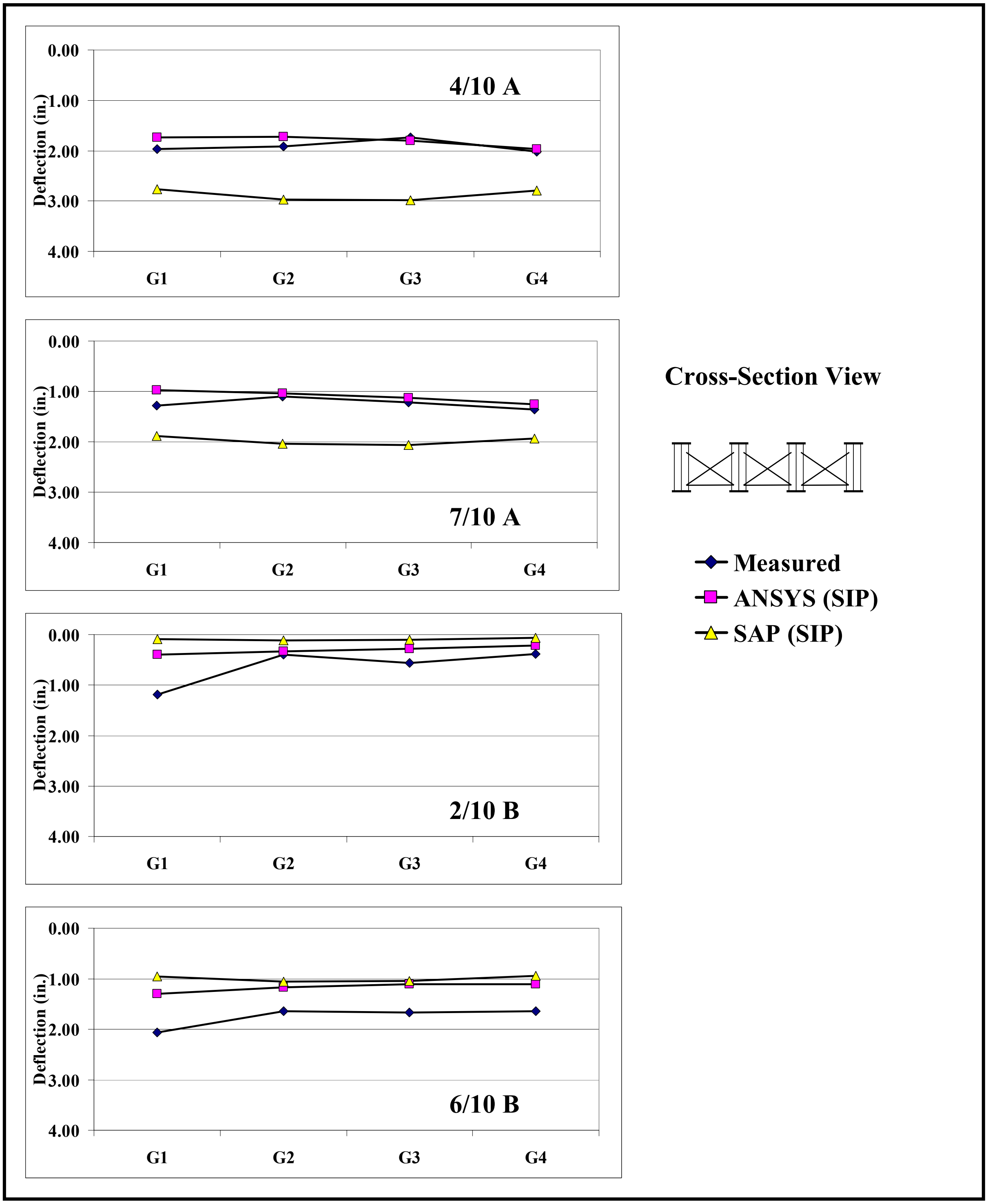


Figure 5-8 SAP Deflections (SIP) vs. Measure and ANSYS Deflections at Each Location of Bridge 10

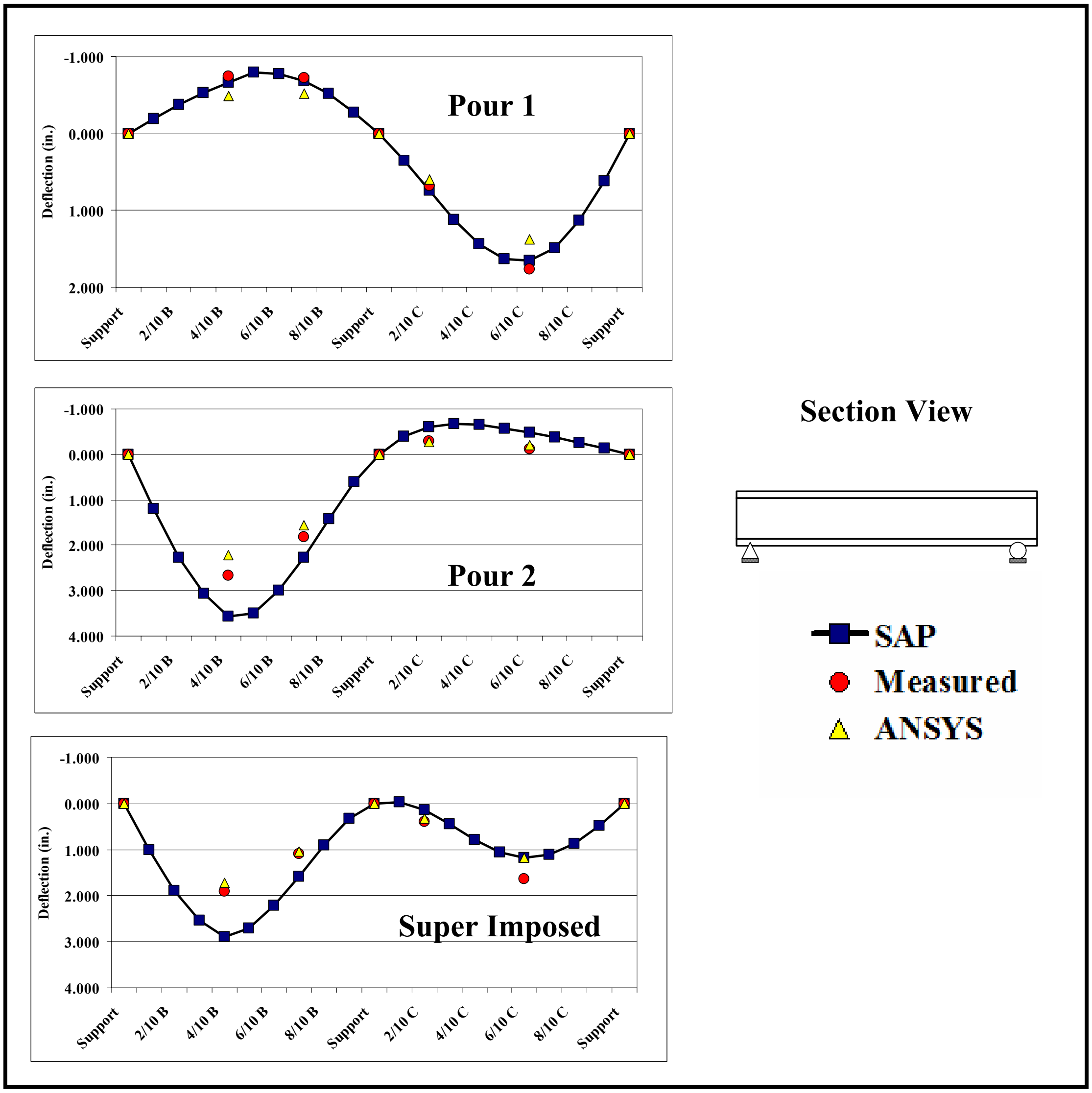


Figure 5-9 SAP Deflections (SIP) vs. Measured and ANSYS Deflections along Girder 2

5.6 Possible Sources of Error

From the comparisons between the results from the SAP prediction and the measured results, it is apparent that the all predicted results are larger than the measured. The possible sources of error are list herein.

1. Since the geometries of the girders were not measured in the field and the modeling was only based on nominal dimension of the bridge girder as provided in the design drawing. The model does not account to the slight

variation of the girder dimension that may have affected the measured behavior.

2. Uncertainty of the applied dead load calculation can introduce slight error between the measured and predicted deflection.

3. Temperature changed due to the air temperature or concrete hydration reaction was not accounted for in the model.

5.7 Summary

The investigations into the use of single girder model predicted the deflection due to the non-composite dead load of steel plate girder bridges proved that model which do not include transverse load distribution parameters, such as cross frame and SIP forms would not provide accurate results. Including cross frame into the three-dimensional SAP models can improve the prediction in non-skewed bridge. However, adding only cross frame to the models seems not to be accurate enough in skewed bridge. It was obviously seen that including SIP form in the skewed bridge model represented bridge behavior more accurately.

SAP simplified modeling methods developed in this study have a good agreement with the measured results and ANSYS results in single span bridge. However, in the continuous, two-span bridge, simplified modeling method is not effective. Composite action was developed to be included in the continuous bridge model. It was found that the composite action has a slight effect on the vertical deflection behavior of the continuous bridge.

Chapter 6 –Recommendations and Conclusions

6.1 General

The observations found during this research are summarized in this chapter. In addition, it contains recommendations for simplified modeling methods, topics for future research and a list of primary conclusions that were made about simplified modeling methods of the non-composite deflections of steel plate girder bridges.

6.2 Observation and Recommendation

During the process of field measurement and development of modeling methods, several observations were made. Based on these observations, recommendations have been made for future research on the non-composite deflection behavior of steel plate girder bridges. The following sections are a discussion of observations and recommendations.

6.2.1 Field Observations and Recommendations

The non-composite deflections of the Wilmington Street Bridge were recorded as a part of this study. As a result, it was observed that the deflected shape at each cross section was markedly different from the deflections predicted by the single girder line model. It was shown that the SIP forms and cross-frame affected the vertical deflection behavior of the bridges. Specifically, the SIP forms were the most significant factor for the skewed bridges. This behavior was later confirmed by the SAP analytical modeling results. Using the SAP 2000 models with SIP forms, deflected shape of the model was similar to the field observed bridge behavior.

Another observation was related to the deflection of the bridge girder changed due to the change of temperature. The bridge girders deflected downward approximately half an inch from the initial reading at about 5 pm to the time concrete started pouring at 8 PM due to the change of air temperature. Since the temperature changed was not included into the primary or secondary factor of this study, the conclusion related to this topic cannot be drawn at the present time. It is recommended that future research of the non-composite deflection behavior of steel plate girder bridges include the effect of temperature change such as effect of air temperature change and the effect of temperature change due to the hydration reaction of cement.

6.2.2 SAP Observation and Recommendation

By using SAP 2000 version 9 creating the simplified models, it is obviously seen that the complexity of the model is less than the finite element model. By using a single frame element to create the entire steel plate girder can reduce time for creating the model compared to using the combination of shell elements in the finite element analysis modeling.

By creating the simplified models with and without SIP form, the results emphasized the importance of SIP form. Including SIP form in skewed bridge models significantly improved accuracy of the prediction. However, the SIP forms had no significant effect to non-skewed bridge. Therefore, it is recommended that SIP form should be included in skewed bridge modeling in order to predict the non-composite deflection more accurately.

From the comparisons of modeling the SIP forms using shell element with and without flexural behavior, the results were more accurate with the flexural behavior included. Orthotropic shell element function in SAP 2000 version 9 is recommended for representing the different stiffness in the different directions of the SIP forms. However, the modeling method used to model SIP form in this study based on the properties from the simulation and the previous testing results without conducting laboratory test on SIP form. It is recommended that future research of the non-composite deflection behavior of steel plate girder bridges include the individual components of bridge testing such as SIP form and cross frame. The flexural and shear behavior of SIP form should be investigated well base on their connections to the steel plate girders for more accurately representing the true deflection behavior of steel plate girder bridges.

6.3 Conclusions

The list below is the primary conclusion drawn from the comparison of field measurement deflections to the predicted deflections from SAP simplified bridge models.

1. Vertical deflections predicted by single girder line model do not correlated well with the measured deflections. In general, the predicted deflection is larger than the measured deflection, especially for interior girder of skewed bridge.
2. In general, three-dimensional SAP model including the cross frame and SIP forms predicts the vertical deflections more accurately than single girder line model. However, deflections predicted by ANSYS agree more closely to the measured deflections than SAP models.
3. By including SIP forms in the SAP simplified models, the predictions are more accurate for the skewed bridges.
4. SAP simplified modeling methods created in this study predicted vertical deflections similar to what was observed in the field and they are adequate for use in prediction the non-composite deflections for single span bridges.

5. Vertical deflections predicted by SAP simplified modeling methods did not correlate well with the measured results in the continuous span bridge.

6.4 Summary

The goal of this study is to create a simplified modeling method to predict the non-composite dead load deflections for steel plate girder bridges. Instead of using a complicated finite element model, simplified models were developed in the limitation of complexity. Description of each bridge in this study, modeling methods, field measured and modeling results were presented.

It was concluded that the single girder line model commonly used to predict the non-composite deflection in bridge plan do not correlate well with the vertical deflections that were observed from the field. The simplified model created in this study predicted vertical deflection similar to the measured deflections in single span bridge. However, deflections predicted by simplified modeling method had poor correlation with the measured deflections in the one continuous span bridge investigated.

REFERENCES

AASHTO/NSBA (2002). *Guidelines for Design for Constructability, G 12.1-2002, Draft for Ballot*, American Association of State Highway and Transportation Officials/National Steel Bridge Alliance.

Bangash, M.Y.H. (1999). *Prototype Bridge Structure: Analysis and Design,* ThomasTelford, Heron Quay, London.

Bishara, A.G., Elmir, W.E. (1990). "Interaction Between Cross Frames and Girders," *Journal of Structural Engineering*, ASCE, 116(5), 1319-1333.

Huang, H., Shenton, H.W., Chajes, M.J. (2004). "Load Distribution for a Highly Skewed Bridge: Testing and Analysis," *Journal of Bridge Engineering,* ASCE, 9(6), 558-562.

Choo, T.W., Linzell, D.G., Lee, J.I., Swanson, J.A. (2005). "Response of Continuous, Skewed, Steel Bridge during Deck Placement," *Journal of Construction Steel Research*, Science Direct, 567-586.

Chung, W., Sotelino, E.D. (2005). "Nonlinear Finite-Element Analysis of Composite Steel Girder Bridges," Journal *of Structural Engineering*, ASCE, 131(2), 304-313.

Ebeido, T., Kennedy, J.B. (1996). "Girder Moments in Simply Supported Skew Composite Bridges," *Journal of Bridge Engineering*, ASCE, 1(1), 37-45.

Elmir, W.E. (1988). "Spatial Interaction between Cross Frames and Girders of Mutti-Stringer Skewed Steel Bridge," M.S. Thesis, Graduate School of the Ohio State University, OH.

Egilmez, O.O., Jetann, C.A., Helwig, T.A. (2003). "Bracing Behavior of Permanent Metal Deck Forms," *Proceedings of the Annual Technical Session and Meeting,* Structural Stability Research Council.

Fisher, S.T. (2005, Draft). "Measurement and Finite Element Modeling of the Non-Composite Deflections of Steel Plate Girder Bridges," M.S. Thesis, Department of Civil, Construction and Environmental Engineering, North Carolina State University, Raleigh, NC.

Gupta, Y.P., Kumar, A. (1983). "Structural Behaviors of Interconnected Skew Slab-Girder Bridges," *Journal of the Institution of Engineers (India), Civil Engineering Division*, 64, 119-124.

Heins, C.P. (1975). *Bending and Torsional Design in Structural Members,* Lexington, M.A.: Lexington Books.

Hilton, M.H. (1972). "Factors Affecting Girder Deflections during Bridge Deck Construction," *Highway Research Record*, HRB, 400, 55-68.

Jetann, C.A., Helwig, T.A., Lowery, R. (2002). "Lateral Bracing of Bridge Girders by Permanent Metal Deck Forms," *Proceedings of the Annual Technical Session and Meeting,* Structural Stability Research Council.

Khaloo, A.R., Mirzabozorg, H. (2003) "Load Distribution factors in Simply Supported Skew Bridges," *Journal of Bridge Engineering,* ASCE, 8(4), 241-243.

Mabsout, M.E., Tarhini, K.M., Frederick, G.R., Tayar, C. (1997). "Finite-Element Analysis of Steel Girder Highway Bridges," *Journal of Bridge Engineering*, ASCE, 2(3), 83-87.

Mabsout, M.E., Tarhini, K.M., Fredericks, G.R., Kesseerwan, A. (1999). "Effect of Multilanes on Wheel Load Distribution in Steel Girder Bridges," *Journal of Bridge Engineering*, ASCE, 4(2), 99-106.

Norton, E.K. (2001). "Response of a Skewed Composite Steel-Concrete Bridge Floor System to Placement of Deck Slab," M.S. Thesis Proposal, Department of Civil and Environmental Engineering, The Pennsylvania State University, University Park, PA.

Norton, E.K., Linzell, D.G., Laman, J.A. (2003). "Examination of Response of a Skewed Steel Bridge Superstructure during Deck Placement," *Transportation Research Record 1845,* Transportation Research Board, Washington D.C.

Padur, D.S. (2004). "Development and Testing of an Enhanced Preprocessor for Creating 3D Finite Element Models of Highway Bridges and a Post Processor for Efficient Result Generation," M.S. Thesis, Department of Electrical, Computer Engineering and Computer Science, University of Cincinnati, Cincinnati, OH.

Wang, L., Helwig, T.A. (2003). "Cross-Frame and Diaphragm Behavior for Steel Bridges with Skewed Supports," *Research Report No. 1772-1, Project No. 0-1772*, Department of Civil and Environmental Engineering, University of Houston, Houston, TX.

Whisenhunt, T.W. (2004). "Measurement and Finite Element Modeling of the Non-Composite Deflections of Steel Plate Girder Bridges," M.S. Thesis, Department of Civil, Construction and Environmental Engineering, North Carolina State University, Raleigh, NC.

Computer and Structural, Inc, (1998). *SAP 2000 Analysis Reference,* Berkeley, California, USA

Samaan, M., Sennah, K., Kennedy, J.B. (2002). "Distribution of Wheel Loads on Continuous Steel Spread-Box Girder Bridges," *Journal of Bridge Engineering,* ASCE, 7(3), 175-183.

Swett, G.D. (1998). "Constructability Issues With Widened and Stage Constructed Steel Plate Girder Bridges," M.S. Thesis, Department of Civil and Environmental Engineering, University of Washington, Seattle, WA.

Swett, G.D., Stanton, J.F., Dunston, P.S. (2000). "Methods for Controlling Stresses and Distortions in Stage-Constructed Steel Bridges," *Transportation Research Record 1712*, Transportation Research Board, Washington D.C.

Appendix A

Deflection Summary for the Eno River Bridge

This appendix contains a detailed description of the Eno River Bridge from previous research by Whisenhunt (2004). It includes bridge geometry, material data, cross-frame type and size, and dead loads calculated from slab geometry. Tables and graphs of the field measured non-composite girder deflections are included.

A summary of the SAP simplified model created for the Eno River Bridge is also included in this appendix. This summary includes a picture of the SAP model, details about the elements used in the model generation, and tables and graphs of the deflections predicted by the model.

