ENGINEERING METHODOLOGY FOR CONSIDERING PERMANENT METAL DECK FORMS FOR STABILITY OF BRIDGES DURING CONSTRUCTION

by

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A THESIS

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Lateral-torsional buckling is a significant consideration in the design of steel bridge girders during construction. Because I-girders have inherently weak torsion resistance, designers provide cross-frames and/or diaphragms at close spacing intervals to minimize the susceptibility of individual girders to instability during construction. A recent increase in fatigue problems around discrete brace connections, along with the costs of fabrication, erection, and inspection associated with cross-frames, has prompted the removal of the minimum spacing requirement from bridge specifications and created great interest in identifying alternative construction bracing approaches. Although permanent metal deck forms (PMDF) are widely used in the construction of steel bridges today, the stability they may provide is not considered in construction sequence engineering. The overall objective of this project was, therefore, to improve bridge design efficiency and construction safety by developing strength definition and engineering methodology that considers the contribution of PMDF to stability during the construction of steel girder bridges. Global tasks included the following: (1) synthesizing all relevant literature; (2) synthesizing state-of-the art design and construction practice relevant to PMDF, including connection details, (3) developing preliminary engineering approach and concepts and identify research focus, and (4) using advanced finite element methodology to develop and verify proposed design methodology.
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TABLE OF CONTENTS  

Page  

ABSTRACT ...................................................................................................................ii  
ACKNOWLEDGEMENT ..............................................................................................iii  
LIST OF TABLES .........................................................................................................vi  
LIST OF FIGURES ........................................................................................................vii  
LIST OF ABBREVIATIONS .........................................................................................ix  

CHAPTER  

1 INTRODUCTION ........................................................................................................1  
1.1 Research overview .........................................................................................1  
1.2 Objective ........................................................................................................3  
1.3 Scope and Methodology .................................................................................4  

2 BACKGROUND ....................................................................................................5  
2.1 Lateral Torsional Buckling .............................................................................5  
2.2 Metal Deck Form Building Application .......................................................9  
2.3 Potential Bridge Application ........................................................................11  
2.4 Beam Bracing ...............................................................................................15  
2.5 Overall PMDF System Overview .................................................................17  
2.5.1 Forming System .......................................................................................17  
2.5.2 Deck Attachments ..................................................................................20  
2.5.2.1 Welded Connection Details ..............................................................20  
2.5.2.2 Strap Connection Details .................................................................21  
2.5.3 Support Angle Configuration ..................................................................22  
2.5.4 Mechanical Fastener Assembly ...............................................................24  

3 LITERATURE SYNTHESIS ..............................................................................26  
3.1 Applications of PMDF System ......................................................................26
3.1.1 Buildings ................................................................. 26
3.1.2 Bridges ................................................................. 28
3.2 Permanent Metal Deck Form Characteristics ......................... 34
  3.2.1 Shear Stiffness of PMDF ........................................ 34
  3.2.2 Shear Strength of PMDF ...................................... 37
3.3 Metal Deck Form Contribution to Brace Bridge Girders ............... 38
3.4 Transverse Stiffening Angle Contribution to the Bracing Behavior of
  PMDF System ................................................................. 46
3.5 Strength Requirement for Shear Diaphragm Bracing ................... 48
3.6 Effect of Fastener Spacing, Panel Width and Girder Spacing ........ 49

4 PROPOSED DESIGN METHODOLOGY ........................................ 51
  4.1 Overview ................................................................. 51
  4.2 Design Recommendations .......................................... 52
    4.2.1 Recommendations for Stiffness .......................... 52
    4.2.2 Recommendations for Strength ......................... 53
  4.3 Design Method ....................................................... 54

5 FINITE ELEMENT ANALYSIS TECHNIQUE AND RESULTS ............ 63
  5.1 Executive Summary and Introduction ............................. 63
  5.2 Plate Girders ........................................................... 67
  5.3 Stiffeners and Supporting angles ................................. 68
  5.4 Permanent Metal Deck Forms ..................................... 70
  5.5 Cross-Frames and Diaphragms .................................... 73
  5.6 Load Calculation and Application ................................ 76
  5.7 Modeling Comparisons and Results .............................. 79
    5.7.1 Summary of ANSYS Results Without PMDF
          contribution ...................................................... 79
    5.7.2 Summary of ANSYS Results With PMDF
          contribution ..................................................... 81
  5.8 Dynamic Behavior of Girders with PMDF ......................... 85

6 SUMMARY AND FUTURE WORK .............................................. 91

LIST OF REFERENCES .......................................................... 97

APPENDIX: ............................................................................. 100
  A LATERAL STIFFNESS CALCULATIONS FROM ANSYS DEFLECTION
     RESULTS ............................................................... 100
  B LATERAL DEFLECTION FOR GIRDER WITH AND WITHOUT PMDF.. 102
  C BRIDGE DESIGN EXAMPLE .............................................. 103
LIST OF TABLES