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: U-2102 (Guess Road over Eno River, Stage 2)
MEASUREMENT DATE: February 28, 2003

BRIDGE DESCRIPTION

TYPE	One Span Simple	
LENGTH	236.02 ft (71.94 m)	
NUMBER OF GIRDERS	5	
GIRDER SPACING	9.65 ft (2.94 m)	
SKEW	90 deg	
OVERHANG	3.41 ft (G1)	(from web centerline)
BEARING TYPE	Pot Bearing	

MATERIAL DATA

STRUCTURAL STEEL	**Grade**	**Yield Strength**
Girder:	HPS70W (HPS485W)	70 ksi (485 MPa)
Other:	AASHTO M270	50 ksi (345 MPa)

CONCRETE UNIT WEIGHT	120 pcf (nominal)
	117 pcf (measured)
SIP FORM WEIGHT	3 psf (nominal)

GIRDER DATA

LENGTH	236.02 ft (71.94 m)
TOP FLANGE WIDTH	20.08 in (510 mm)
BOTTOM FLANGE WIDTH	22.84 in (580 mm)
WEB THICKNESS	0.55 in (14 mm)
WEB DEPTH	101.58 in (2580 mm)

FLANGES	**Thickness**	**Begin**	**End**
Top:	1.10 in (28 mm)	0.00	118.01 ft (35.97 m)
Bottom:	1.18 in (30 mm)	0.00	58.76 ft (17.91 m)
	1.97 in (50 mm)	58.76 ft (17.91 m)	118.01 ft (35.97 m)

STIFFENERS	
Longitudinal:	PL 0.63" × 6.30" (16 mm × 160 mm)
Bearing:	PL 1.10" × 11.02" (28 mm × 280 mm)
Intermediate:	PL 0.79" × NA (20 mm × NA, connector plate)
	PL 0.47" × 4.724" (12 mm × 120 mm)

CROSS-FRAME DATA

	Type	**Diagonals**	**Horizontals**
END	K	L 5×5×5/16	MC 12×31 (top)
			L 5×5×5/16 (bottom)
INTERMEDIATE	X	L 5×5×5/16	L 5×5×5/16 (bottom)

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: U-2102 (Guess Road over Eno River, Stage 2)
MEASUREMENT DATE: February 28, 2003

DECK LOADS

	Concrete*		Deck Slab**		Ratio
Girder	**lb/ft**	**N/mm**	**lb/ft**	**N/mm**	
G1	770.19	11.24	891.47	13.01	0.86
G2	814.04	11.88	992.20	14.48	0.82
G3	803.76	11.73	992.20	14.48	0.81
G4	848.99	12.39	992.20	14.48	0.86
G5	604.36	8.82	670.83	9.79	0.90

*calculated with measured slab thicknesses
**includes slab, buildups, and stay-in-place forms (nominal)

SLAB DATA

THICKNESS	9.06 in	(nominal)
BUILD-UP	2.95 in	(nominal)
REBAR	**Size**	**Spacing**
LONGITUDINAL	(metric)	(nominal)
Top:	#13	460 mm
Bottom:	#16	220 mm
TRANSVERSE		
Top:	#16	150 mm
Bottom:	#16	150 mm

GIRDER DEFLECTIONS (data in inches, full span concrete deflections less bearing settlement)

MEASURED

	1/4 Span Loading			1/2 Span Loading		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	0.96	0.63	0.66	3.00	2.95	2.44
G2	0.91	0.56	0.59	2.74	2.58	2.21
G3	0.87	0.43	0.56	2.51	2.18	2.01
G4	0.82	0.35	0.55	2.29	1.67	1.83
G5	0.76	0.41	0.50	2.05	1.61	1.61
	3/4 Span Loading			**Full Span Loading**		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	5.53	6.34	5.56	5.91	6.94	6.01
G2	5.22	5.83	5.04	5.57	6.41	5.64
G3	4.90	5.36	4.72	5.23	5.88	5.28
G4	4.61	4.61	4.46	4.93	5.14	4.98
G5	4.26	4.38	4.14	4.54	4.89	4.57

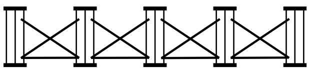
MEASURED BEARING

	Total Settlement		
Point	**End 1**	**End 2**	**Avg.**
G1	0.07	0.11	0.09
G2	0.09	0.12	0.11
G3	0.08	0.14	0.11
G4	0.07	0.11	0.09
G5	0.07	0.18	0.12

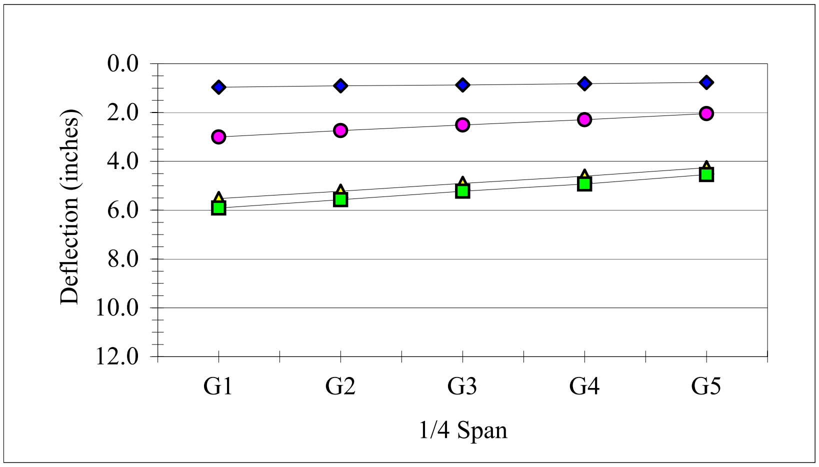
FIELD MEASUREMENT SUMMARY

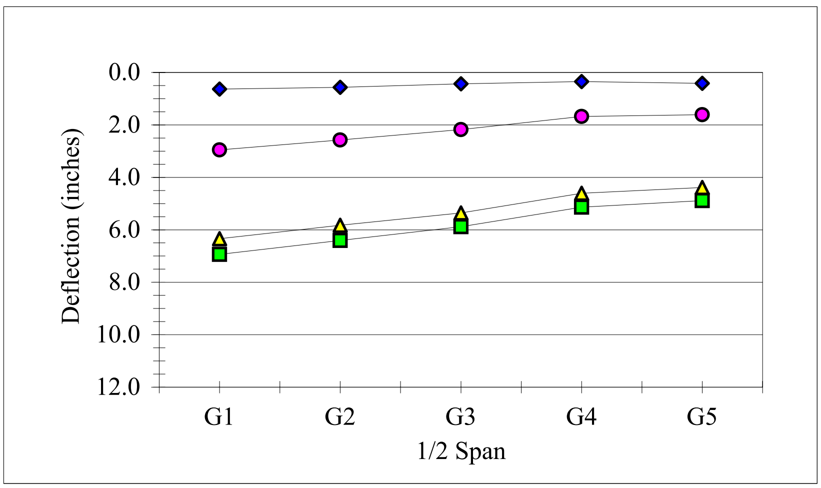
PROJECT NUMBER: U-2102 (Guess Road over Eno River, Stage 2)
MEASUREMENT DATE: February 28, 2003

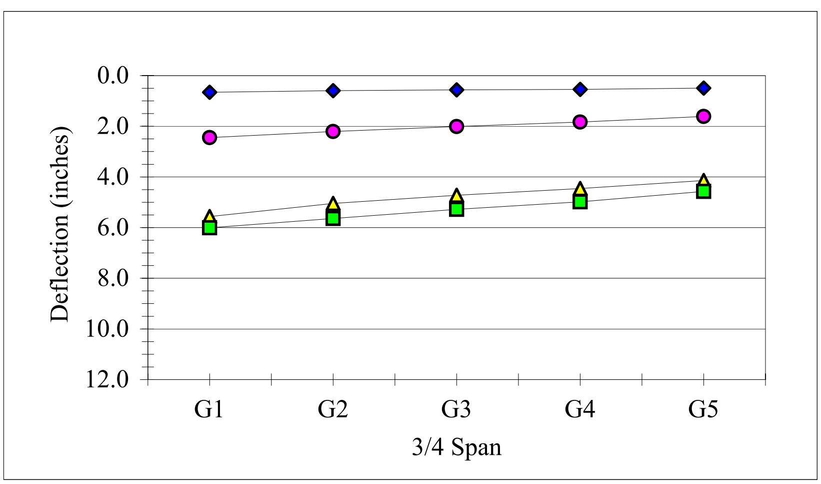
GIRDER DEFLECTIONS
CROSS SECTION VIEW



1/4 Span Loading
1/2 Span Loading
3/4 Span Loading
Full Span Loading



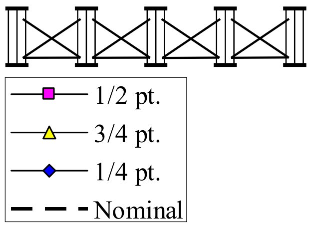


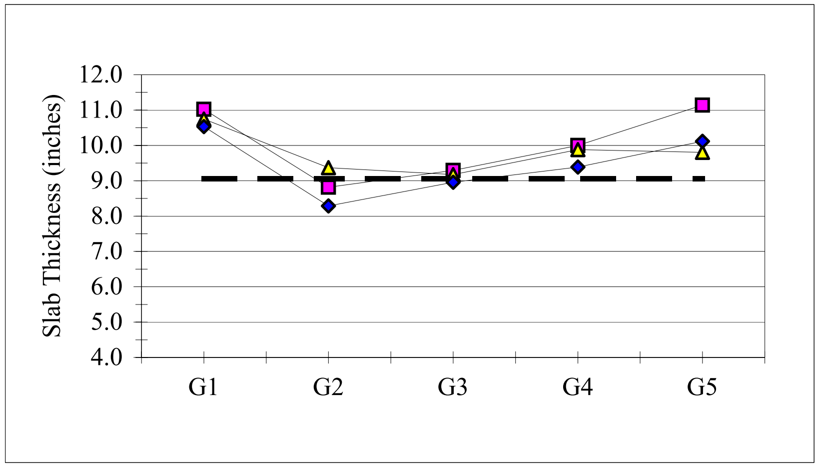


FIELD MEASUREMENT SUMMARY

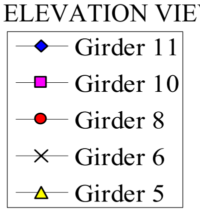
PROJECT NUMBER: U-2102 (Guess Road over Eno River, Stage 2)
MEASUREMENT DATE: February 28, 2003

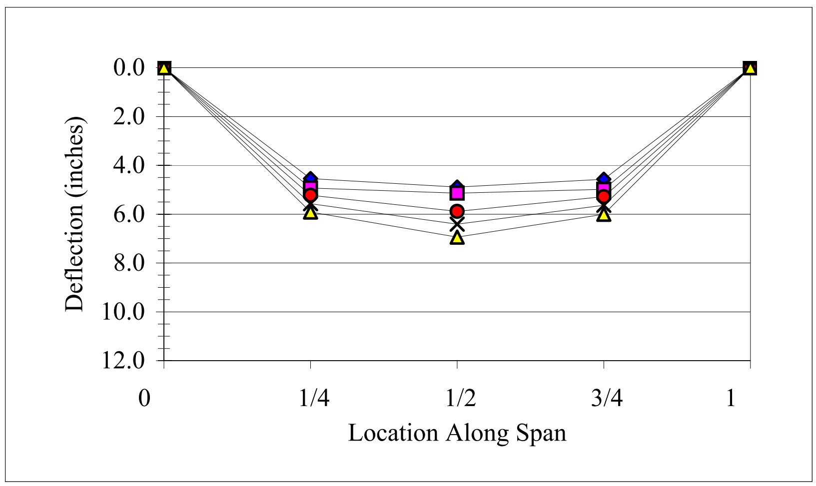
SLAB THICKNESS
CROSS SECTION VIEW



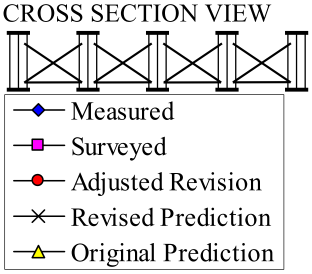


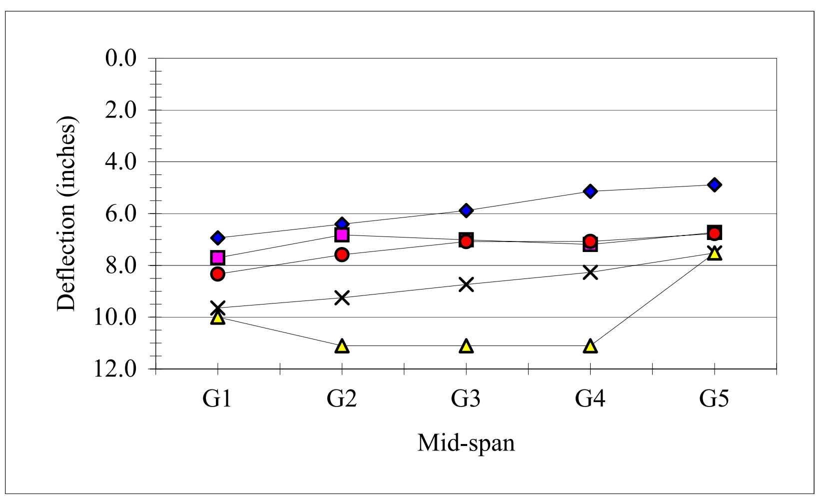
*GIRDER DEFLECTIONS
ELEVATION VIEW





GIRDER DEFLECTIONS
CROSS SECTION VIEW

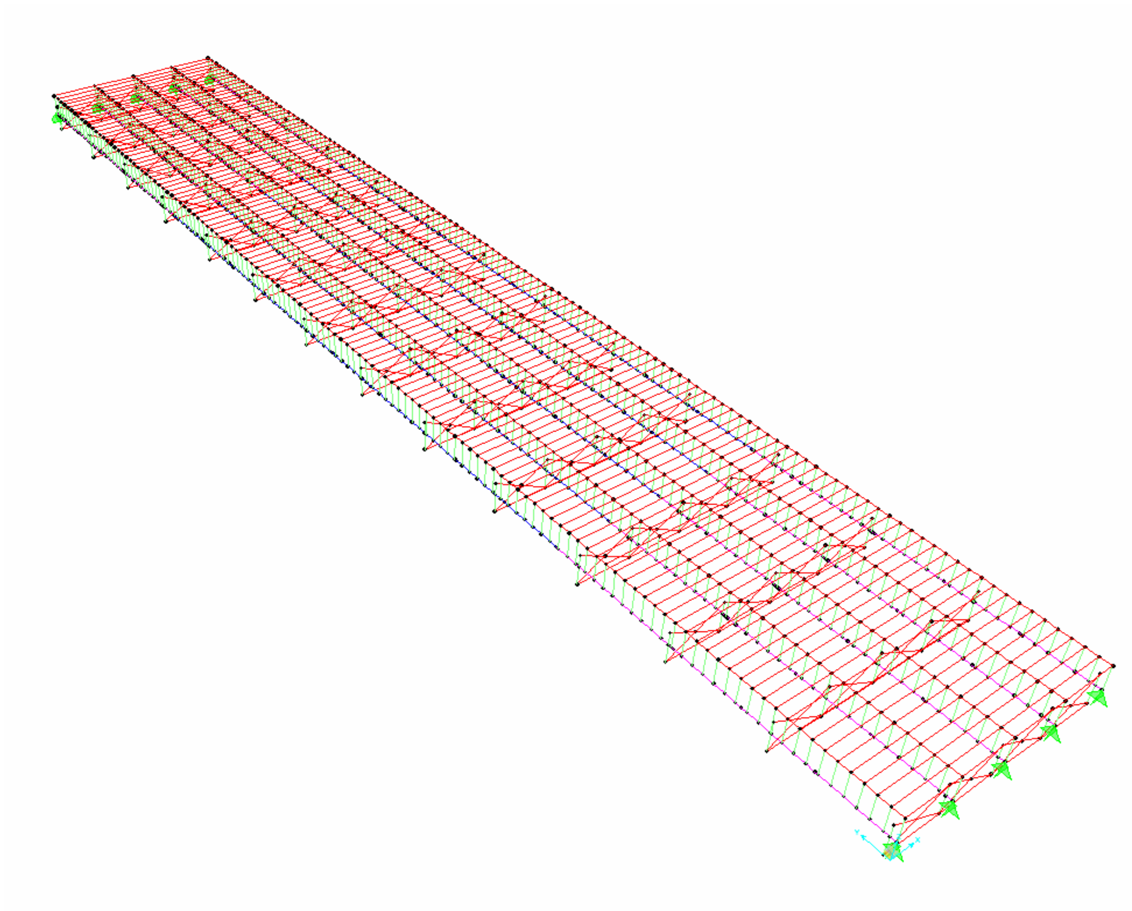




SAP 2000 MODELING SUMMARY

PROJECT NUMBER: U-2102 (Guess Road over Eno River, Stage 2)

MODEL PICTURE (Steel only, isometric view)



SAP2000 MODELING SUMMARY

PROJECT NUMBER: U-2102 (Guess Road over Eno River, Stage 2)

MODEL DESCRIPTION

COMPONENT	**Element Type**
Girder:	Frame Element
Cross Frame Members:	Frame Element
Stay-in-Place Deck Forms:	Area Element* (Shell Element) Frame Element
Rigid Link:	Frame Element

* See Shell Properties in Appendix F

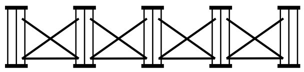
GIRDER DEFLECTIONS

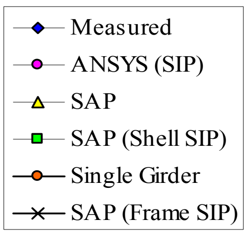
SAP				**Single Girder Line**		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	6.94	9.64	6.94	6.49	9.00	6.49
G2	6.74	9.35	6.74	6.86	9.52	6.86
G3	6.51	9.03	6.51	6.77	9.40	6.77
G4	6.24	8.66	6.24	7.15	9.93	7.15
G5	5.94	8.26	5.94	5.09	7.07	5.09
SAP (Shell SIP)				**ANSYS (SIP)**		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	6.97	9.66	6.96	6.42	8.98	6.43
G2	6.71	9.30	6.72	6.20	8.67	6.21
G3	6.44	8.92	6.46	5.99	8.37	5.99
G4	6.14	8.53	6.18	5.78	8.06	5.77
G5	5.83	8.11	5.87	5.56	7.76	5.55
SAP (Frame SIP)						
Point	**1/4**	**1/2**	**3/4**			
G1	6.96	9.66	6.95			
G2	6.70	9.30	6.72			
G3	6.43	8.92	6.48			
G4	6.14	8.53	6.24			
G5	5.84	8.11	5.99			