<table>
<thead>
<tr>
<th>Table</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>Design values of m</td>
</tr>
<tr>
<td>4.1</td>
<td>Design k values</td>
</tr>
<tr>
<td>5.1</td>
<td>Lateral displacements for girders without PMDF</td>
</tr>
<tr>
<td>5.2</td>
<td>Lateral displacements for girders with PMDF</td>
</tr>
<tr>
<td>5.3</td>
<td>Lateral stiffness for girders with PMDF</td>
</tr>
<tr>
<td>5.4</td>
<td>Natural frequencies for girders with and without PMDF</td>
</tr>
<tr>
<td>5.5</td>
<td>Natural period for girders with and without PMDF</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>Metal deck form building application</td>
<td>10</td>
</tr>
<tr>
<td>2.2</td>
<td>Metal deck form bridge application</td>
<td>11</td>
</tr>
<tr>
<td>2.3</td>
<td>Typical bridge application with differential camber between adjacent girders</td>
<td>13</td>
</tr>
<tr>
<td>2.4</td>
<td>Short-span shallow and long-span deep bridge plate girder assembly</td>
<td>14</td>
</tr>
<tr>
<td>2.5</td>
<td>Metal deck form plan and cross section view</td>
<td>19</td>
</tr>
<tr>
<td>2.6</td>
<td>Typical welded connection detail in bridge industry</td>
<td>20</td>
</tr>
<tr>
<td>2.7</td>
<td>Typical strap angle connection detail in bridge industry</td>
<td>21</td>
</tr>
<tr>
<td>2.8</td>
<td>Fields of tension and compression within the panel system</td>
<td>23</td>
</tr>
<tr>
<td>3.1</td>
<td>Girder/metal deck form connection with stiffening angle</td>
<td>32</td>
</tr>
<tr>
<td>3.2</td>
<td>Shear stiffness determination of girder/metal deck form system</td>
<td>36</td>
</tr>
<tr>
<td>3.3</td>
<td>Girder failure modes when braced with diaphragm on compression flanges</td>
<td>39</td>
</tr>
<tr>
<td>5.1</td>
<td>Single plate girder model</td>
<td>68</td>
</tr>
<tr>
<td>5.2</td>
<td>Oblique and front view of girder with stiffeners</td>
<td>69</td>
</tr>
<tr>
<td>5.3</td>
<td>3-D metal deck form model</td>
<td>71</td>
</tr>
<tr>
<td>5.4</td>
<td>Typical Girder Connection detail</td>
<td>72</td>
</tr>
<tr>
<td>5.5</td>
<td>Isometric and plan view of Girder/PMDF system Model</td>
<td>73</td>
</tr>
<tr>
<td>Figure</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
<td>------</td>
</tr>
<tr>
<td>5.6</td>
<td>K-type Cross-frame FEM Model</td>
<td>74</td>
</tr>
<tr>
<td>5.7</td>
<td>3-D finite element model of four girders with cross-frame</td>
<td>77</td>
</tr>
<tr>
<td>5.8</td>
<td>3-D finite element model of four girders with metal deck form</td>
<td>78</td>
</tr>
<tr>
<td>5.9</td>
<td>Deflected finite element model of twin girder system</td>
<td>80</td>
</tr>
<tr>
<td>5.10</td>
<td>Deflected profile for girders with PMDF</td>
<td>82</td>
</tr>
<tr>
<td>5.11</td>
<td>Mode shapes for two girder system without PMDF</td>
<td>89</td>
</tr>
<tr>
<td>5.12</td>
<td>Mode shapes for four girder system with PMDF</td>
<td>90</td>
</tr>
</tbody>
</table>
# LIST OF ABBREVIATIONS

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Full Form</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>ALDOT</td>
<td>Alabama Department of Transportation</td>
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<td>FEM</td>
<td>Finite Element Modeling</td>
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<td>AISC</td>
<td>American Institute of Steel Construction</td>
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<tr>
<td>SDI</td>
<td>Steel Deck Institute</td>
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<tr>
<td>PMDF</td>
<td>Permanent Metal Deck Form</td>
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<tr>
<td>SSRC</td>
<td>Structural Stability Research Council</td>
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<td>LRFD</td>
<td>Load and Resistance Factor Design</td>
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</tbody>
</table>
CHAPTER 1
INTRODUCTION

1.1 Research Overview

Lateral-torsional buckling often controls the design of steel bridge girders during construction. Lateral-torsional buckling can be roughly characterized as a phenomenon of sudden lateral displacement coupled with rotation of girder length that is not sufficiently braced. The predominant geometric characteristics that define the resistance to lateral torsional buckling are torsional rigidity, lateral bending rigidity, and length between brace points. The susceptibility of I-shaped plate girders used in bridges to instability during construction is largely due to the fact that they are optimized to carry vertical load in the composite traffic-bearing configuration of the completed bridge structure but have inherently weak torsion resistance during the various phases of construction prior to the hardening of the concrete deck. In addition to lateral torsional instability, bridge girders are also prone to load-induced instability during construction from such circumstances as lifting and handling, inadequate temporary shoring, concrete deck machinery weight, and wind loads. Therefore, designers have traditionally provided cross-frames and/or diaphragms (discrete point bracing) at close spacing intervals to minimize the susceptibility of individual girders to instability during construction.
Prior to the most recent LRFD (Load and Resistance Factor Design) Standard Specifications for Highway Bridges (AASHTO), the maximum spacing between cross-frames and diaphragms was limited to 25 feet. However, following a recent increase in awareness and incidence of fatigue problems encountered around discrete brace connections, the maximum spacing requirement was removed from the LRFD bridge design specifications. Furthermore, cross-frames and diaphragms complicate girder fabrication and erection, which leads to increased construction costs and greatly increases long-term inspection costs. For these reasons, among others, alternative bracing systems and engineering methodologies are needed.

Deck forms are used to support wet concrete during construction in both buildings and bridge industries before the concrete cures and composite action between the concrete slab and the steel girders is achieved. The girders alone must carry the loads induced during construction. The building design and construction industry has long relied on the in-plane strength and stiffness of metal deck forms. In the typical installation of deck forms to resist lateral-torsional buckling, AASHTO (American Association of State Highway and Transportation Officials) concrete decking bridge specifications do not allow deck forms to considered for providing stability. The primary difference between the formwork used in the building and bridge industries are the connection details that are employed between the formwork and the steel girders. In building construction, the forms are typically fastened directly to the top flange using mechanical fasteners or by welding shear studs directly through the formwork. Bridge decking is typically connected to the girders using steel angles that allow the contractor to adjust the form elevation for changes in flange thickness along the girder length and/or
differential camber between adjacent girders. These support angles lead to eccentric connections that reduce the in-plane stiffness available to resist lateral instability.