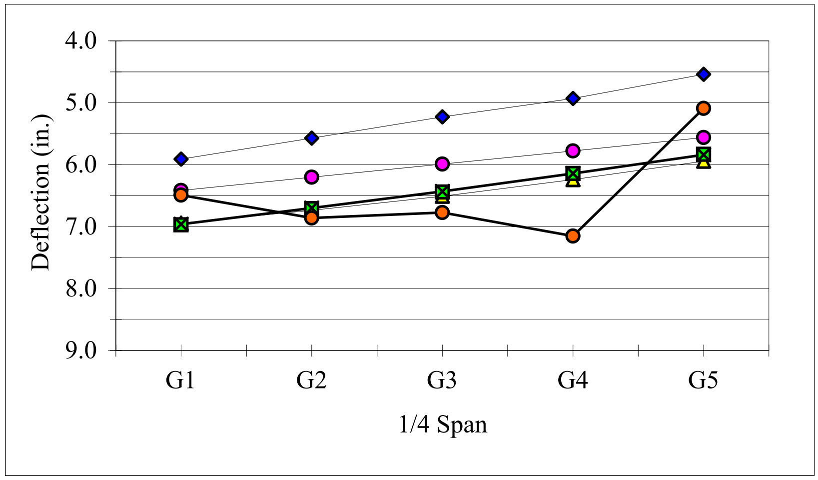
SAP 2000 MODELING SUMMARY

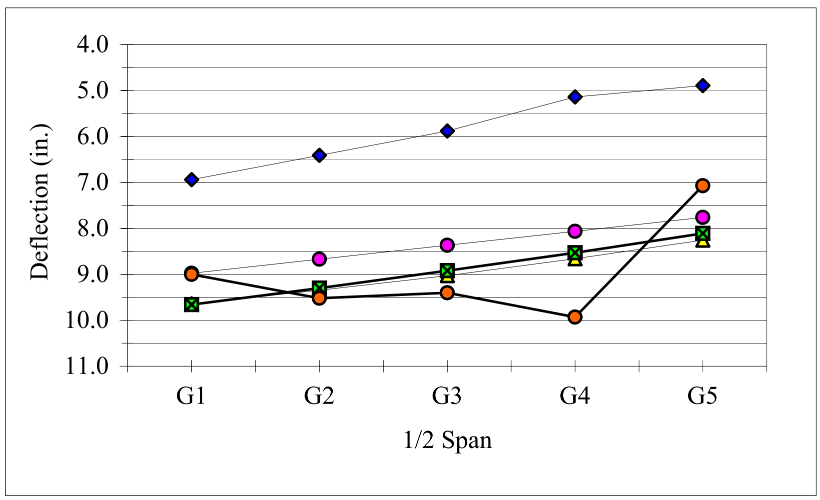
PROJECT NUMBER: U-2102 (Guess Road over Eno River, Stage 2)

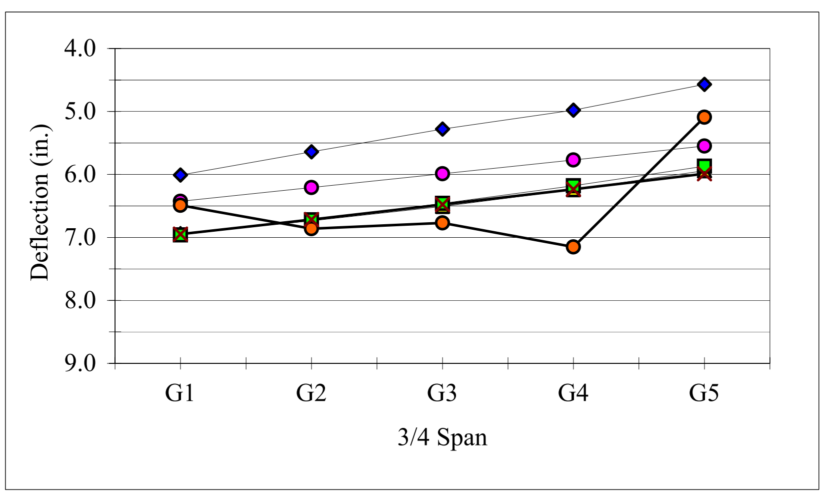
*GIRDER DEFLECTIONS
CROSS SECTION VIEW











Appendix B

Deflection Summary for the US 29

This appendix contains a detailed description of the US 29 from previous research by Whisenhunt (2004). It includes bridge geometry, material data, cross-frame type and size, and dead loads calculated from slab geometry. Tables and graphs of the field measured non-composite girder deflections are included.

A summary of the SAP simplified model created for the Eno River bridge is also included in this appendix. This summary includes a picture of the SAP model, details about the elements used in the model generation, and tables and graphs of the deflections predicted by the model.

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: R-0984B (US 29 over NC 150, southbound)
MEASUREMENT DATE: May 6, 2004

BRIDGE DESCRIPTION

TYPE	One Span Simple	
LENGTH	123.83 ft (34.74 m)	
NUMBER OF GIRDERS	7	
GIRDER SPACING	7.75 ft (2.36 m)	
SKEW	46 deg	
OVERHANG	2.29 ft (symmetric)	(from web centerline)
BEARING TYPE	Elastomeric Pad	

MATERIAL DATA

STRUCTURAL STEEL	**Grade**	**Yield Strength**
Girder:	AASHTO M270	50 ksi (345 MPa)
Other:	AASHTO M270	50 ksi (345 MPa)

CONCRETE UNIT WEIGHT	150 pcf (nominal)
SIP FORM WEIGHT	3 psf (nominal)

GIRDER DATA

LENGTH	123.83 ft (34.74 m)
TOP FLANGE WIDTH	15 in (381 mm)
BOTTOM FLANGE WIDTH	15 in (381 mm)
WEB THICKNESS	0.5 in (13 mm)
WEB DEPTH	52 in (1321 mm)

FLANGES	**Thickness**	**Begin**	**End**
Top:	0.75 in (19.05 mm)	0.00	61.92 ft (18.872 m)
Bottom:	0.75 in (19.05 mm)	0.00	26.92 ft (8.20 m)
	1.38 in (34.93 mm)	26.92 ft (8.20 m)	61.92 ft (18.87 m)

STIFFENERS	
Longitudinal:	NONE
Bearing:	PL 0.75" × 7.25" (19 mm × 184 mm)
Intermediate:	PL 0.375" × NA (10 mm × NA, connector plate)
End Bent Connector:	PL 0.375" × NA (10 mm × NA, connector plate)

CROSS-FRAME DATA

	Type	**Diagonals**	**Horizontals**
END	K	WT 4×9	C 15×33.9 (top)
			WT 4×9 (bottom)
INTERMEDIATE	K	L 3½×3½×5/16	L 3½×3½×5/16 (bottom)

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: R-0984B (US 29 over NC 150, southbound)
MEASUREMENT DATE: May 6, 2004

DECK LOADS

	Concrete*		Deck Slab**		Ratio
Girder	**lb/ft**	**N/mm**	**lb/ft**	**N/mm**	
G1	746.40	10.89	794.64	11.60	0.94
G2	866.40	12.64	926.04	13.51	0.94
G4	866.40	12.64	926.04	13.51	0.94
G6	866.40	12.64	926.04	13.51	0.94
G7	746.40	10.89	794.64	11.60	0.94

*calculated with measured slab thicknesses
**includes slab, buildups, and stay-in-place forms (nominal)

SLAB DATA

THICKNESS	8.25 in	(nominal)
BUILD-UP	2.50 in	(nominal)
REBAR	**Size**	**Spacing**
LONGITUDINAL	(US)	(nominal)
Top:	#4	18 in
Bottom:	#5	10 in
TRANSVERSE		
Top:	#5	7 in
Bottom:	#5	7 in

GIRDER DEFLECTIONS (data in inches, full span concrete deflections less bearing settlement)

MEASURED

	1/4 Span Loading			1/2 Span Loading		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	0.07	-0.10	0.05	0.95	1.87	1.62
G2	0.02	-0.04	0.11	0.83	1.69	1.40
G4	0.43	0.21	0.29	0.93	1.62	1.37
G6	0.31	0.64	0.64	0.91	1.85	1.62
G7	0.57	0.93	0.79	1.21	2.08	1.85
	3/4 Span Loading			**Full Span Loading**		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	2.73	4.03	3.02	9.20	4.44	3.29
G2	2.47	3.61	2.66	7.55	4.13	2.97
G4	2.47	3.36	2.50	6.90	3.98	2.91
G6	2.67	3.70	2.83	6.70	4.31	3.18
G7	3.12	4.12	3.13	6.70	4.65	3.39

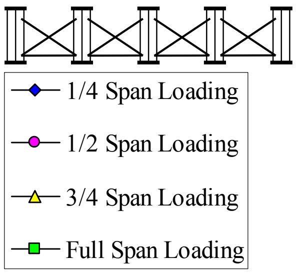
MEASURED BEARING

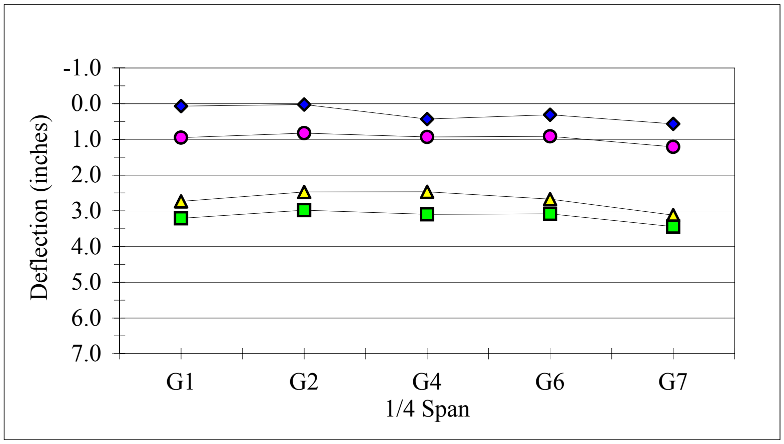
	Total Settlement		
Point	**End 1**	**End 2**	**Avg.**
G1	0.13	0.07	0.10
G2	0.13	0.04	0.09
G4	0.11	0.16	0.14
G6	0.14	0.13	0.13
G7	0.11	0.08	0.09

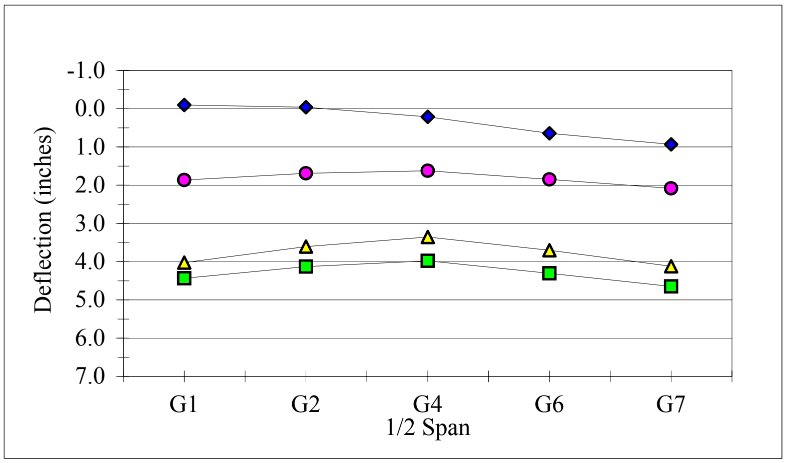
FIELD MEASUREMENT SUMMARY

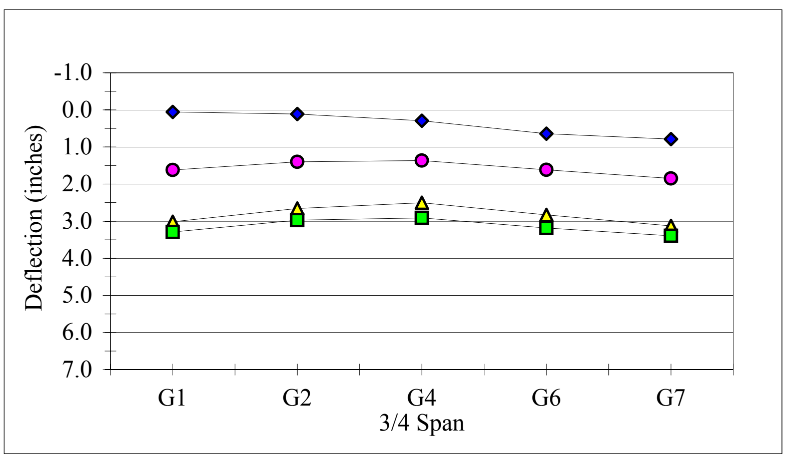
PROJECT NUMBER: R-0984B (US 29 over NC 150, southbound)
MEASUREMENT DATE: May 6, 2004

GIRDER DEFLECTIONS
CROSS SECTION VIEW





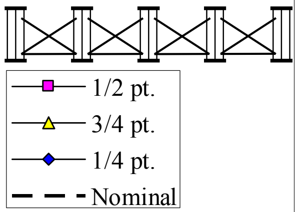


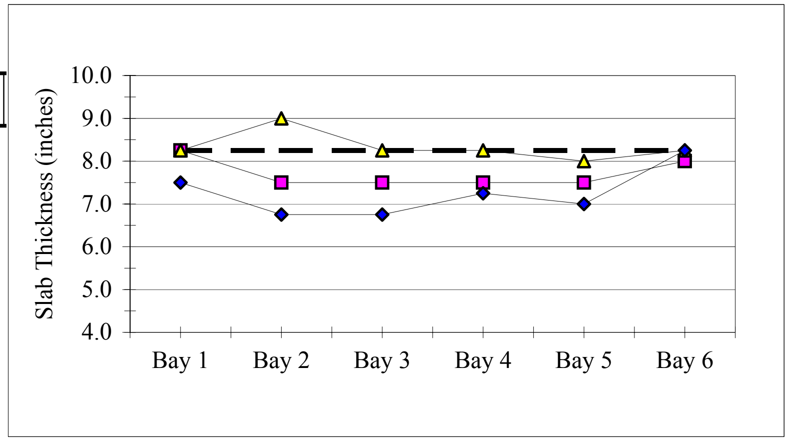


FIELD MEASUREMENT SUMMARY

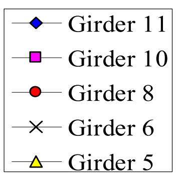
PROJECT NUMBER: R-0984B (US 29 over NC 150, southbound)
MEASUREMENT DATE: May 6, 2004

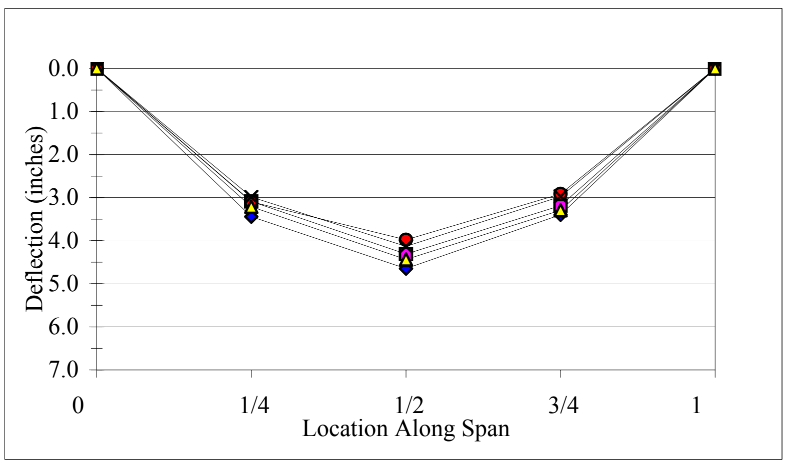
SLAB THICKNESS
CROSS SECTION VIEW



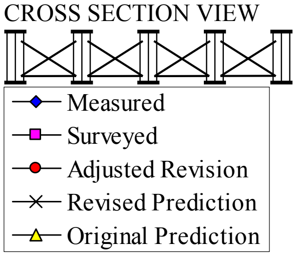


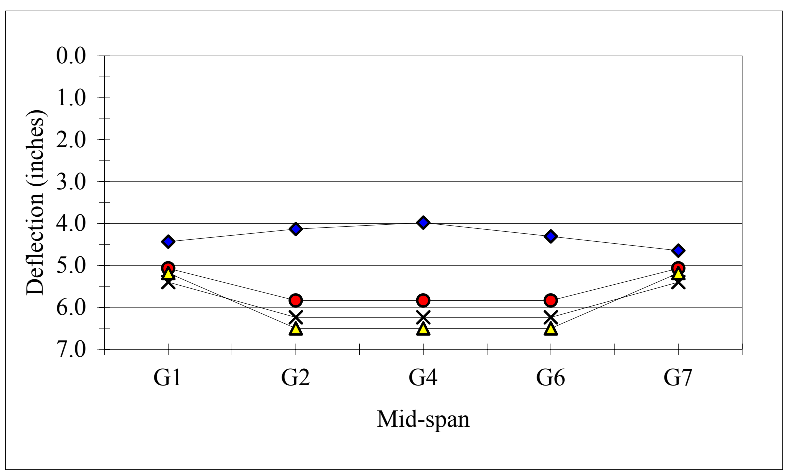
GIRDER DEFLECTIONS
ELEVATION VIEW





GIRDER DEFLECTIONS
CROSS SECTION VIEW

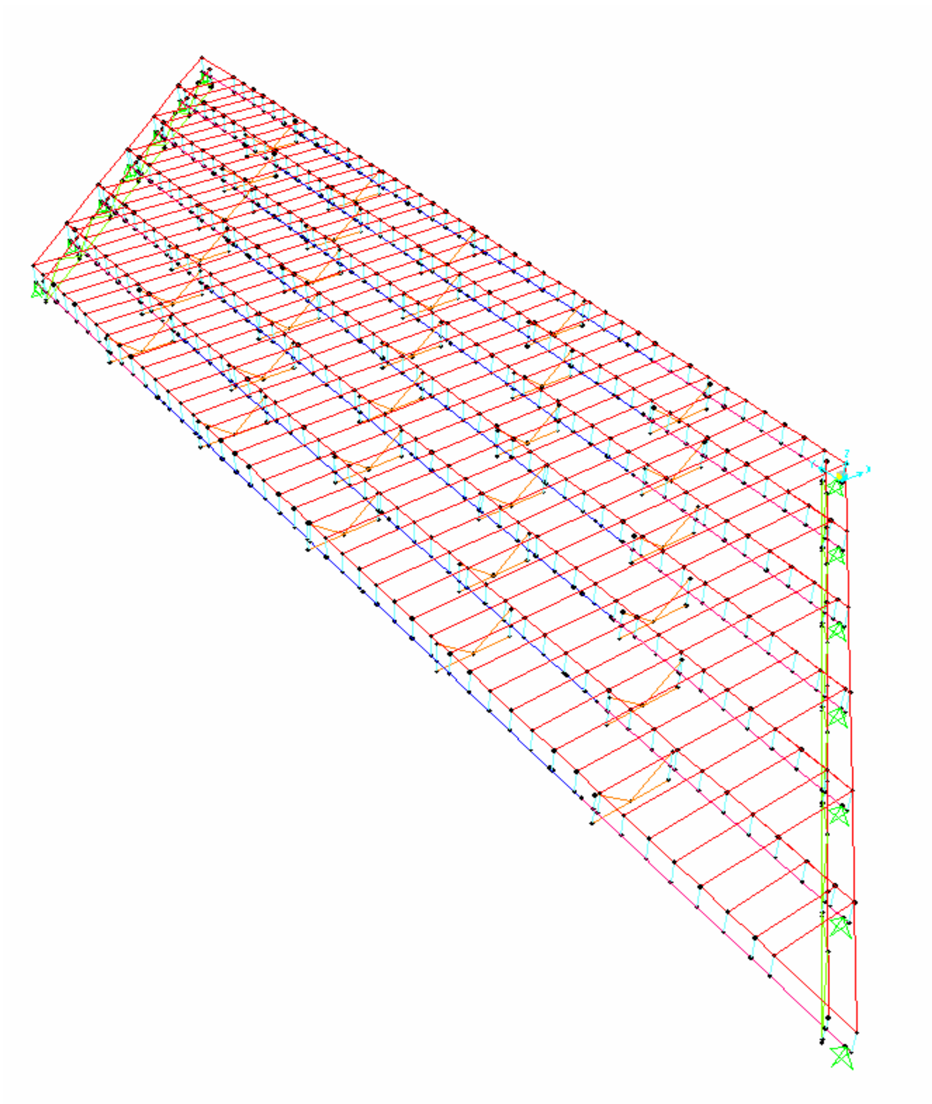




SAP 2000 MODELING SUMMARY

PROJECT NUMBER: R-0984B (US 29 over NC 150, southbound)