The overall objective of the effort is therefore to improve bridge design efficiency and construction safety by developing strength definition and engineering methodology that considers the contribution of concrete deck forms to stability during the construction of steel girder bridges. Design calculations with improved connection details were presented using a permanent metal deck form as a bracing element for the stability of girders during construction. The results will lead to safer and more-efficient construction practices and the ability to minimize the number of temporary and permanent cross-frames, which would reduce the incidence of fatigue problems and routine inspection costs.

1.2 Objective

The overall objective of the project is to improve bridge design efficiency and construction safety by developing strength definition and engineering methodology that considers the contribution of metal deck forms to stability during the construction of steel girder bridges. Additional goals of the project include providing a systematic design procedure considering a permanent metal deck form as bracing element, and opportunities for integrating improved construction stability practices into ALDOT (Alabama Department of Transportation) standard specifications.
1.3 Scope and Methodology

The scope of the project entails a thorough review and synthesis of relevant literature and current practice, the development of concepts to improve engineering and construction practice, the use of FEM (Finite Element Modeling) to investigate system behavior, and the dissemination of results. This study leverages recent testing and analytical research conducted by the University of Texas at Austin and the University of Texas at Houston, and a part of this report is essentially an orientation to the existing literature. This thesis will address safety aspects by investigating the stability during construction and the parameters necessary to understand the bracing behavior of permanent metal deck forms.
CHAPTER 2
BACKGROUND

2.1 Lateral Torsional Buckling

The flexural capacity of beams with large, unbraced length is often limited by a mode of failure known as lateral torsional buckling, which generally involves both an out-of-plane displacement and a twist of the beam cross-section. The perfectly straight beam that is subjected to bending moments around the strong axis will deflect in the plane of applied moments until the moments reach a certain critical value. When the buckling moment is reached, lateral torsional buckling is initiated by lateral deflection and the twisting of the beam. Because of the lateral torsional buckling behavior of beams, bracing requirements of beams are more complex than those of columns. Four different types of braces, lateral, rotational, warping, and torsional, can be used individually or in combination to prevent lateral torsional buckling. The location of the braces within the cross-section influences the effectiveness of each.

Elastic torsional buckling strength of beams was solved mathematically by Timoshenko and Gere [1961], and presented in the equation (2.1) for the elastic critical buckling moment of doubly symmetric beams under uniform moment. It is applicable to beams where a twist of the unbraced length is prevented.
The standard AASHTO specifications provide expression for estimating the lateral torsional buckling capacity of singly and doubly symmetric girder cross-sections and can be calculated by using the equation (2.2).

\[
M_{cr} = \frac{\pi}{L_b} \sqrt{EI_yGJ + \frac{\pi^2 E^2 I^2_y h^2}{4L_b^2}} \tag{2.1}
\]

\[
M_{AASHTO} = 91 \times 10^8 \left( \frac{I_{yc}}{L_b} \right) \left[ 0.772 \left( \frac{J}{I_{yc}} \right) + 9.87 \left( \frac{d}{L_b} \right)^2 \right] \tag{2.2}
\]

Where,

- \(L_b\) = Unbraced length (the distance between points of full lateral support)
- \(E\) = Modulus of Elasticity
- \(I_y\) = Weak axis moment of inertia
- \(G\) = Shear modulus
- \(J\) = Torsional constant
- \(h\) = Distance flange centroids

Lateral torsional buckling involves a twist of the cross-section and a lateral movement of the compression flange. When the top flange is in compression, the metal deck form behaves like a shear diaphragm to brace the flange from lateral movement. The lateral movement of the top flange imposes a shearing distortion on the deck panel, and the ability of the deck to resist shear distortion is available to brace the girder.

For the girders subjected to constant moment, Helwig [1994] showed that the contribution of the metal deck form in increasing the buckling was somewhat linear and dependent on the girder depth, and was not significantly affected by girder length or other cross-sectional properties. For constant moment, the recommendations by Nethercot and
Trahair [1975] and Errera and Apparao [1976] provide a good estimate of buckling capacity.

\[ M_{cr} = M_{AASHTO} + Q d \]  

(2.3)

The buckling load for the cases with moment gradient are actually less than the buckling capacity for cases with uniform moment. To account for the moment gradient effect, Helwig [1994] found that, instead of applying \( C_b \) to the entire expression, which produces un-conservative results, apply \( C_b \) factor to \( M_{AASHTO} \) only.

\[ M_{cr} = C_b M_{AASHTO} + Q d \]  

(2.4)

In general, Helwig [1994] showed that the effectiveness of the deck is reduced when the girders are subjected to moment gradient.

The fourth edition of SSRC (Structural Stability Research Council) [2007] has tabulated factors that can be used for load height effects in doubly-symmetric beams by modifying the \( C_b \) factor. The modified \( C_b \) value for the load height effects will be referred to as \( C_b^* \). The buckling capacity is calculated by multiplying the buckling moment from the AASHTO equation to the value of \( C_b^* \) for the corresponding load case.

i.e.  

\[ M_{cr} = C_b^* M_{AASHTO} \]  

(2.5)

This will provide the estimated capacity of girder without the metal deck form. The load height factor \( C_b^* \) is calculated as the ratio of \( C_b \) to \( B \).

\[ C_b^* = \frac{C_b}{B} \]  

(2.6)

The \( C_b \) factor accounts for the moment gradient along the girder length and can be calculated using AISC specifications. The method in SSRC [2007] proceedings guide uses a different combination of two variables, \( A \) and \( B \) to account for load height effects.
The variable $A$ is defined as traditional $C_b$ value, the SSRC [2007] guide recommends values of 1.35 for a point load at midspan and 1.13 for a uniform distributed load. The variable $B$ is dependent on the type of loading and the warping stiffness of the cross section. For the two basic load cases of a point load at midspan, and a uniform distributed load, the SSRC [2007] Proceedings guide uses the following two expressions for the variable $B$.