MODEL PICTURE (Steel only, oblique view)



SAP 2000 MODELING SUMMARY

PROJECT NUMBER: R-0984B (US 29 over NC 150, southbound)

MODEL DESCRIPTION

COMPONENT	**Element Type**
Girder:	Frame Element
Cross Frame Members:	Frame Element
Stay-in-place Deck Forms:	Area Element* (Shell Element) Frame Element
Rigid Link:	Frame Element

* See Shell Properties in Appendix F

GIRDER DEFLECTIONS

SAP				Single Girder Line		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	3.61	5.02	3.61	3.63	5.03	3.63
G2	4.08	5.68	4.08	4.22	5.84	4.22
G4	4.16	5.78	4.16	4.22	5.84	4.22
G6	4.08	5.68	4.08	4.22	5.84	4.22
G7	3.61	5.02	3.61	3.63	5.03	3.63

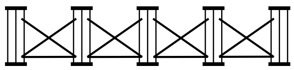
SAP (SIP)				ANSYS (SIP)		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	3.86	5.38	3.87	3.10	4.41	3.16
G2	3.98	5.54	3.99	3.06	4.33	3.09
G4	4.04	5.64	4.06	3.04	4.30	3.07
G6	3.98	5.50	3.95	3.07	4.32	3.07
G7	3.86	5.35	3.84	3.14	4.40	3.13

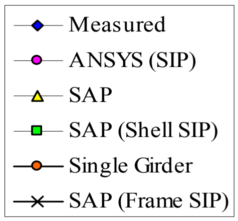
SAP (Frame SIP)			
Point	**1/4**	**1/2**	**3/4**
G1	3.71	5.16	3.61
G2	4.07	5.65	4.08
G3	4.17	5.78	4.16
G4	4.09	5.68	4.08
G5	3.71	5.13	3.68

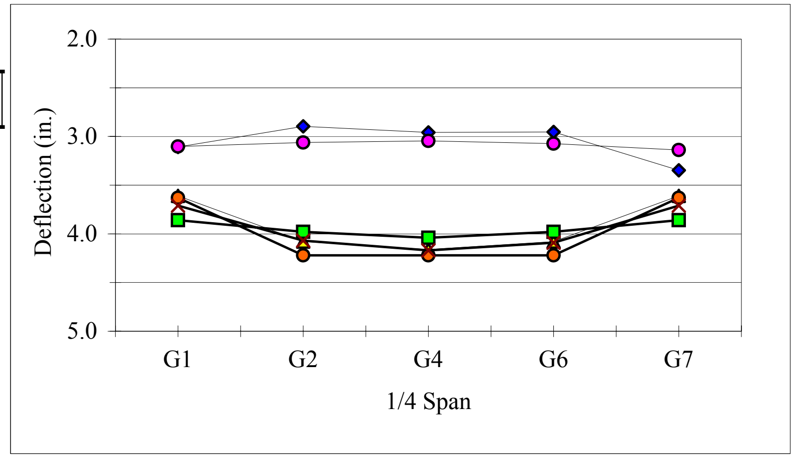
SAP 2000 MODELING SUMMARY

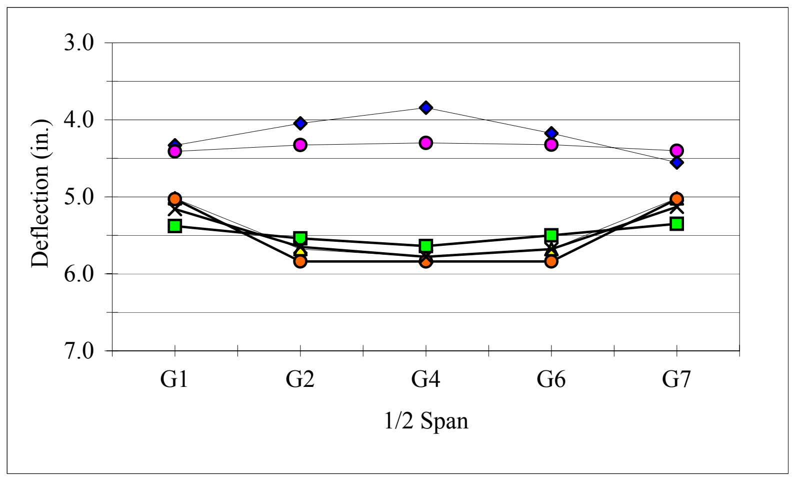
PROJECT NUMBER: R-0984B (US 29 over NC 150, southbound)

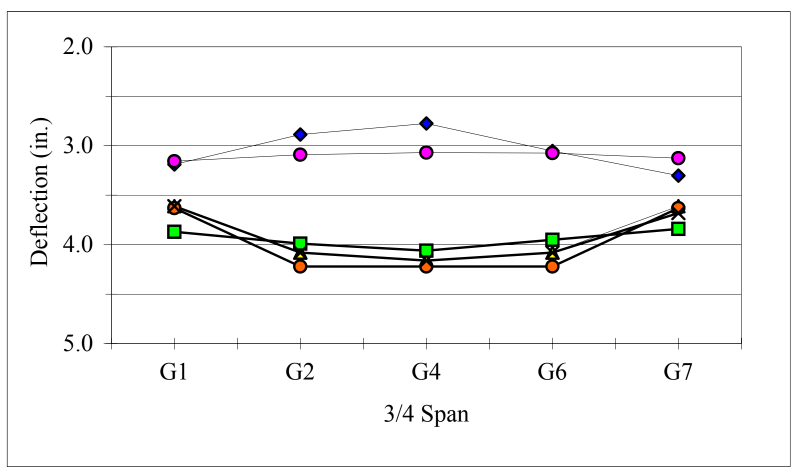
GIRDER DEFLECTIONS
CROSS SECTION VIEW











Appendix C

Deflection Summary for the Bridge 8

This appendix contains a detailed description of the Bridge 8 from recent research by Fisher (2005). It includes bridge geometry, material data, cross-frame type and size, and dead loads calculated from slab geometry. Tables and graphs of the field measured non-composite girder deflections are included.

A summary of the SAP simplified model created for the Bridge 8 is also included in this appendix. This summary includes a picture of the SAP model, details about the elements used in the model generation, and tables and graphs of the deflections predicted by the model.

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: R-2547 (EB Bridge on US 64 Bypass over Smithfield Rd.)
MEASUREMENT DATE: August 24, 2004

BRIDGE DESCRIPTION

TYPE	One Span Simple	
LENGTH	153.04 ft (46.648 m)	
NUMBER OF GIRDERS	6	
GIRDER SPACING	11.29 ft (3.440 m)	
SKEW	60 deg	
OVERHANG	2.85 ft (870 mm)	(from web centerline)
BEARING TYPE	Elastomeric Pad	

MATERIAL DATA

STRUCTURAL STEEL	**Grade**	**Yield Strength**
Girder:	AASHTO M270	50 ksi (345 MPa)
Other:	AASHTO M270	50 ksi (345 MPa)

CONCRETE UNIT WEIGHT	150 pcf (nominal)
SIP FORM WEIGHT	4.69 psf (CSI Catalog)

GIRDER DATA

LENGTH	153.04 ft (46.648 m)
TOP FLANGE WIDTH	17.99 in (457 mm)
BOTTOM FLANGE WIDTH	17.99 in (457 mm)
WEB THICKNESS	0.63 in (16 mm)
WEB DEPTH	68.03 in (1728 mm)

FLANGES	**Thickness**	**Begin**	**End**
Top:	2.00 in (51 mm)	0.00	153.04 ft (46.648 m)
Bottom:	2.00 in (51 mm)	0.00	40.98 ft (12.490 m)
	3.00 in (76 mm)	40.98 ft (12.490 m)	112.07 ft (34.158 m)
	2.00 in (51 mm)	112.07 ft (34.158 m)	153.04 ft (46.648 m)

CROSS-FRAME DATA

	Type	Diagonals	Horizontals
END BENT (Type K)	**WT 4 x 14**	**C 15 x 33.9 (top)** WT 4 x 14 (bottom)	**NA**
MIDDLE BENT	NA	NA	NA
INTERMEDIATE (Type X)	L 3 x 3 x 3/8"	L 3 x 3 x 3/8" (bottom	NA

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: R-2547 (EB Bridge on US 64 Bypass over Smithfield Rd.)
MEASUREMENT DATE: August 24, 2004

DECK LOADS

	Concrete*		Deck Slab**		Ratio
Girder	**lb/ft**	**N/mm**	**lb/ft**	**N/mm**	
G1	1186.8	17.32	1256.7	18.34	0.94
G2	1415.0	20.65	1517.8	22.15	0.93
G3	1415.0	20.65	1517.8	22.15	0.93
G4	1415.0	20.65	1517.8	22.15	0.93
G5	1415.0	20.65	1517.8	22.15	0.93
G6	1186.8	17.32	1256.7	18.34	0.94

*calculated with measured slab thicknesses
**includes slab, buildups, and stay-in-place forms (nominal)

SLAB DATA

THICKNESS	9.25 in	(nominal)
BUILD-UP	3.74 in	(nominal)
REBAR	**Size**	**Spacing**
LONGITUDINAL	(US)	(nominal)
Top:	#13	17.72 in
Bottom:	#16	8.27 in
TRANSVERSE		
Top:	#16	5.51 in
Bottom:	#16	5.51 in

GIRDER DEFLECTIONS (data in inches, full span concrete deflections less bearing settlement)

MEASURED

	1/4 Span Loading			1/2 Span Loading		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	0.50	0.45	0.28	1.22	1.44	0.98
G2	0.49	0.64	0.29	1.16	1.49	0.95
G3	0.59	0.68	0.26	1.21	1.43	0.85
G4	0.57	0.75	0.32	1.11	1.36	0.97
G5	0.60	0.82	0.32	1.07	1.35	0.87
G6	0.62	0.81	0.34	1.02	1.31	0.86
	3/4 Span Loading			**Full Span Loading**		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	2.15	2.75	2.24	2.29	2.89	2.38
G2	2.19	2.94	2.19	2.35	3.14	2.40
G3	2.25	2.94	2.09	2.43	3.17	2.34
G4	2.24	3.02	2.22	2.42	3.26	2.49
G5	2.26	3.05	2.13	2.43	3.30	2.42
G6	2.23	3.03	2.17	2.34	3.24	2.45

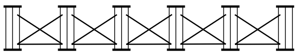
MEASURED BEARING

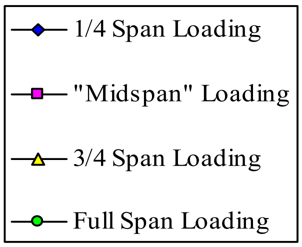
	Total Settlement		
Point	**End 1**	**End 2**	**Avg.**
G1	0.09	0.10	0.09
G2	0.13	0.11	0.12
G3	---	0.12	0.12
G4	0.04	0.04	0.04
G5	0.07	0.09	0.08
G6	0.08	0.10	0.09

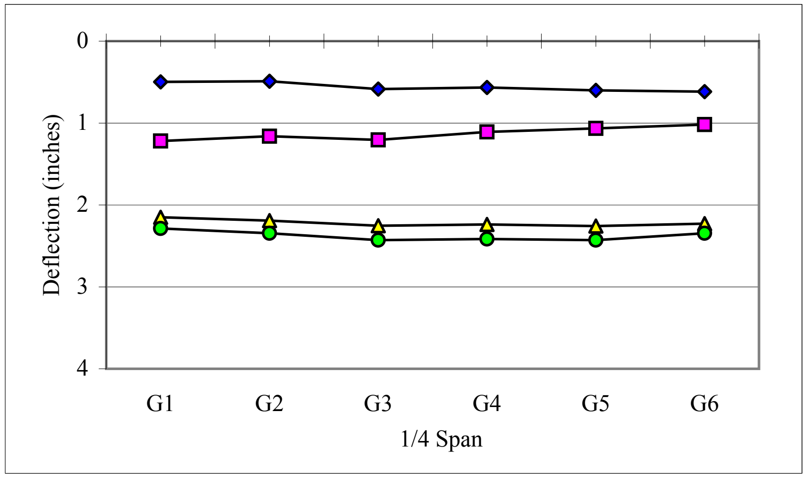
FIELD MEASUREMENT SUMMARY

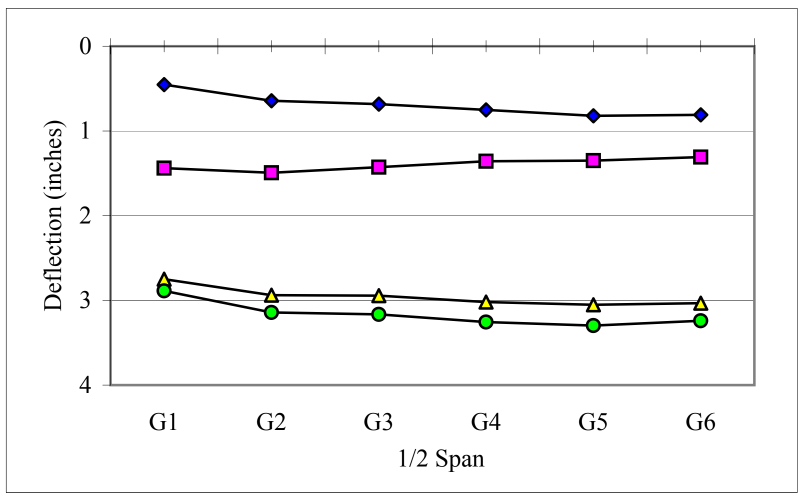
PROJECT NUMBER: R-2547 (EB Bridge on US 64 Bypass over Smithfield Rd.)
MEASUREMENT DATE: August 24, 2004

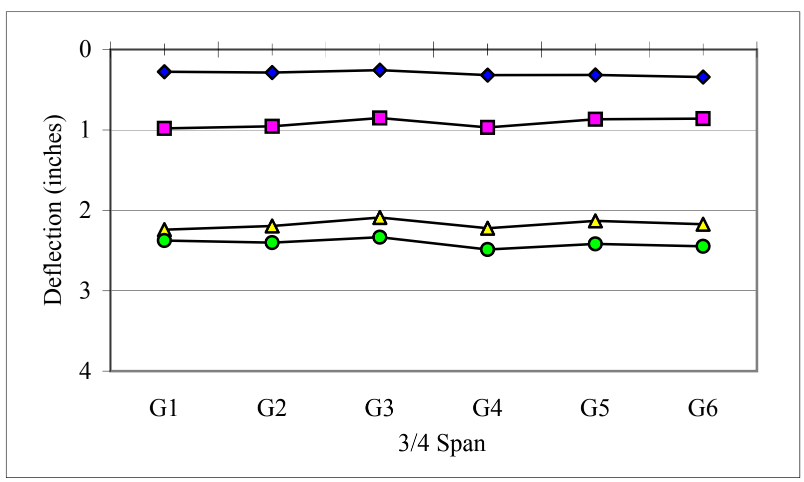
GIRDER DEFLECTIONS
CROSS SECTION VIEW







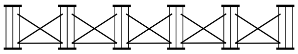


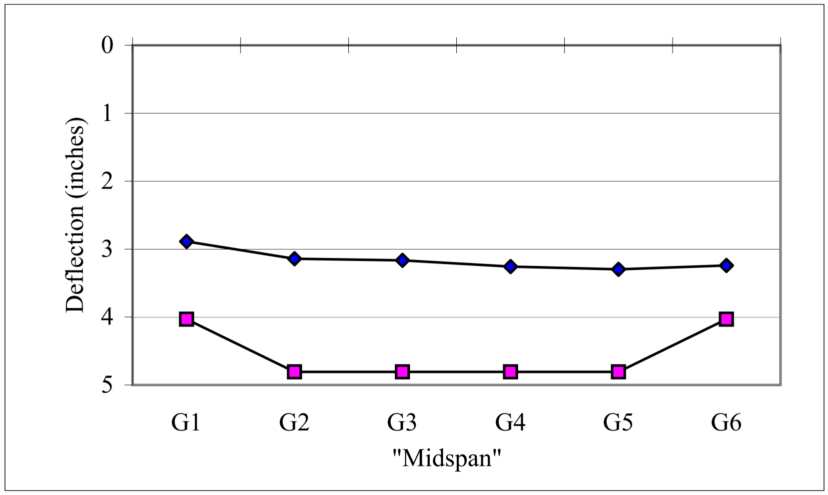


FIELD MEASUREMENT SUMMARY

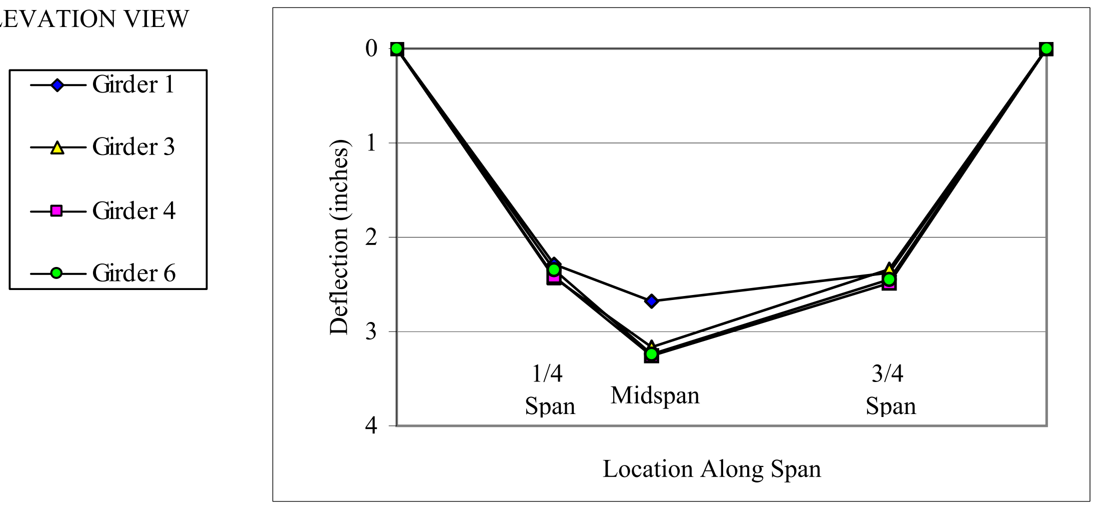
PROJECT NUMBER: R-2547 (EB Bridge on US 64 Bypass over Smithfield Rd.)
MEASUREMENT DATE: August 24, 2004