Point load at midspan: \[ B = 1 - 0.180W^2 + 0.649W \]

Uniform distributed Load: \[ B = 1 - 0.154W^2 + 0.535W \]

The coefficient $W$ is sometimes referred to as beam parameter and is given by equation (2.7).

\[ W = \pi \sqrt{\frac{EC_w}{L(GJ)}} \]  \hspace{1cm} (2.7)

Where:

$L = \text{Distance between discrete braces}$

$E = \text{Modulus of elasticity}$

$C_w = \text{Warping coefficient}$

$G = \text{Shear modulus, and}$

$J = \text{St. Venant torsional constant}$

The SSRC [2007] proceeding guide provisions for load height provide a relatively accurate method for calculating the buckling load when transverse loading is applied at the top or bottom flange of doubly-symmetric sections.

A desirable solution for approximating load height effects in singly-symmetric girders would be a method similar to the equations for doubly symmetric sections. It is possible to check the accuracy of these equations on the singly-symmetric section. The
method uses equation (2.7) to calculate the beam parameter $W$. For singly symmetric sections, the warping term $C_w$ is defined by the equation (2.8).

\[ C_w = I_y d^2 \rho (1 - \rho) \]  

(2.8)

The variable $\rho$ is equal to $I_{yc}/I_y$, where $I_{yc}$ is the moment of inertia of the compression flange about an axis through the web, and $I_y$ is the weak axis moment of inertia. For a doubly-symmetric section $\rho = 0.5$, which results in a simpler expression for $C_w = I_y d^2 / 4$.

### 2.2 Metal Deck Form Building Application

In the building industry, for the bracing application, deck forms have traditionally been modeled as a shear diaphragm that restrains the lateral movement of the top flange of the beams to which they are attached. This diaphragm action provides the planar system with a definite capacity to resist in-plane deformations caused by the lateral loads. Previous studies have shown that permanent metal deck forms have a substantial amount of stiffness and strength in the plane of sheeting, which tends to provide significant bracing to the beams or girders to which they are attached. Current AASHTO specifications require bracing in the form of cross frames or other discrete diaphragms at a maximum spacing of 25 feet. The maximum spacing has been relaxed in the new AASHTO Specifications.

Although the building industry has long relied upon the in-plane capacity of light metal sheeting, currently AASHTO does not permit PMDF (Permanent Metal Deck Forms) to be utilized for bracing in steel bridge girders, mainly due to the flexible connection details between the girders and deck forms. There are several differences, however, in the type of metal deck forms that are used in the building industry versus the
type used in the bridge industry. Some of these differences are subtle while others are more significant. The subtle differences between PMDF used in the building and bridge industries can be observed in the type and depth of profile for the deck, as well as the thickness of the sheet metal used to make the form. The PMDF used in the building industry uses a smaller deck profile and thickness of sheet metal as compared to the PMDF used in the bridge industry.

Other significant differences between the metal deck forms used in the building industry versus the bridge industry include the span and shape of the deck panel, as well as the method by which it is attached to the girders. In the building industry, the forms are typically attached directly to the top flange of the beam by welding shear studs directly through the forms or by using puddle welds or mechanical fasteners.

This allows the use of a metal deck fabricated in long lengths that spans over number of beams. Figure 2.1 shows a typical PMDF panel arrangement for building application in which the forms are continuous over the tops of the forms. In buildings, a
deck panel is delineated by the parallel and perpendicular members to which the sheeting is attached. Because the metal is often fastened around the perimeter of the panels, the fastener forces are well distributed.

2.3 Potential Bridge Application

The first and major difference between the building and bridge industries is in the arrangement of connection details. First of all, some applications in the building industry may have a deck panel supported on all four sides; there is only one arrangement of deck sheeting possible for bridge deck construction. Instead of running continuously over the top of the girders, the steel deck must span between the bridge girders. It should be noted that deck forms are fastened to the bridge girders only at the ends of individual deck sheets as, there are no intermediate members between the girders. Because of this simple span arrangement, the only fasteners needed for the installation of bridge deck forms are deck sheet-supporting member fasteners at the deck sheet ends and sheet-to-sheet fasteners at individual deck sheet seams.

Figure 2.2. Metal deck form bridge application
Secondly, the attachment of the deck panels to the bridge girders by welding mechanical shear connectors through the deck is not permitted. Attachment of the deck panel to the supporting member is usually accomplished through the use of self-tapping TEK screws whose strength will often control the capacity of the diaphragm system. As a result of the spanning between adjacent girders, the corrugated ends of each deck sheet is closed, which provides a seal for the concrete, hence the closed ends of corrugations tend to stiffen the forms and individual sheets become stiffer compared to the building forms, in which the stiffness can be reduced due to the warping deformation of corrugations. However, although the bridge forms may be very stiff, the overall system stiffness of the formwork used in the bridge industry is usually substantially lower than similar systems, in the building industry. The larger difference in the system stiffness is due to the connection details that are utilized in bridge industry.

Metal deck forms used in bridge application are typically supported on a cold-formed support angle (Figure 2.3). The support angles allow the contractor to adjust the form elevation to account for differential camber between adjacent girders and changes in the flange thickness along the girder length. Although the adjustable support angle provides convenience with respect to constructability issues, the eccentricity produced by this connection can substantially reduce the stiffness and strength of the deck form system. AASHTO do not currently allow permanent metal deck forms to be considered for bracing in steel bridge girders, mainly due to the softening effect of the eccentric support angles. However, due to the scale of the members involved, design and construction engineers have an increased sense of awareness to compression flange stability prior to deck cure. Previous studies have shown that the girder/metal deck form
system may possess substantial in-plane shear stiffness and strength, which could be used to brace bridge girders during construction. Steel plate girder bridges are viable structural solutions to spanning long distances; therefore, in recent years, the use of long-span steel bridge girders increased due to the following reasons:

1. Developments in fabrication capabilities in the United States of America.
2. Economics of bridge construction.
3. Desired structural redundancy.
4. Improvements in construction methods.
5. Awareness of bridge aesthetics for long-span structures.

Long-span steel bridges are prone to failure during construction because of the following:

1. The overall stiffness of the structure during construction is significantly less compared to the bridge in service conditions.
2. Accurate understanding the behavior of such bridges under wind loads is critical.
3. Wind load effects complicate not only design but also erection, which often overlooked by contractors.

4. It is important to investigate how the various parameters influence the bracing/stability during construction.

Figure 2.4. Short-span shallow and long-span deep bridge plate girder assembly

Issues regarding long-span deep girders:

Upon the complete erection of all long-span steel girders, compression flange lateral stability should be checked because of the following reasons.

1. Increased web depths.
2. Diaphragm configurations.
3. Undesirable lateral compression flange deflections.
5. Vibration of girders during high wind.
2.4 Beam Bracing

Most structural forms include members such as beams for which elastic lateral torsional buckling is a possible mode of failure. The determination of the load level, which would cause such a failure, is a problem for the designer because it is one of the limits to overall load capacity. An estimate of this load must be made by determining an effective length, or in some other rational manner. Bracing members are deliberately placed to provide support against buckling. These members are usually assumed to be elastic, in which case they may be characterized by their elastic stiffness.

The bracing effectiveness is determined by its ability to prevent a twist of the cross-section. For this reason a brace should be placed at a point where it will counteract the twisting of the cross-section. To be effective in preventing twist, bracing must provide adequate stiffness and strength. Designing a brace to support some percentage (say 2%) of the compressive bending force in the beam usually provides sufficient strength in the brace, but it does not guarantee that the brace will provide sufficient stiffness to raise the buckling load of the critical member to the desired level Helwig [1994].

Theoretically the brace forces will be infinity when the buckling load is reached if the ideal brace stiffness is used. The ideal stiffness is defined as the stiffness required to force the member to buckle between the brace points. A brace system will not be satisfactory if the theoretical ideal stiffness is provided because the brace forces get too large. If the brace stiffness is overdesigned, then the brace forces will be more reasonable. The brace strength requirement is measured in terms of the force exerted on the brace by the members. Previous studies show that the stiffer brace would reduce the
brace strength requirement. There are a number of factors that affect the stability brace forces, including the shape and magnitude of the imperfection, the distribution of the imperfection along the length, and the value of the moment at the location of the brace. To develop suitable bracing design provisions, it is necessary to determine the maximum brace forces that are likely to occur in typical applications.

In general, beam bracing can be done in a variety of ways to increase beam buckling strength. Beam braces can be provided continuously along its length, as in the case of metal deck forms or braces can be placed at discrete intervals as in case of cross-frames.

Lateral bracing restrains lateral displacement of the top flange, and the lateral brace effectiveness is directly proportional to the degree that a twist of the cross-section is restrained. A lateral brace is most efficient in restricting twist when it is located at the top or compression flange. For the uniform moment, as the center of twist is located at a point near or outside of the tension flange, lateral bracing applied at the bottom flange of a simply supported beam is almost totally ineffective.

A torsional brace can be differentiated from a lateral brace in that the twist of the cross-section is restrained directly, as in the case of cross-frames or diaphragms located between adjacent members. Twist can also be restrained by cross frames that prevent the relative movement of the top and bottom flanges. Although bracing the girders with the help of cross-frames or diaphragms proved effective, it dramatically complicates girder fabrication and erection, increases construction and inspection costs and, most importantly, causes long-term fatigue problems. Therefore, alternative bracing systems and engineering methodologies for short-span bridge girders have been investigated by
others. Previous studies shown that permanent metal deck forms have a substantial amount of stiffness and strength in the plane of sheeting, which tends to provide significant bracing to the short-span shallow girders to which they are attached. Additional research may be required to demonstrate the bracing behavior of metal deck forms for long-span, deep-plate girder bridges.

In this study, the capability of metal deck forms to brace long-span, deep-bridge girders during construction is checked. Initially, work done on shallow, small-span bridge girders by prior researchers is presented. Design equations proposed by others are applied to long-span, deep-bridge plate girder stability by incorporating bracing contributions from metal deck forms. FEM is also carried to check the applicability of these design equations to long-span, deep-plate girder bridges.

2.5 Overall PMDF System Overview

2.5.1 Forming System

Developments in bridge construction techniques in recent years have led to several innovations. One of the innovative areas is forming systems, which are used to support wet concrete during the placement of the concrete deck. Traditionally, due to relative ease material availability, plywood forms and concrete panels were the first choice to support wet concrete during construction. However, these forms have major drawbacks, as panels have limited spans (maximum 8ft) which may reduce girder spacing and increase the number of girders required which affect bridge economy, and the task of removing the temporary forms is difficult since contractor must go under the bridge to remove the forms. In areas where form removal is expensive or hazardous, the use of
permanent deck forms may be desirable. Several permanent deck form systems have been developed which eliminate the need for temporary form removal.

One of the permanent deck form systems which has been used in the last few decades is precast concrete panels. Although these panels are economical, there are a few major drawbacks, as these forms are very heavy and the placement of concrete panels is labor intensive, usually requiring an external crane or trolley. Another major drawback is that the camber between adjacent girders differs significantly. Since the concrete panels rest on the flanges of the girders, the differential camber in adjacent girders must be accounted for by pouring a large volume of concrete over the lower girder.

Another permanent deck form system is permanent metal deck forms, which will be referred to as either PMDF or metal deck forms. Figure 2.5 shows the profile of conventional metal deck forms with open corrugations as well as a plan view of the forming system. The metal deck form system consists of corrugated steel sheets, which usually range from 24 to 36 in wide. Adjacent sheets are usually fastened together with self-tapping screws along the seams. These self-tapping screws are also used to fasten the deck down at the ends. One attractive feature of this type of forming system is the connection assembly to attach the metal deck forms to girders.