SLAB THICKNESS
CROSS SECTION VIEW





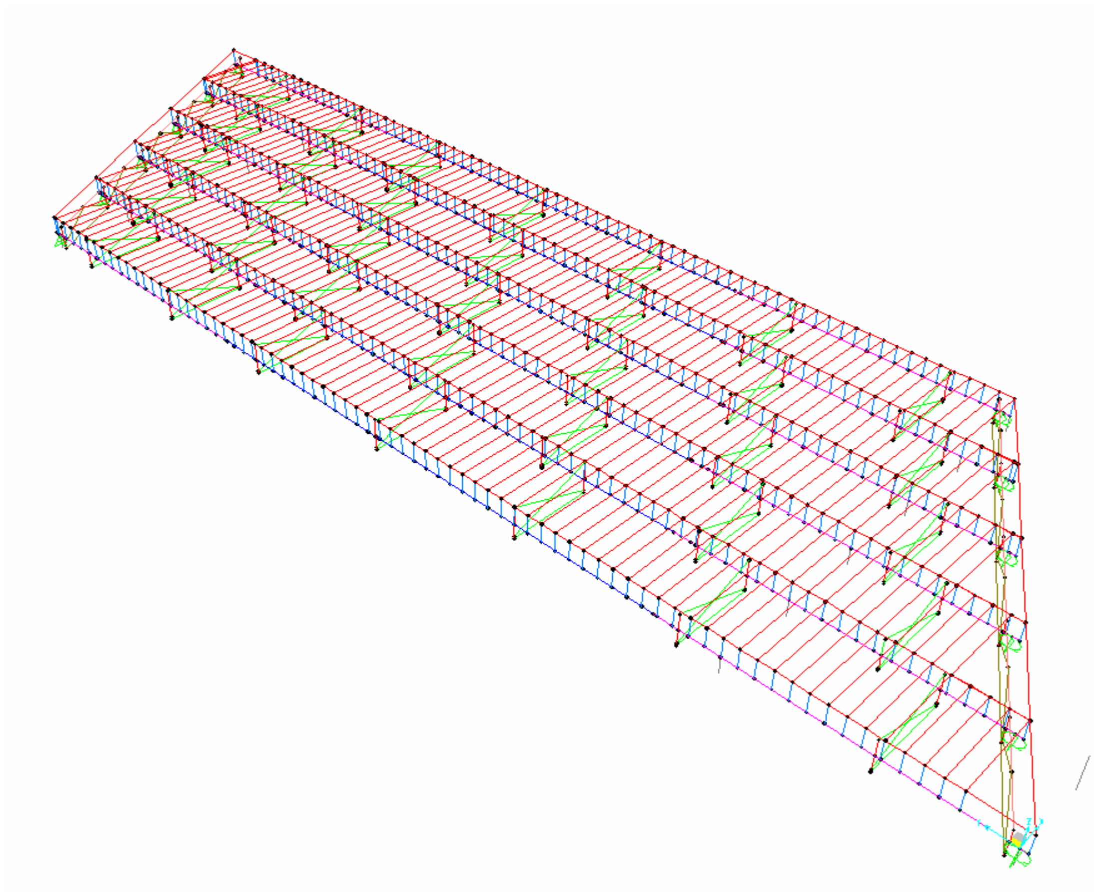
*GIRDER DEFLECTIONS
ELEVATION VIEW



SAP 2000 MODELING SUMMARY

PROJECT NUMBER: R-2547 (EB Bridge on US 64 Bypass over Smithfield Rd.)

MODEL PICTURE (Steel only, isometric view)



SAP2000 MODELING SUMMARY

PROJECT NUMBER: R-2547 (EB Bridge on US 64 Bypass over Smithfield Rd.)

MODEL DESCRIPTION

COMPONENT	**Element Type**
Girder:	Frame Element
Cross Frame Members:	Frame Element
Stay-in-place Deck Forms:	Area Element* (Shell Element)
Rigid Link:	Frame Element

* See Area Properties in Appendix F

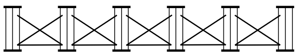
GIRDER DEFLECTIONS

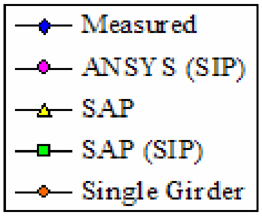
	SAP			Single Girder Model		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	3.02	4.18	3.02	2.95	4.08	2.95
G2	3.42	4.73	3.42	3.52	4.87	3.52
G3	3.54	4.90	3.55	3.52	4.87	3.52
G4	3.54	4.90	3.55	3.52	4.87	3.52
G5	3.42	4.73	3.42	3.52	4.87	3.52
G6	3.02	4.18	3.02	2.95	4.08	2.95
	SAP (SIP)			**ANSYS (SIP)**		
Point	**1/4**	**1/2**	**3/4**	**1/4**	**1/2**	**3/4**
G1	3.07	4.25	3.08	3.03	4.11	3.04
G2	3.12	4.31	3.11	3.06	4.14	3.07
G3	3.15	4.35	3.14	3.08	4.17	3.09
G4	3.14	4.35	3.14	3.09	4.17	3.08
G5	3.12	4.31	3.12	3.06	4.14	3.05
G6	3.07	4.26	3.07	3.04	4.11	3.02

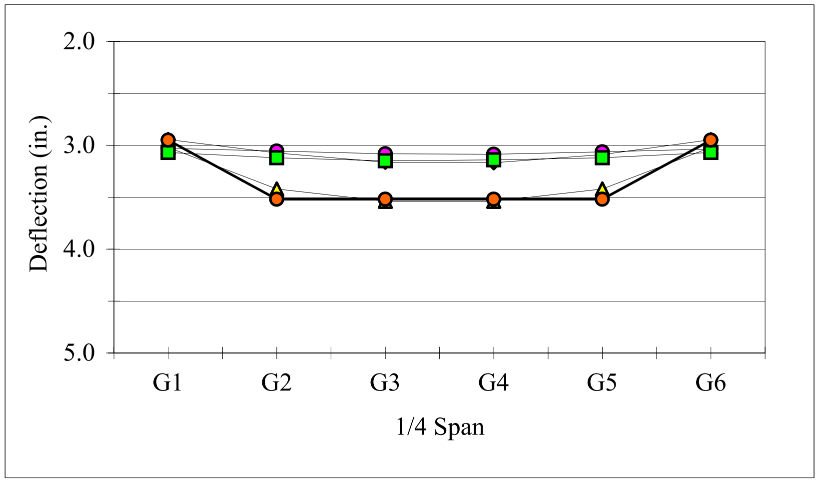
SAP 2000 MODELING SUMMARY

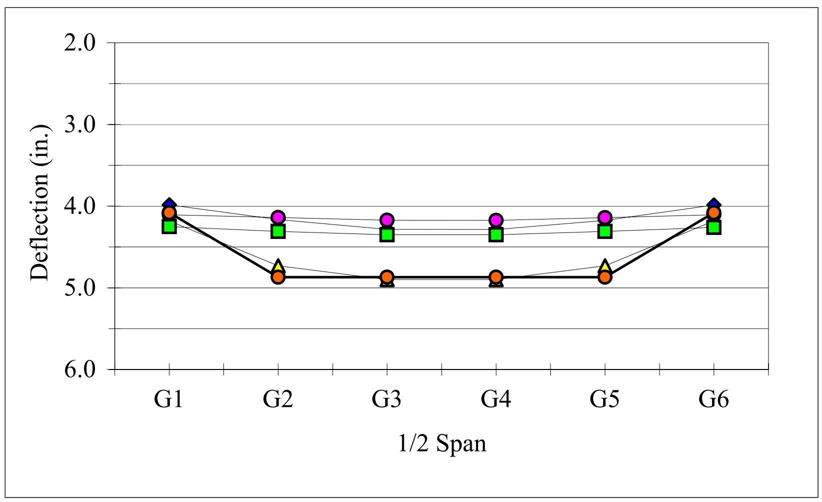
PROJECT NUMBER: R-2547 (EB Bridge on US 64 Bypass over Smithfield Rd.)

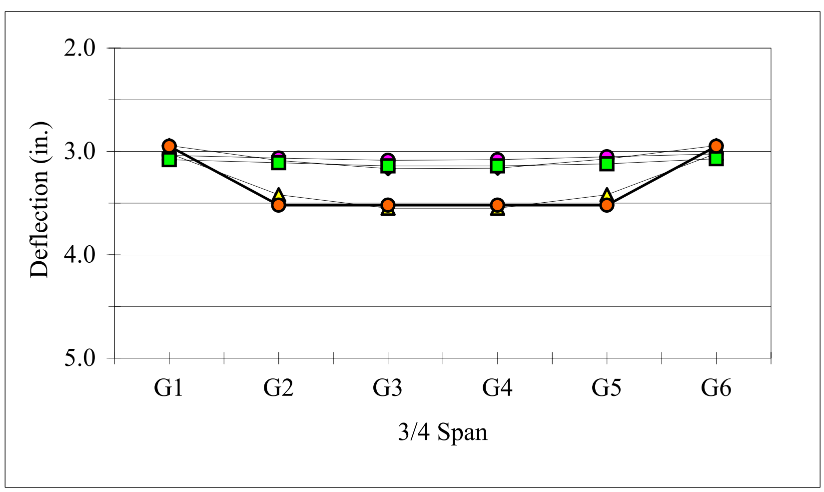
*GIRDER DEFLECTIONS
CROSS SECTION VIEW











Appendix D

Deflection Summary for the Wilmington Street Bridge

This appendix contains a detailed description of the Wilmington Street Bridge including bridge geometry, material data, cross-frame type and size, and dead loads calculated from slab geometry. Tables and graphs of the field measured non-composite girder deflections are included.

A summary of the SAP simplified model created for the Wilmington Street Bridge is also included in this appendix. This summary includes a picture of the SAP model, details about the elements used in the model generation, and tables and graphs of the deflections predicted by the model.

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: B-3257 (South Wilmington Street Bridge)
MEASUREMENT DATE: November 1, 2004

BRIDGE DESCRIPTION

TYPE	One Span Simple
LENGTH	149.50 ft (44.85 m)
NUMBER OF GIRDERS	5
GIRDER SPACING	8.25 ft (2.475 m)
SKEW	152 deg
OVERHANG	3.042 ft (Overhang Side)
	1 ft (ADJ to Stage I side)
BEARING TYPE	Pot Bearing

MATERIAL DATA

STRUCTURAL STEEL	**Grade**	**Yield Strength**
Girder:	AASHTO M270	50 ksi (345 MPa)
Other:	AASHTO M270	50 ksi (345 MPa)

CONCRETE UNIT WEIGHT	118 pcf (measured)
SIP FORM WEIGHT	3 psf (nominal)

GIRDER DATA

LENGTH	149.50 ft (44.85 m)
TOP FLANGE WIDTH	16 in. (406.4 mm)
BOTTOM FLANGE WIDTH	20 in. (508.0 mm)
WEB THICKNESS	0.5 in. (13 mm)
WEB DEPTH	54 in. (1371.6 mm)

FLANGES	**Thickness**	**Begin**	**End**
Top:	1 in. (25.4 mm)	0.00	31.25 ft (9.375 m)
	1.375 in. (34.93 mm)	31.25 ft (9.375 m)	118.25 ft (35.475 m)
	1 in. (25.4 mm)	118.25 ft (35.475 m)	149.5 ft (44.85 m)
Bottom:	1.125 ft (28.575 mm)	0.00	31.25 ft (9.375 m)
	1.875 in. (34.93 mm)	31.25 ft (9.375 m)	118.25 ft (35.475 m)
	1.125 ft (28.575 mm)	118.25 ft (35.475 m)	149.5 ft (44.85 m)

STIFFENERS	
Longitudinal:	NONE
Bearing:	PL 1" × 7" (25.4 mm × 177.8 mm)
Intermediate:	PL 0.5 " x NA (12.7 mm x NA connector Plate)
End Bent Connector:	PL 0.5 " x NA (12.7 mm x NA connector Plate)

CROSS-FRAME DATA

	Type	**Diagonals**	**Horizontals**
END	K	WT 4×12	C 15×50 (top)
			WT 4×12 (bottom)
INTERMEDIATE	K	L 3×3×5/16	L 3×3×5/16 (bottom)

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: B-3257 (South Wilmington Street Bridge)
MEASUREMENT DATE: November 1, 2004

DECK LOADS

	Concrete*		Deck Slab**		Ratio
Girder	**lb/ft**	**N/mm**	**lb/ft**	**N/mm**	
G6	518.88	7.57	549.50	8.02	0.94
G7	800.12	11.68	861.49	12.57	0.93
G8	769.70	11.23	831.07	12.13	0.93
G9	800.12	11.68	861.49	12.57	0.93
G10	743.48	10.85	774.10	11.30	0.96

*calculated with measured slab thicknesses
**includes slab, buildups, and stay-in-place forms (nominal)

SLAB DATA

THICKNESS	8.50 in.	(nominal)
BUILD-UP	2.50 in.	(nominal)
REBAR	**Size**	**Spacing**
LONGITUDINAL	(US)	(nominal)
Top:	#4	18 in.
Bottom:	#5	10 in.
TRANSVERSE		
Top:	#5	7 in.
Bottom:	#5	7 in.

GIRDER DEFLECTIONS (data in inches, full span concrete deflections less bearing settlement)

MEASURED

	1/4 Span Loading			1/2 Span Loading		
Point	**1/4**	**6/10**	**3/4**	**1/4**	**6/10**	**3/4**
G6	1.52	2.01	1.30	2.28	3.01	2.13
G7	1.42	1.75	1.04	2.20	2.81	1.85
G8	1.36	1.65	0.94	2.15	2.80	1.79
G9	1.38	1.67	1.00	2.20	2.99	1.93
G10	1.59	1.93	1.04	2.60	3.46	2.15
	3/4 Span Loading			**Full Span Loading**		
Point	**1/4**	**6/10**	**3/4**	**1/4**	**6/10**	**3/4**
G6	2.68	3.88	2.93	2.64	3.80	2.83
G7	2.70	3.76	2.72	2.64	3.70	2.72
G8	2.72	3.84	2.80	2.72	3.78	2.80
G9	2.97	4.25	3.17	2.97	4.19	3.17
G10	3.66	5.10	3.64	3.62	5.04	3.70

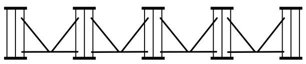
MEASURED BEARING

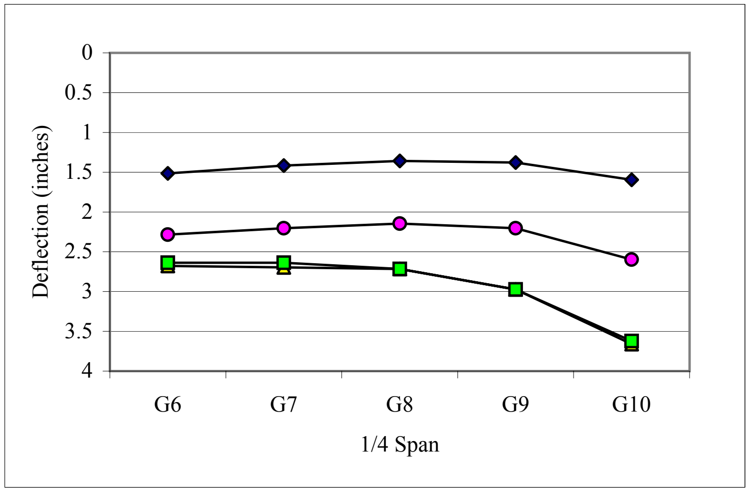
	Total Settlement		
Point	**End 1**	**End 2**	**Avg.**
G6	-	-	-
G7	-	-	-
G8	-	-	-
G9	-	-	-
G10	-	-	-

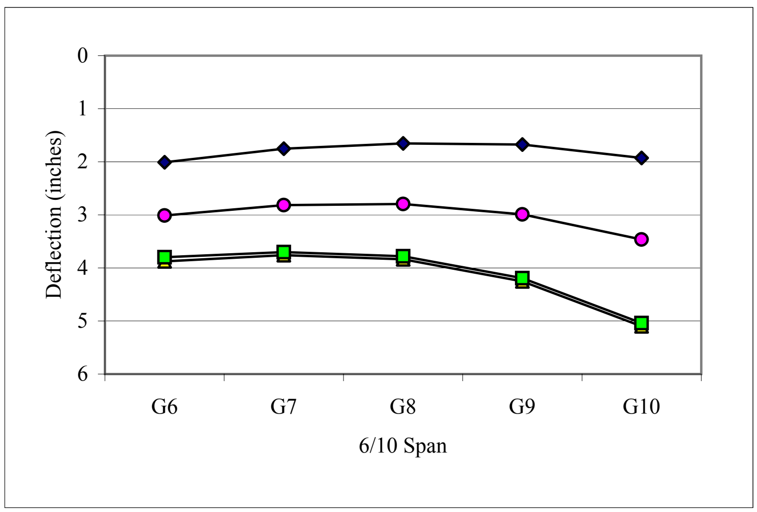
FIELD MEASUREMENT SUMMARY

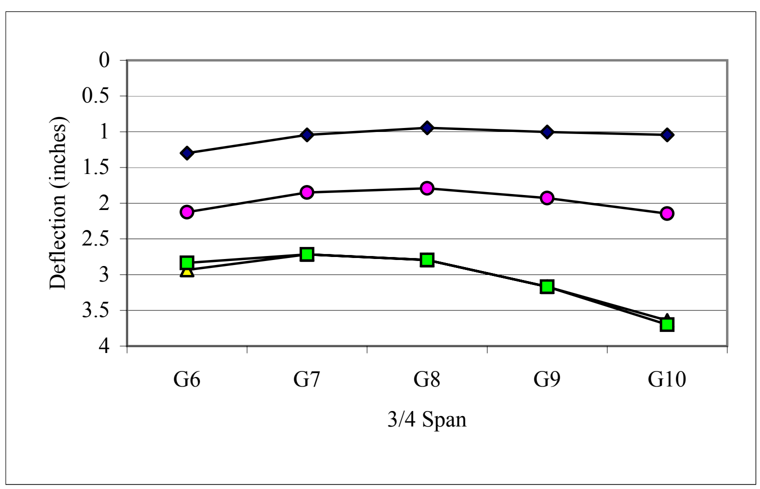
PROJECT NUMBER: B-3257 (South Wilmington Street Bridge)
MEASUREMENT DATE: November 1, 2004