The main advantage of PMDF system over other forming systems is the much longer deck span, which allows it to cover more girders along the width of the bridge. Typical spans of PMDF are between 9–12 feet; however, some configurations of heavy gage deck can span 15 feet. In this connection assembly, metal deck forms are supported on a cold-formed angle that allows the contractor to adjust the form elevation at the ends to account for differential camber between adjacent girders. Uniform deck thickness can
be achieved by adjusting the form elevation and eliminating the requirement of larger volumes of concrete over any one girder.

Figure 2.5. Metal deck form plan view and cross-section view

One of the downsides to conventional permanent metal deck forms is that additional concrete is required to fill the corrugations. To avoid the cost of extra concrete, contractors often fill the corrugations with Styrofoam or another cheap filling material.

An alternative to using a metal deck form with open corrugation is to use a cellular deck with closed corrugation. Cellular decks overlap the ribs of the adjacent sheet to create a flat form surface, thus eliminating open corrugation. Metal deck forms are
usually more expensive than either plywood forms or precast concrete panels; however, the higher cost is often offset by the advantages mentioned. Additional economy and better service behavior from the bridge may also be possible if the PMDF are considered as a bracing element against lateral torsional buckling.

2.5.2 Deck Attachments

There are several methods of fastening PMDF to their supporting girders. Two are used most commonly in the bridge industry today.

2.5.2.1 Welded Connection Details

The first configuration, shown in Figure 2.6, is used when welding to the girder flange is permitted and consists of welding supporting angles directly to the top of the girder top flange. Once the support tangles are welded to the girders, the deck panels can be fastened with end fasteners to the angles.

Figure 2.6. Typical welded connection detail in bridge industry

In many practical cases, welding directly to the top flange is not permitted because of the potential fatigue problems in tensile stress regions. It is required that a
minimum distance of 1/2" be maintained between the end fastener centerline and both the
deck end and the angle edge.

2.5.2.2 Strap Connection Details

When the welding of girder is not allowed, a more complicated method of deck
support angle attachment is used (Figure 2.7). This method usually consists of welding
deck support angles to loose strap angles, which are typically spaced at approximately
one foot on center along the girder span. These strap angles are not welded to the girder;
however, hold-down clips are used to prevent any uplift of the deck panels. The deck
panels are then fastened to deck support angles.

![Figure 2.7. Typical strap angle connection detail in bridge industry](image)

It should be noted that both methods of deck support can introduce an eccentricity in the
transfer of the lateral deck panel load to the top flange of the bridge girder. Because of
the eccentricity, the flexibility of the deck support angle may substantially affect the
overall stiffness of the girder/deck panel system.
2.5.3 Support Angle Configuration

Adequate bracing must possess sufficient strength and stiffness to design loads and control deformations. In the building industry, metal deck forms in the roof or flooring systems are often assumed to act similarly to short, deep beams that help to resist lateral deformations from wind loads and are routinely relied upon to provide stability bracing to beams or columns. In these building applications, the forms are typically attached directly to the top flange by welding shear studs to the forms, or by using puddle welds or mechanical fasteners.

The formwork connection differs significantly in the bridge industry. Instead of being continuous over the top of the beam or the girders, the deck form sheets are fastened to cold-formed angles (support angles) that are attached to the girder as shown in Figure 2.6 and 2.7. The forms are typically fastened to the support angle and adjacent sheets using TEKS screws. The support angles allow the contractor to adjust the form elevation to account for the differential camber between adjacent girders or changes in flange thickness along the girder length. To facilitate proper erection of bridge deck forms, this elevation adjustment capability is very desirable. Although the adjustable support angle connection provides convenience with respect to constructability issues, the eccentricity produced by the connection can substantially reduce the stiffness and strength of the deck form system. Figure 2.8 shows the typical support details of the deck/girder system in the bridge industry.

In shear tests performed at the University of Houston, PMDF showed a tendency to produce fields of tension and compression within the panel system shown in Figure 2.8. This causes the support angle to pull away from the tension flange and push the angle
under the compression flange. The effective angle eccentricity in the region subjected to compression is therefore decreased by an amount equal to the thickness of the flange. The connection stiffness is therefore higher than the corresponding connection in the tension region [Jetann et al. 2002]. Due to the eccentricity that can lead to severe deformation of the support angle, the shear stiffness of metal deck forms reduces substantially. The equation for springs in series can be used as an analytical basis for the reduction in shear stiffness:

\[
\frac{1}{\beta_{sys}} = \frac{1}{\beta_{deck}} + \frac{1}{\beta_{conn}} \tag{2.9}
\]

In order to determine the shear rigidity of the connection, the values of the normalized connection shear rigidities should be divided by half span of the deck then shear rigidity

Figure 2.8. Fields of tension and compression within the panel system
of the system, $Q_{sys}$, can be calculated using:

$$\frac{1}{Q_{sys}} = \frac{1}{Q_{deck}} + \frac{1}{Q_{conn}}$$  \hspace{1cm} (2.10)

Where:

$Q_{deck}$ = Shear rigidity of the deck

$Q_{conn}$ = Connection shear rigidity

It should be noted that $Q_{sys}$ must be less than or equal to the smallest of either $Q_{deck}$ or $Q_{conn}$.

2.5.4 Mechanical Fasteners Assembly

The in-plane shear behavior of deck diaphragms, i.e., the ultimate capacity and the stiffness, is based on connection, deck shear, and deck warping characteristics [Currah 1993]. In almost all design situations, the connector capacity controls the strength of the diaphragm. Similarly, connection performance has a significant effect on the overall diaphragm stiffness, although the shear stiffness and warping stiffness of the deck are also influential. The commonly used SDI (Steel Deck Institute) [1995] Diaphragm Manual design method is based on the assumption that shear stresses across the width of a single panel are linearly related, where both side lap and deck-to-frame connections at the edges of the panel provide the greatest proportion of the shear resistance in a diaphragm system.