GIRDER DEFLECTIONS
CROSS SECTION VIEW





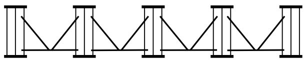


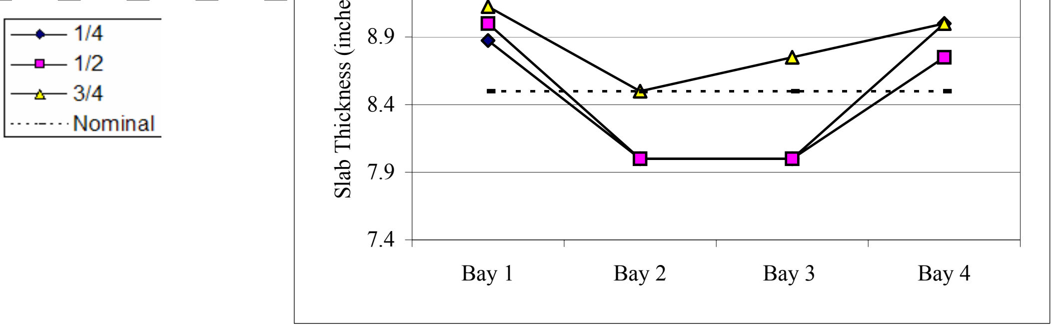


FIELD MEASUREMENT SUMMARY

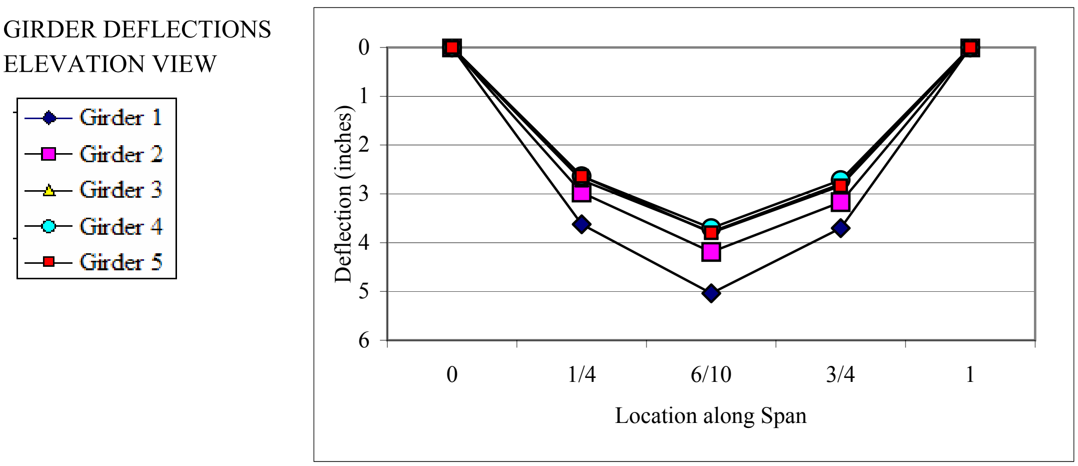
PROJECT NUMBER: B-3257 (South Wilmington Street Bridge)
MEASUREMENT DATE: November 1, 2004

SLAB THICKNESS
CROSS SECTION VIEW





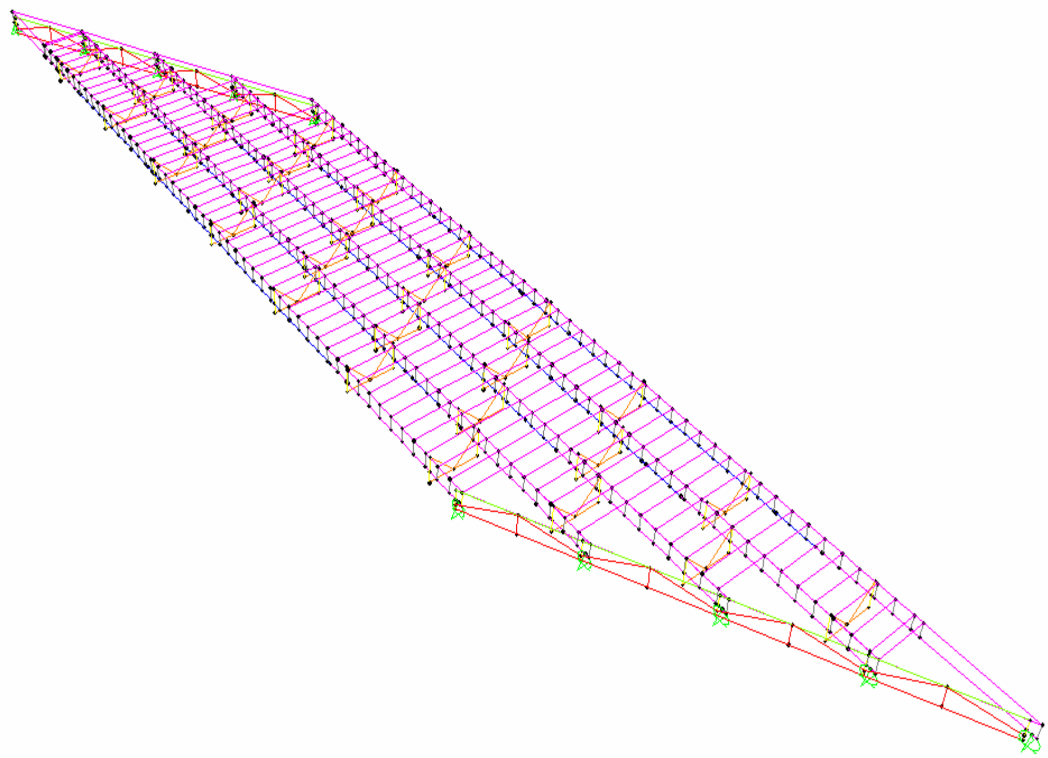
GIRDER DEFLECTIONS
ELEVATION VIEW



SAP 2000 MODELING SUMMARY

PROJECT NUMBER: B-3257 (South Wilmington Street Bridge)

MODEL PICTURE (Steel only, isometric view)



SAP 2000 MODELING SUMMARY

PROJECT NUMBER: B-3257 (South Wilmington Street Bridge)

MODEL DESCRIPTION

COMPONENT	**Element Type**
Girder:	Frame Element
Cross Frame Members:	Frame Element
Stay-in-place Deck Forms:	Area Element* (Shell Element)
Rigid Link:	Frame Element

* See Area Properties in Appendix F

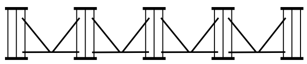
GIRDER DEFLECTIONS

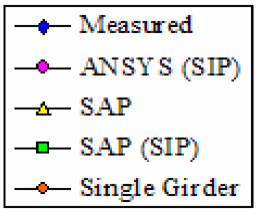
SAP				**Single Girder Model**		
Point	**1/4**	**6/10**	**3/4**	**1/4**	**6/10**	**3/4**
G6	3.06	4.04	3.07	2.94	3.87	2.94
G7	4.27	5.64	4.28	4.53	5.97	4.53
G8	4.50	5.91	4.48	4.36	5.74	4.36
G9	4.42	5.83	4.43	4.53	5.97	4.53
G10	4.25	5.60	4.24	4.21	5.54	4.21
SAP (SIP)				**ANSYS (SIP)**		
Point	**1/4**	**6/10**	**3/4**	**1/4**	**6/10**	**3/4**
G6	3.48	4.67	3.55	2.44	3.20	2.32
G7	3.56	4.73	3.59	2.24	3.02	2.22
G8	3.69	4.89	3.71	2.27	3.03	2.22
G9	3.92	5.20	3.95	2.45	3.28	2.40
G10	4.44	5.76	4.33	2.68	3.86	2.92

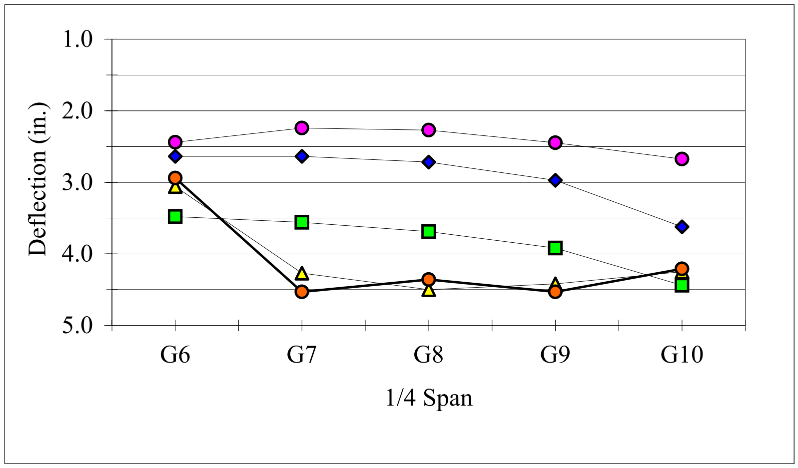
SAP 2000 MODELING SUMMARY

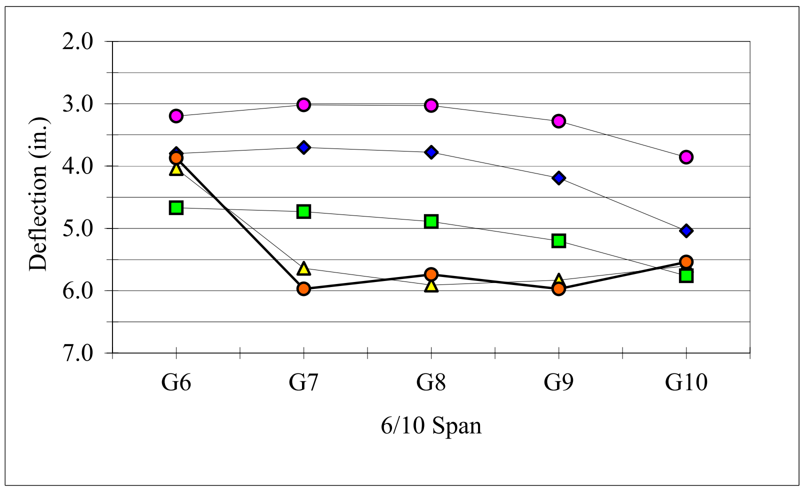
PROJECT NUMBER: B-3257 (South Wilmington Street Bridge)

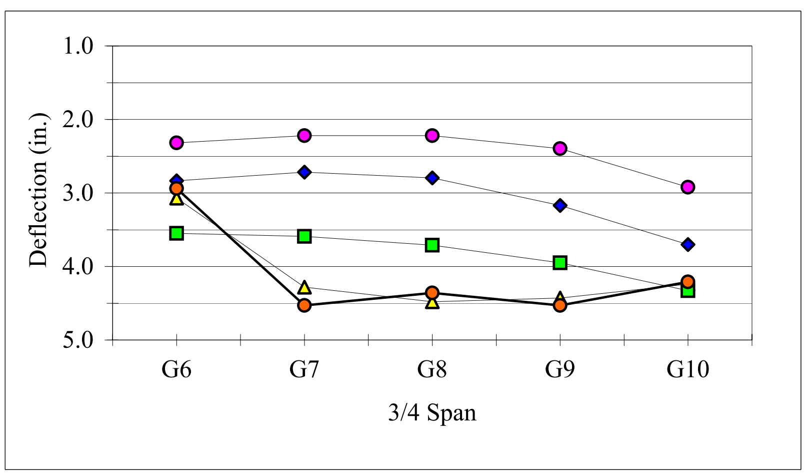
*GIRDER DEFLECTIONS
CROSS SECTION VIEW











Appendix E

Deflection Summary for the Bridge 10

This appendix contains a detailed description of the Bridge 10 from recent research by Fisher (2005). It includes bridge geometry, material data, cross-frame type and size, and dead loads calculated from slab geometry. Tables and graphs of the field measured non-composite girder deflections are included.

A summary of the SAP simplified model created for the Bridge 10 is also included in this appendix. This summary includes a picture of the SAP model, details about the elements used in the model generation, and tables and graphs of the deflections predicted by the model.

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)
MEASUREMENT DATE: March 20 & March 29, 2004

BRIDGE DESCRIPTION

TYPE	Two Span Continous, Two Simple Spans (Continuous Spans Measured)	
LENGTH	300.19 ft (91.5 m)	
NUMBER OF GIRDERS	4	
GIRDER SPACING	9.51 ft (2.9 m)	
SKEW	147.1 deg	
OVERHANG	3.02 ft (920 mm)	(from web centerline)
BEARING TYPE	Elastomeric Pad	

MATERIAL DATA

STRUCTURAL STEEL	**Grade**	**Yield Strength**
Girder:	AASHTO M270	50 ksi (345 MPa)
Other:	AASHTO M270	50 ksi (345 MPa)

CONCRETE UNIT WEIGHT	150 pcf (nominal)
SIP FORM WEIGHT	2.57 psf (CSI Catalog)

GIRDER DATA

LENGTH	155.51 ft (47.4 m)	"Span B"
	144.68 ft (44.1 m)	"Span C"

WEB THICKNESS	0.55 in (14 mm)
WEB DEPTH	75.79 in (1925 mm)

FLANGE	**Thickness**	**Begin**	**End**
Top:	1.26 in (32 mm)	0.00	112.86 ft (34.4 m)
	1.26 in (32 mm)	112.86 ft (34.4 m)	132.55 ft (40.4 m)
	1.97 in (50 mm)	132.55 ft (40.4 m)	178.48 ft (54.4 m)
	1.26 in (32 mm)	178.48 ft (54.4 m)	199.80 ft (60.9 m)
	1.26 in (32 mm)	199.80 ft (60.9 m)	300.19 ft (91.5 m)
FLANGE	**Width**	**Begin**	**End**
	15.75 in (400 mm)	0.00	112.86 ft (34.4 m)
	18.50 in (470 mm)	112.86 ft (34.4 m)	132.55 ft (40.4 m)
	18.50 in (470 mm)	132.55 ft (40.4 m)	178.48 ft (54.4 m)
	18.50 in (470 mm)	178.48 ft (54.4 m)	199.80 ft (60.9 m)
	15.75 in (400 mm)	199.80 ft (60.9 m)	300.19 ft (91.5 m)

Bottom: Same as Top Flange

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)
MEASUREMENT DATE: March 20 & March 29, 2004

STIFFENERS

Longitudinal:	N/A
Bearing:	PL 0.79" × 7.09" (20 mm × 180 mm)
Intermediate:	PL 0.47" × NA (12 mm × NA, connector plate)
	PL 0.55" × 5.91" (14 mm × 150 mm)
Middle Bearing:	PL 1.10" × 8.27" (28 mm × 210 mm)
End Bent Connector:	PL 0.47" × NA (12 mm × NA)

CROSS-FRAME DATA

	Diagonals	**Horizontals**	**Verticals**
END BENT (Type K)	WT 4×12	MC 18×42.7	WT 4×12
		WT 4×12	
MIDDLE BENT	NA	NA	NA
INTERMEDIATE (Type X)	WT 4×12	WT 4×12 (bottom)	NA

SLAB DATA

THICKNESS	8.86 in (225 mm)	nominal
BUILD-UP	2.56 in (65 mm)	nominal

Over Middle Bent:

LONGITUDINAL REBAR	SIZE (metric)	SPACING (nominal)
Top:	#19	6.69 in (170 mm)
Bottom:	#16	9.45 in (240 mm)
TRANSVERSE REBAR		
Top:	#16	6.30 in (160 mm)
Bottom:	#16	6.30 in (160 mm)

Otherwise:

LONGITUDINAL REBAR	SIZE (metric)	SPACING (nominal)
Top:	#16	13.39 in (340 mm)
Bottom:	#16	9.45 in (240 mm)
TRANSVERSE REBAR		
Top:	#16	6.30 in (160 mm)
Bottom:	#16	6.30 in (160 mm)

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)
MEASUREMENT DATE: March 20 & March 29, 2004

DECK LOADS

Girder	Concrete* lb/ft	Concrete* N/mm	Slab** lb/ft	Slab** N/mm	Ratio
G1	1014.81	14.81	1081.27	15.78	0.94
G2	1138.83	16.62	1227.91	17.92	0.93
G3	1138.83	16.62	1227.91	17.92	0.93
G4	1014.81	14.81	1081.27	15.78	0.94

* Calculated with nominal slab thicknesses

** Includes slab, buildups, and stay-in-place forms (nominal)

BEARING SETTLEMENTS*** (data in inches, negative is deflection upwards)

	Pour 1 Settlement		
Point	**End 1**	**Middle**	**End 2**
G1	0.05	0.06	---
G2	0.03	0.07	---
G3	0.03	0.06	---
G4	0.01	0.06	---

	Pour 2 Settlement		
Point	**End 1**	**Middle**	**End 2**
G1	-0.09	-0.06	0.01
G2	-0.12	-0.08	0.00
G3	-0.14	-0.08	0.01
G4	-0.18	-0.09	0.00

***Noticeably, the settlement totaled from the two pours was very close to zero.

GIRDER DEFLECTIONS (data in inches, negative is deflection upwards)

POUR 1 MEASURED

	7/10 Span C Loading				8/10 Span C Loading			
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	-0.70	-0.60	1.23	1.51	-0.83	-0.71	1.32	1.75
G2	-0.60	-0.58	0.51	1.35	-0.73	-0.69	0.61	1.55
G3	-0.69	-0.60	0.61	1.26	-0.82	-0.64	0.73	1.51
G4	-0.73	-0.55	0.62	1.44	-0.88	-0.63	0.73	1.60

	End of Span C			
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	-0.87	-0.71	1.42	1.99
G2	-0.75	-0.73	0.67	1.76
G3	-0.83	-0.71	0.78	1.66
G4	-0.89	-0.63	0.71	1.68

FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)
MEASUREMENT DATE: March 20 & March 29, 2004

GIRDER DEFLECTIONS (data in inches, negative is deflection upwards)

POUR 2 MEASURED

	3/10 Span B Loading				5/10 Span B Loading			
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	0.85	0.56	-0.08	0.20	2.08	1.39	-0.38	-0.12
G2	0.98	0.54	-0.15	-0.04	2.10	1.34	-0.43	-0.36
G3	1.08	0.70	-0.08	0.10	2.13	1.50	-0.36	-0.19
G4	1.46	0.79	-0.19	0.07	2.63	1.66	-0.49	-0.24

	7/10 Span B Loading				Middle Bent Loading			
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	2.91	2.02	-0.57	-0.32	3.28	2.41	-0.73	-0.50
G2	2.76	1.90	-0.62	-0.58	3.07	2.23	-0.75	-0.69
G3	2.77	2.09	-0.58	-0.40	3.01	2.34	-0.68	-0.51
G4	3.26	2.31	-0.76	-0.51	3.40	2.52	-0.86	-0.60

	Complete Loading			
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	2.83	2.00	-0.23	0.08
G2	2.66	1.83	-0.28	-0.12
G3	2.57	1.92	-0.22	0.00
G4	2.92	1.99	-0.33	-0.04

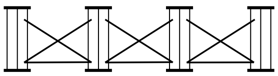
TOTAL MEASURED

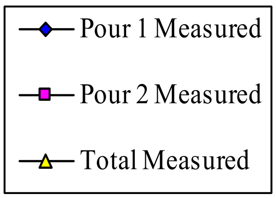
	Super-Imposed Total			
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	1.97	1.29	1.18	2.07
G2	1.91	1.10	0.39	1.64
G3	1.74	1.21	0.56	1.66
G4	2.02	1.36	0.38	1.64

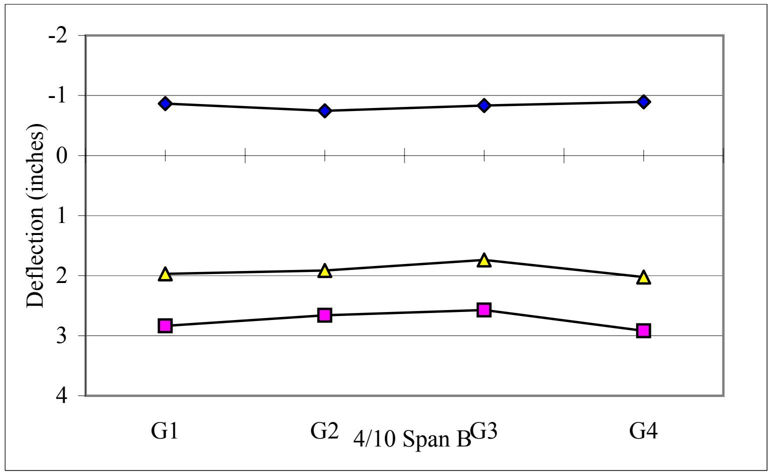
FIELD MEASUREMENT SUMMARY

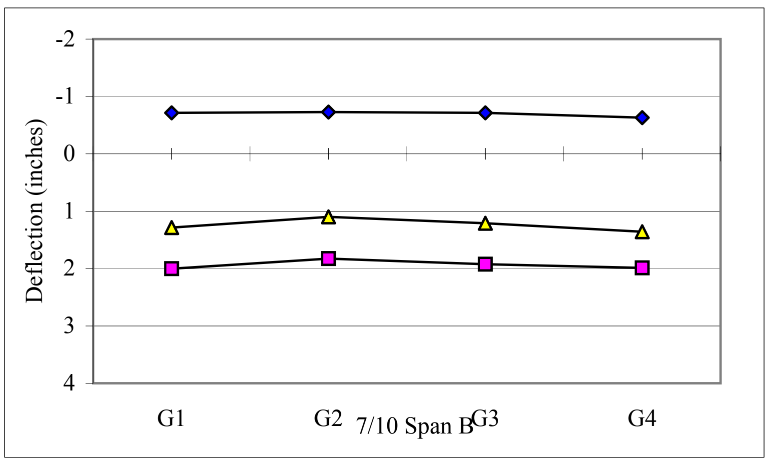
PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)
MEASUREMENT DATE: March 20 & March 29, 2004

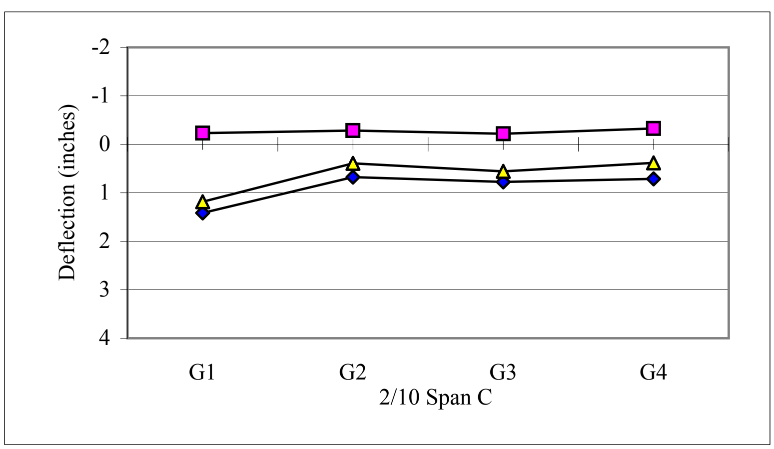
GIRDER DEFLECTIONS
CROSS SECTION VIEW







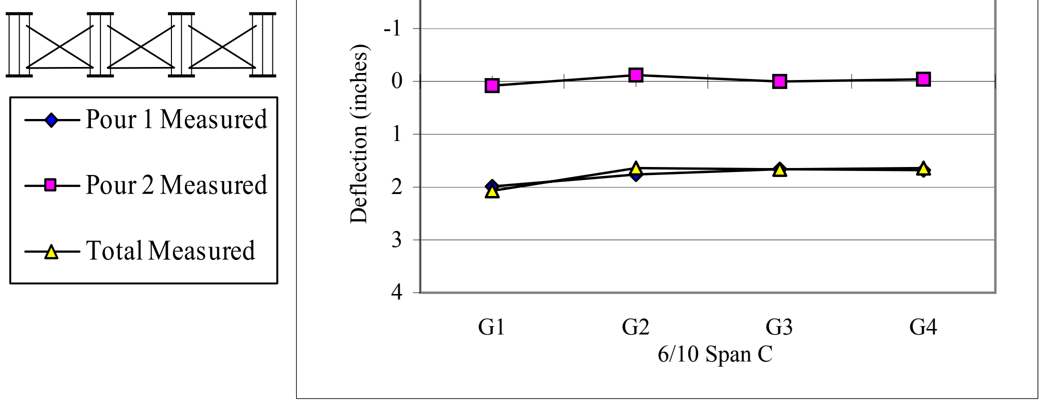


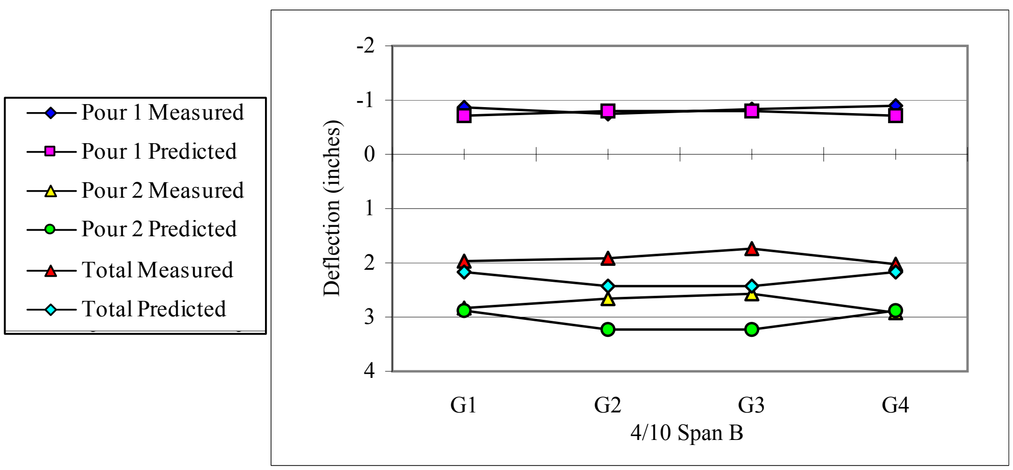


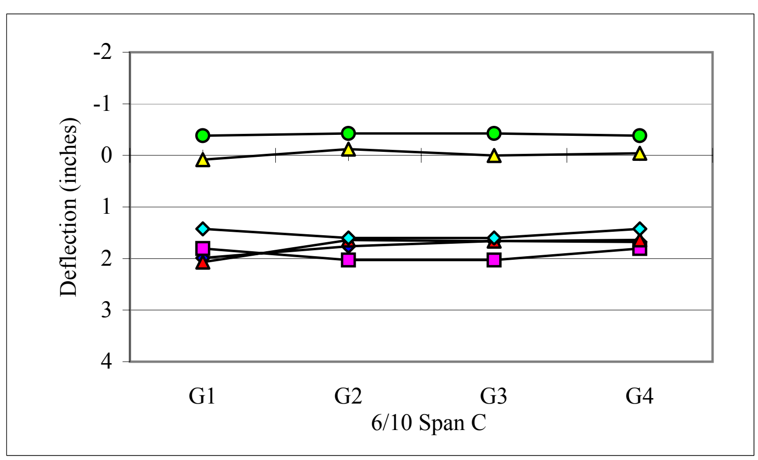
FIELD MEASUREMENT SUMMARY

PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)
MEASUREMENT DATE: March 20 & March 29, 2004

GIRDER DEFLECTIONS
CROSS SECTION VIEW



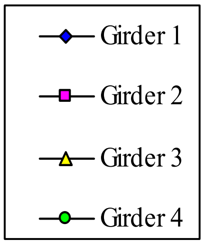


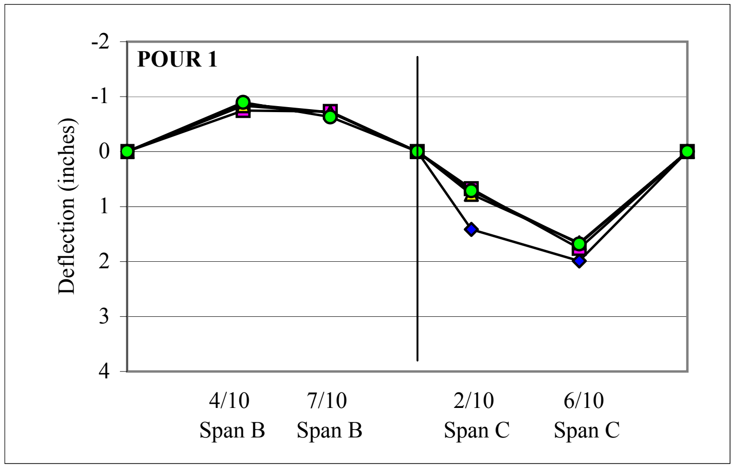


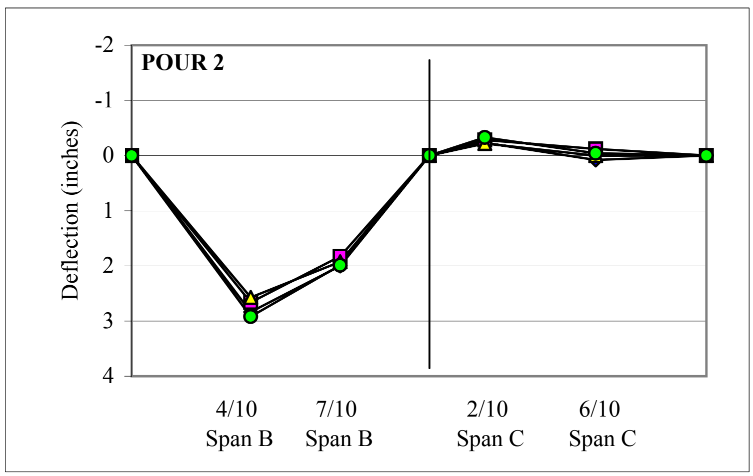
FIELD MEASUREMENT SUMMARY

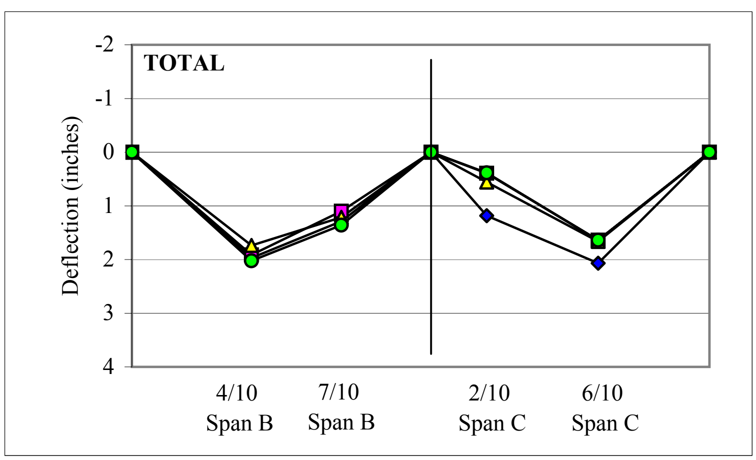
PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)
MEASUREMENT DATE: March 20 & March 29, 2004

GIRDER DEFLECTIONS
ELEVATION VIEW





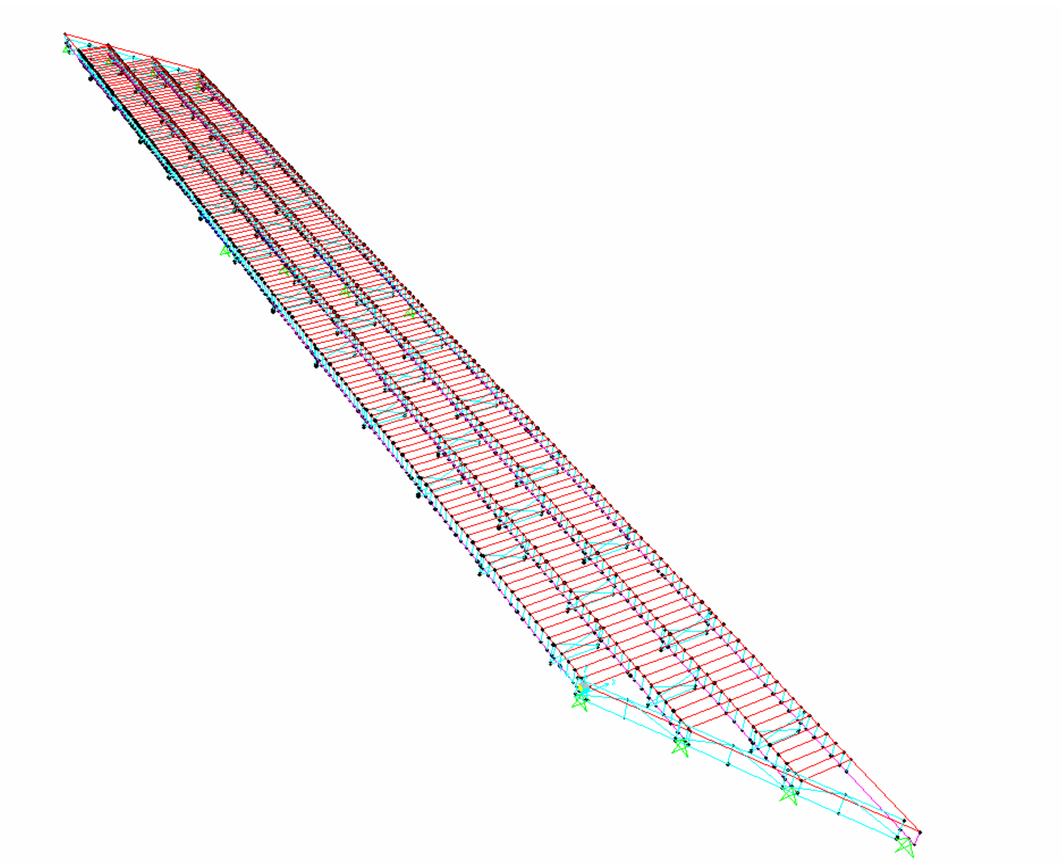




SAP 2000 MODELING SUMMARY

PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)

MODEL PICTURE (Steel only, isometric view)



SAP2000 MODELING SUMMARY

PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)

MODEL DESCRIPTION

COMPONENT	**Element Type**
Girder:	Frame Element
Cross Frame Members:	Frame Element
Stay-in-place Deck Forms:	Area Element* (Shell Element)
Rigid Link:	Frame Element

* See Shell Properties in Appendix F

GIRDER DEFLECTIONS

SAP Pour 1					SAP (SIP) Pour 1			
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	2.74	1.89	0.09	0.94	-0.68	-0.74	0.75	1.54
G2	3.03	2.09	0.11	1.05	-0.69	-0.75	0.78	1.63
G3	3.03	2.10	0.10	1.05	-0.68	-0.75	0.77	1.62
G4	2.76	1.91	0.08	0.93	-0.66	-0.72	0.73	1.53
SAP Pour 2					**SAP (SIP) Pour 2**			
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	3.36	2.57	-0.59	-0.45	3.42	2.62	-0.60	-0.46
G2	3.70	2.83	-0.64	-0.48	3.62	2.76	-0.61	-0.47
G3	3.70	2.83	-0.64	-0.49	3.62	2.77	-0.62	-0.46
G4	3.37	2.58	-0.60	-0.46	3.45	2.64	-0.62	-0.48
SAP Super Imposed					**SAP (SIP) Super Imposed**			
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	2.69	1.84	0.15	1.1	2.74	1.88	0.15	1.08
G2	2.99	2.05	0.16	1.19	2.93	2.01	0.17	1.16
G3	3.00	2.06	0.15	1.17	2.94	2.02	0.15	1.16
G4	2.72	1.87	0.12	1.04	2.79	1.92	0.11	1.05

SAP2000 MODELING SUMMARY

PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)

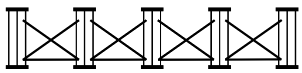
GIRDER DEFLECTIONS

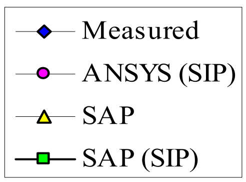
ANSYS Pour 1 Loading (SIP)				
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	-0.56	-0.60	0.69	1.51
G2	-0.49	-0.52	0.60	1.38
G3	-0.47	-0.49	0.58	1.34
G4	-0.48	-0.51	0.59	1.40
ANSYS Pour 2 Loading (SIP)				
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	2.30	1.57	-0.29	-0.20
G2	2.21	1.56	-0.27	-0.20
G3	2.27	1.61	-0.31	-0.22
G4	2.45	1.77	-0.38	-0.29
ANSYS Total (SIP)				
Point	**4/10 B**	**7/10 B**	**2/10 C**	**6/10 C**
G1	1.74	0.98	0.40	1.30
G2	1.72	1.04	0.33	1.17
G3	1.80	1.12	0.28	1.11
G4	1.97	1.26	0.21	1.11

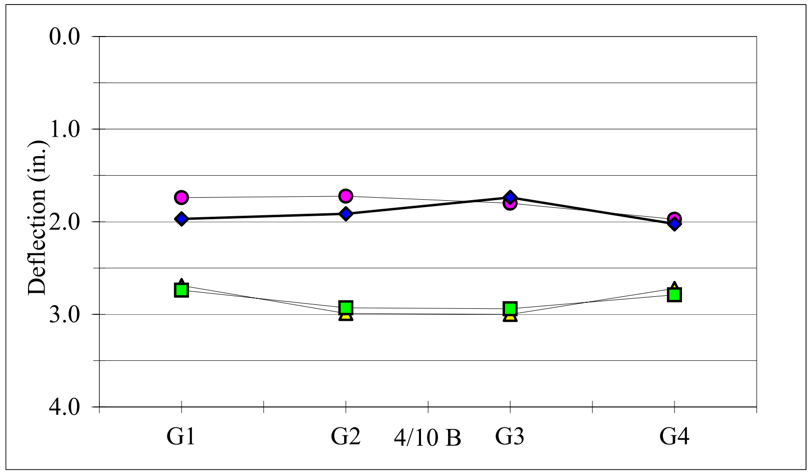
SAP 2000 MODELING SUMMARY

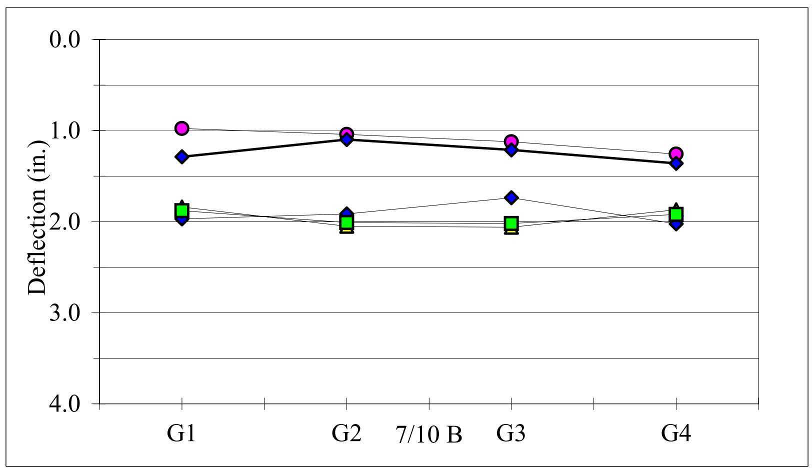
R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)