Fasteners required in the erection of permanent metal deck forms consist of end fasteners, which fasten the deck sheets to the girders, and seam fasteners, referred to as side lap fasteners, which connect individual sheets together at sheet overlaps. End fasteners, which connect light-gage deck sheets to the heavier support members attached
to the girders, customarily consist of arc spot welds, self-drilling TEK screws, self-tapping screws or powder actuated pin fasteners. Side-lap fasteners connecting individual light-gage deck sheets at their seams include arc spot welds, self-drilling TEK screws, or button-punched material.

Past studies have shown that fastener stiffness generally does not have much effect on the overall stiffness. Using rigid fasteners increased the system stiffness by 5-13% for different gages Egilmez et al. [2005]. Generally maximum fastener forces occur at edges due to the rotation of the support angle. Egilmez et al. [2005] revealed that using deck systems with stiffening angles provides uniformity between fastener forces, and the magnitude of fastener forces becomes approximately half the corresponding values for the unstiffened PMDF systems.

Presently self-drilling TEK screws are the dominant method of attachment of bridge deck forms for both end and side-lap fastening. The equation (2.11 and 2.12) shows the flexibility of the TEK screws from the Steel Diaphragm Design Manual [SDI 1995] for the deck to support angle flexibility, $S_f$, deck to deck (side-lap) flexibility, $S_s$, respectively:

$$S_f = 0.0013/t^{0.5} \text{ (in/kip)}$$  \hspace{1cm} (2.11)

$$S_s = 0.003/t^{0.5} \text{ (in/kip)}$$  \hspace{1cm} (2.12)

The background and importance of each component of bridge superstructure/substructure is explained. Research work done on shallow, small-span bridge girders by other researchers is presented in chapter Three of this report. Then design methodology proposed by others is applied to long-span, deep plate girder stability by incorporating bracing contributions from metal deck forms.
3.1 Applications of PMDF system

Engineers have been aware for many years of the high in-plane stiffness of profiled sheeting or decking, generally used in building and bridge industry to support wet concrete during construction. A considerable amount of work on shear diaphragm was done during the 1960’s and 1970’s.

3.1.1 Buildings

Nilson et al. [1960] laid down the foundation of the work considering the effects of end closures and marginal beams, and observed that the stiffness of the diaphragm increases with the span and depth of the panel profile.

Nilson’s work has been extended by Lutrell and Apparao [1967], who investigated the effect of panel configuration, material properties, and span length and particularly the method of fastening the diaphragm. Lutrell and Apparao [1967] developed a semi-empirical formula for estimating the shear stiffness of standard corrugated panels and noted that shear stiffness was mainly dependent on the length of the diaphragm and the type and spacing of the fasteners.

Bryan [1973] developed a simplified approach for analyzing the resistance
provided by building diaphragms. He derived simple expressions for the strength and stiffness of rectangular shape diaphragms. In his approach, the flexibility of the diaphragm is estimated as the sum of various components, i.e., distortion of corrugated sheets, shear strain in the sheeting, movement in the sheet purlin fasteners, movement in seam fasteners, movement in the shear connector fasteners, and axial strain in the purlins.

Nilson et al. [1974] used the finite element model of shear diaphragms to calculate the effective shear modulus and strength, and compared the results to the experimental data. Plane stress finite elements were used for the panels, line elements for purlins, and linkage elements for connectors. He recommended limiting the analysis to the elastic range and pointed out that the elastic response is limited to 40% of the failure load and that the connectors are the main source of nonlinearity.

Davies [1976] applied Bryan’s approach by addressing changes in component flexibility with internal force distribution. The method proposed was evaluated for three shear panels through finite element and experimental work. Davies [1977] later extended his approach to model actual light-weight diaphragms. He considered the different modes of failure and assumed a suitable distribution of internal forces within the diaphragm in order to obtain a simplified model. The model simulates the diaphragm as a frame consisting of prismatic elements that could be solved with computer capabilities available in the 1970. The results were verified by comparing them to detailed FEM analysis, and it was shown that the method can be extended to elastic–plastic behavior.

Errera et al. [1976] presented a procedure for the design of I-section beams with diaphragm bracing. The procedure uses the shear strength and shear rigidity of the diaphragms to estimate the ultimate load capacity of the fully braced beam. It was
demonstrated that the lateral torsional buckling moment of the diaphragm-braced beam is conservatively estimated as twice the product of the shear rigidity of the diaphragm, the distance between the center of gravity of the member and the plane of the diaphragm.

### 3.1.2 Bridges

Texas researchers have been involved in a comprehensive research program that includes experimental and analytical studies of the bracing of bridge girders using permanent metal deck forms during construction.

Currah [1993] and Soderberg [1994] investigated the bracing ability of permanent metal deck forms acting as shear diaphragms. Currah [1993] indicated that the shear stiffness of permanent metal deck forms is dependent upon material strength, modulus of elasticity, deck thickness, deck profile, pitch of deck corrugations, deck panel span, presence of end closures, number of end fasteners, number of seam fasteners, and flexibility of the permanent supporting members.

The primary objective of Currah [1993] study is to determine the shear stiffness of permanent metal deck panels without any effects from the supports used to attach the forms to the girders. Currah [1993] also investigated the potentially mitigating effect of the permanent metal deck panel supporting members on the shear strength of the diaphragm system. The permanent metal deck forms used in his study were supported by thin angles that are either welded to the top flanges of the girders or the use of connecting strap angles that saddle the top flanges. Currah [1993] noted that both connection details can introduce an eccentricity in the transfer of the loads from the permanent metal deck forms to the top girder flanges.

Currah [1993] concluded that the flexibility of the supporting angles should be
carefully considered if the permanent decking is to be considered as a lateral bracing system. Currah [1993] noted that the connection stiffness was a controlling factor in the stiffness of the metal deck form diaphragm system. Some of the diaphragm system stiffnesses were reduced by more than 80% when using the typical eccentric support angle instead of a rigid connection. Currah [1993] explained that shear strength is controlled by a combination of one or more failure modes. Currah [1993] also used the SDI design manual to evaluate the shear strength and shear stiffness of bridge permanent metal deck forms and compare them to experimental values. The SDI design method was modified to account for the differences found in bridge applications of permanent metal deck panel forms.