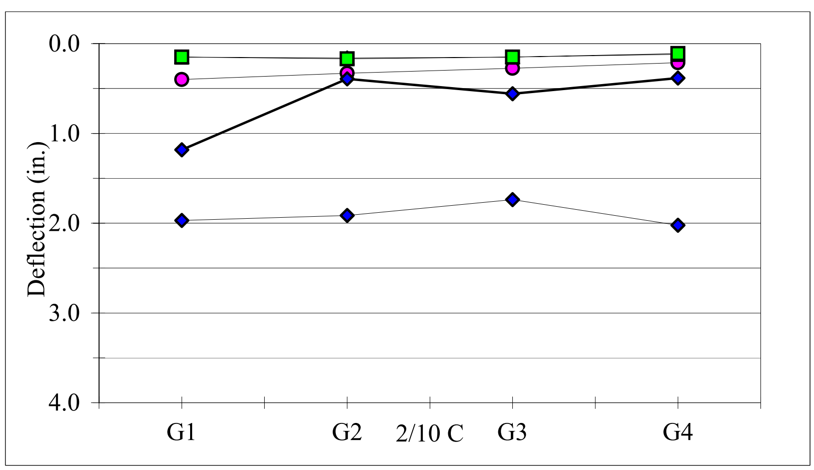
CROSS SECTION VIEW







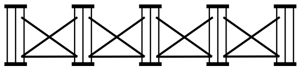


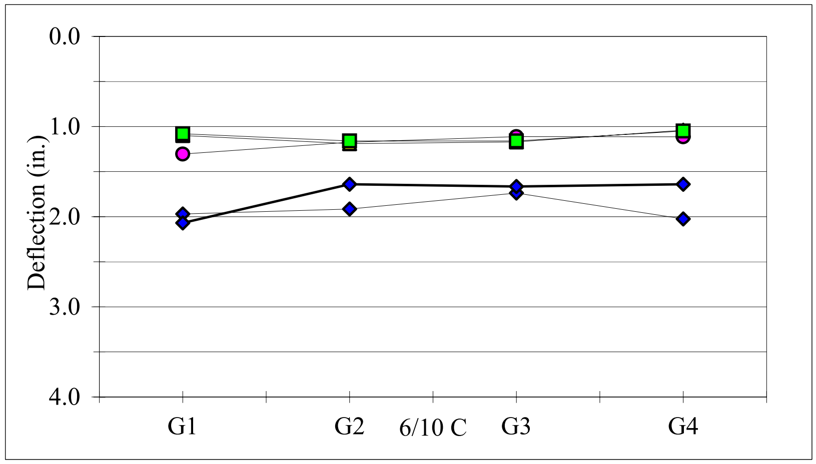


SAP 2000 MODELING SUMMARY

PROJECT NUMBER: R-2547 (Knightdale-Eagle Rock Rd. Over US-64 Bypass)

*GIRDER DEFLECTIONS
CROSS SECTION VIEW





Appendix F

Sample Calculation of SIP Deck Metal Form Properties

This Appendix contains a sample calculation of the SIP metal deck form properties that were used in the SAP bridge models. The geometry and properties of the shell element used in SAP models for each bridge model were tabulated and included in here.

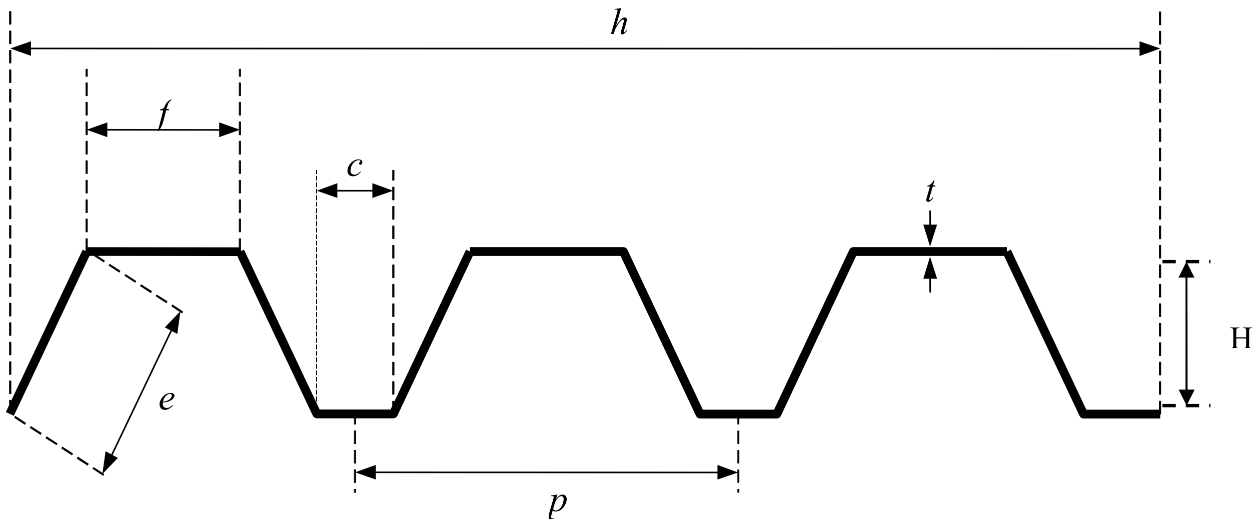


Figure F-1 Typical Stay-in-Place Metal Deck Form Profile

Table F-1 Stay-in-Place Metal Deck Form Data

Bridge	*H* (in.)	*h* (in.)	*p* (in.)	*f* (in.)	*c* (in.)	*e* (in.)	*t* (in.)
US 29	2.5	32	8	5	2.5	2.51	0.036
Wilmington St.	2.5	32	8	5	2.5	2.51	0.036
Bridge 8	3	24	8	5.25	1.75	3.04	0.067
Eno	3	24	8	5.25	1.75	3.04	0.036
Bridge 10	3	24	8	5.25	1.75	3.04	0.036

Sample Calculation of Stay-in-place Metal Deck Form Properties for US 29:

Calculate flattened out panel width, w:

$$w = (8\text{x } e) + (4\text{x } f) + (4\text{x } c) = (8\text{x}2.51) + (4\text{x}5) + (4\text{x}2.5) = 50.08 \; in.$$

Calculate total cross-area section of the panel, A_{Panel}.

$A_{Panel} = w \text{ x } t = 50.08\text{x}.0.036 = 1.80\ in.^2$

Calculate Shell Element thickness, *th*.

$$th = \frac{A_{Panel}}{w} = \frac{1.80}{32} = 0.056\ in.$$

Calculate Shell Element thickness of bending, *thb*.

$$I = \frac{1}{12}bt^3 : \frac{32}{12}x0.6302 = \frac{1}{12}x32xt^3$$

$t = thb = 0.84\ in.$

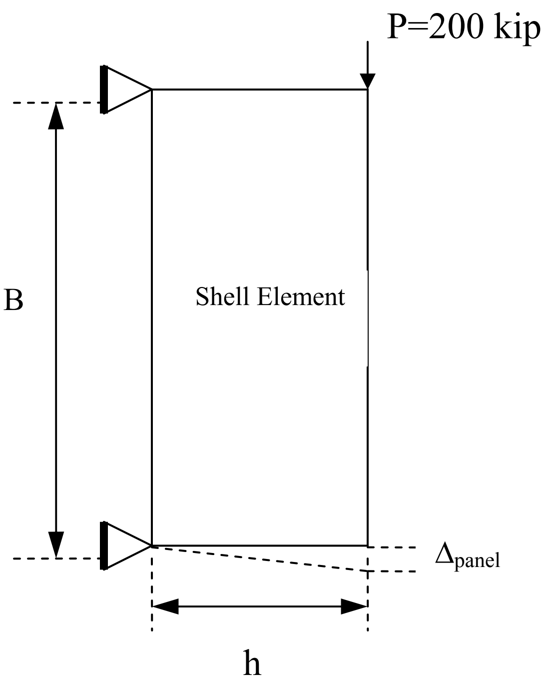
where: I = Moment of inertia of the SIP (CSI catalog)
b = Width of the SIP *(h)*
t = Thickness of Bending *(thb)*

Calculate the stiffness modifier, $f_{11.}$

$$f_{11} = \frac{\textit{Shell Element Thickness}}{\textit{SIP Thickness}} = \frac{0.056}{0.036} = 1.56\ in.$$

Using Trial & Error by changing the shear modulus of the panel until obtain the same deflection.

Calculate the shear stiffness of the Panel using Analytical model and SAP 2000



From Jetann (2002) G' = 11 *kip/in.*

$$\Delta_{panal} = \frac{Ph}{G'B} = \frac{200x32}{11x12x7.75} = 6.26\,ft$$

where: P = applied force (used 200 kip)
h = SIP width
B = Girder Spacing

Figure F-2 SAP, SIP Diaphragm Analytical Model, f_{12}

By assigning the thickness of the shell element equal to the real thickness of SIP form and using Trial & Error by changing the shear modulus of the panel until obtain the same deflection as analytical results.

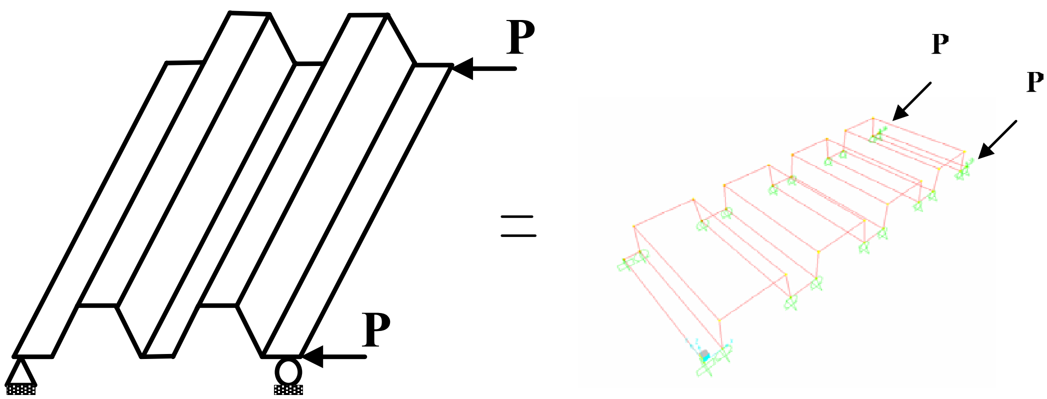
By Trial & Error

Shear modulus = 26.445 *kip/ft*

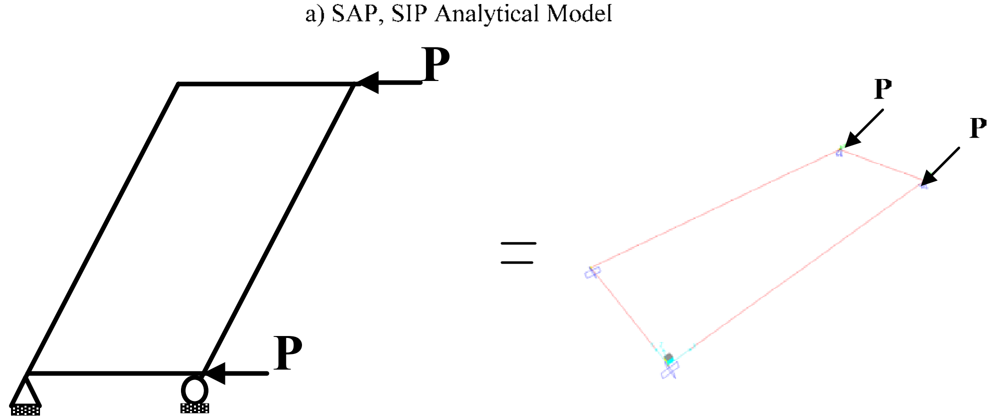
Calculate the stiffness modifier, f_{12}

$$f_{12} = \frac{\textit{Shear modulus of Shell Element}}{\textit{Shear Modulus of SIP}} = \frac{26.445}{11153.846} \approx 0.00237$$

Calculate stiffness modifier f_{22} by using Trial & Error and SAP modeling.



a) SAP, SIP Analytical Model



b) SAP, Shell Element Analysis

Figure F-3 SAP, f_{22} Analytical Models

Using Trial & Error to get the thickness of the simulate panel

Thickness of the panel = 2.5x10 $^{-6}$ *in.*

Calculate stiffness modifier, f_{22}

$$f_{22} = \frac{Thickness\ of\ Panel}{Thickness\ of\ SIP} = \frac{2.5x10^{-6}}{0.036} \approx 0.00007$$

Calculate stiffness modifier, m_{22}

$$Moment\ resistance\ of\ member \propto t^3$$

$$m_{22} = \frac{(Thickness\ in\ direction\ 22)^3}{(Thickness\ in\ direction\ 11)^3} = \frac{(0.036)^3}{(0.86)^3} \approx 0.00007$$

Calculate stiffness modifier, m_{12}

$$Moment\ resistance\ of\ member \propto t^3$$

$$m_{22} = \frac{(Thickness\ in\ direction\ 12)^3}{(Thickness\ in\ direction\ 11)^3} = \frac{(0.036)^3}{(0.86)^3} \approx 0.00007$$

The following tables contain the SIP properties and modifier calculated for each section followed CSI catalog:

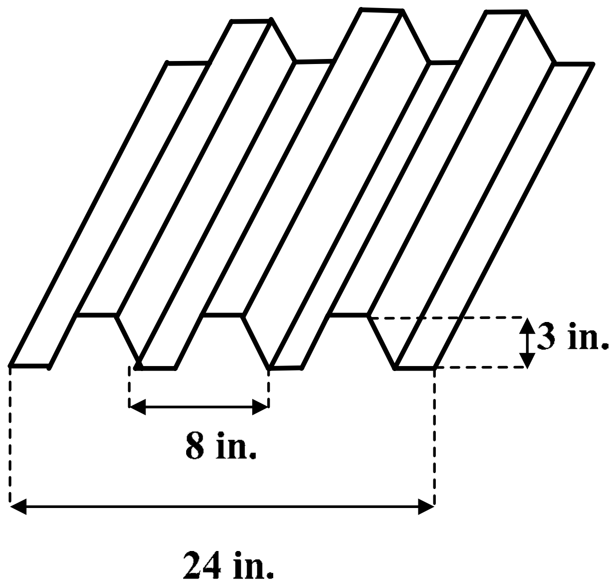


Figure F-4 SIP Form 24 in. Cover Width

Table F-2 SIP Form Properties used in SAP2000 Shell Element for SIP Form 24 in. Cover Width

Gage	Thickness (in.)	Cross Section Area (in.^2)	I (in.^4)	Thickness of Bending of simulated Pan (in.)	SIP stiffness modifier				
					f11	f22	m11	m22	m12
22	0.03	1.18	0.7704	0.917	1.635	0.00004	1	3.50E-05	3.50E-05
21	0.033	1.29	0.8601	0.951	1.635	0.00004	1	4.18E-05	4.18E-05
20	0.036	1.41	0.9511	0.983	1.635	0.00005	1	4.91E-05	4.91E-05
19	0.042	1.65	1.1368	1.044	1.635	0.00007	1	6.52E-05	6.52E-05
18	0.047	1.84	1.2946	1.090	1.635	0.00009	1	8.02E-05	8.02E-05
17	0.053	2.08	1.4872	1.141	1.635	0.00014	1	1.00E-04	1.00E-04
16	0.059	2.32	1.6593	1.184	1.635	0.00018	1	1.24E-04	1.24E-04
15	0.067	2.63	1.8843	1.235	1.635	0.00019	1	1.60E-04	1.60E-04
14	0.074	2.90	2.0811	1.277	1.635	0.00038	1	1.95E-04	1.26E-04

Table F- 3 SIP Form Property, f_{12}, with Different Girder Spacing for SIP Form 24 in. Cover Width

Gage	Thickness (in.)	f 12 with different spacing (ft)						
		7	8	9	10	11	12	13
22	0.03	0.00270	0.00252	0.00243	0.00243	0.00250	0.00274	0.00274
21	0.033	0.00251	0.00231	0.00227	0.00227	0.00228	0.00250	0.00249
20	0.036	0.00224	0.00211	0.00202	0.00202	0.00228	0.00229	0.00226
19	0.042	0.00193	0.00179	0.00178	0.00178	0.00195	0.00197	0.00191
18	0.047	0.00175	0.00158	0.00158	0.00158	0.00173	0.00177	0.00174
17	0.053	0.00157	0.00142	0.00134	0.00134	0.00157	0.00156	0.00156
16	0.059	0.00142	0.00126	0.00126	0.00128	0.00141	0.00140	0.00141
15	0.067	0.00121	0.00114	0.00113	0.00113	0.00126	0.00123	0.00123
14	0.074	0.00108	0.00101	0.00101	0.00101	0.00116	0.00110	0.00112

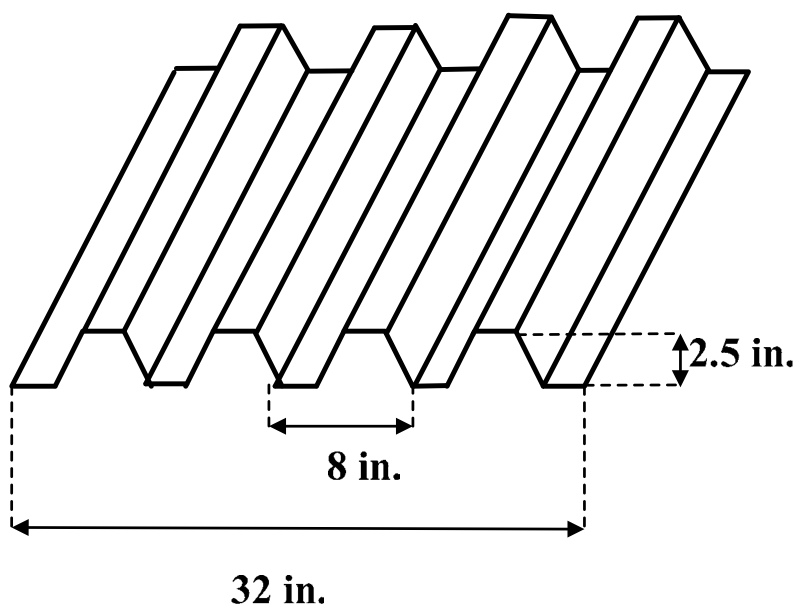


Figure F-5 SIP Form 32 in. Cover Width

Table F- 4 SIP Form Properties used in SAP2000 Shell Element for SIP Form 32 in. Cover Width

Gage	Thickness (in.)	Cross Section Area (in.^2)	I (in.^4)	Thickness of Bending of simulated Pan (in.)	SIP stiffness modifier				
					f11	f22	m11	m22	m12
22	0.03	1.50	0.5251	0.807	1.565	0.00004	1	5.14E-05	5.14E-05
21	0.033	1.65	0.5777	0.833	1.565	0.00005	1	6.22E-05	6.21E-05
20	0.036	1.80	0.6302	0.857	1.565	0.00007	1	7.40E-05	7.40E-05

Table F-5 SIP Form Property, f_{12}, with Different Girder Spacing for SIP Form 32 in. Cover Width

Gage	Thickness (in.)	f 12 with different spacing (ft)						
		7	8	9	10	11	12	13
22	0.03	0.00272	0.00281	0.00281	0.00285	0.00283	0.00250	0.00274
21	0.033	0.00255	0.00256	0.00256	0.00248	0.00244	0.00249	0.00249
20	0.036	0.00237	0.00230	0.00230	0.00229	0.00229	0.00230	0.00229