The work by Currah [1993] revealed that the decking had significant stiffness and that the connection detail controlled the stiffness of the diaphragm system. Soderberg [1994] continued the work of Currah [1993] by further investigating the connection stiffness of permanent metal deck forms and also determined ways to improve the connection. He proposed a modified strap connection detail that showed improved connection stiffness throughout the testing but required significant fabrication and placement efforts. Throughout the testing, various modifications were made to the diaphragm system to determine their effects upon the system response to lateral and vertical loads. The responses of the system were compared to analytical results to determine the diaphragm stiffness.

Soderberg [1994] performed three different experimental sets of tests. The first set of tests were conducted to measure the in-plane (transverse to the girder) stiffness of various connection details and to propose an improved strap connection detail. The
second set of tests, using a shear frame test set-up that was constructed and used by Currah [1993], was conducted to determine the diaphragm shear stiffness improvements provided by improved connection detail. Also, strength and ductility issues were addressed concerning the improved strap detail. The final scale “Twin Girder” tests were conducted to determine the effect of the improved connection detail on the diaphragm stiffness and buckling load of the girder system. Buckling results from these tests were compared with the recommended bracing design method developed in the analytical study by Helwig [1994]. It was found that the bracing provided by the deck form diaphragm system is significant, and the estimated buckling capacity of the twin girder system tested based on Helwig’s [1994] bracing formula and recommended diaphragm stiffness is reasonable.

Helwig [1994] studied the lateral bracing ability of permanent metal deck forms commonly used in steel bridge construction. He stated that, prior to deck placement, the steel must support all construction loads until composite behavior is developed. Therefore, lateral torsional buckling of the steel plate girders is critical during the non-composite stage of construction. Helwig [1994] stated that permanent metal deck forms provide continuous bracing against lateral movement along the girder, thus behaving as a shear diaphragm. He performed rigorous FEM analysis on twin-girder systems with a shear diaphragm at the top flange. These analyses were used to determine the effect of the deck shear rigidity on the buckling capacity of a twin-girder system. The FEM analysis results were compared to existing solutions for beams braced by shear diaphragms. These solutions were used to develop a design approach for single-span and continuous girders braced by the permanent metal deck forms. It was found that this design approach
reduces the number of cross-frames required to laterally brace the girders.

Helwig and Frank [1999] used FEM methodology to analyze singly symmetric I-beams subjected to transverse loading applied at different heights. The results from analytical studies were presented for different cross-sections under single point and uniform transverse loads. The goal of their study was to introduce a solution for the general loading case that is compatible with lateral torsional buckling solutions. They concluded that the height of the transverse load has a significant effect on the buckling capacity and proposed a modification factor for $C_b$ equation to account for the effect of the reverse-curvature bending for singly symmetric sections.

Helwig and Frank [1999] also presented the results from an analytical study that looked at diaphragm stiffness, load type, load position, cross-sectional shape, and web slenderness. They pointed out that the variable used to determine the contribution of the diaphragm was defined differently in two research studies: Lawson and Nethercot [1985] defined “$e$” as the distance between the plane of the decking and shear centre of the beam, while Errera and Apparao [1976] defined it as the distance from the plane of the decking to the centre of the gravity of the beam.

Texas researchers continued the work done by Helwig [1994] of studying the lateral bracing ability of permanent metal deck forms in steel plate girder bridges. Researchers were particularly focused on improving the connection detail between the top girder flanges with the permanent metal deck forms and proposed improved connection details which involve a transverse stiffening angle that spans between adjacent girders to control the support angle deformation. The effect of different parameters, such as the metal gage of the deck forms, the span of the forms,
connection detail, and panel aspect ratio, was investigated.

It was found that the failure of deck panels with maximum eccentricity is due to the severe deformation of the support angles at the corners of the panel. To control this angle deformation, a transverse stiffening angle was placed to coincide with a side-lap seam so the deck could be fastened directly to the angle, which spans between adjacent girders. While the main purpose of the stiffening angle is to control deformation of the support angles, they also provide the support to decks at the end of the panel. The PMDF/stiffening angle assembly is shown in Figure 3.1.

![Girder/metal deck form connection with stiffening angle](image)

**Figure 3.1.** Girder/metal deck form connection with stiffening angle

To study the effect of eccentricity on the strength and stiffness of the PMDF system, tests were conducted with maximum and zero support angle eccentricity on stiffened and unstiffened connection details. The twin-girder system was used to perform
lateral stiffness tests and buckling tests. Finally, the results from laboratory tests were compared with FEM analysis results, and the comparison revealed that simple modifications to the connection can greatly improve the shear strength and stiffness of the permanent metal deck form system. After comparing the laboratory and FEA values of effective shear modulus $G'_{exp}$ for eccentric unstiffened and stiffened connections, it was concluded that providing stiffening angles to this system increases the stiffness by more than a factor of four.

Extensive research has been done on the use of metal deck forms for stability of bridge girders during construction. These studies have demonstrated the stability advantage provided by metal deck forms during the construction of span lengths typical of highway overpass bridges, but additional work may be needed to determine whether this method can be used for very long, deep-bridge girders. This first phase of the project largely leverages recent stability research conducted by others, but with interest towards application to very long, deep bridge girders. Next phases of the work focus on the use of FEM to validate application to very long, deep-bridge girders, and also, due to vibration phenomena encountered by the Alabama Department of Transportation, consider the influence of PMDF on the vibration characteristics of bridge superstructures.

### 3.2 Permanent Metal Deck Form Characteristics

Permanent metal deck forms have been treated as shear diaphragms in building and bridge industry. Before making use of shear diaphragms in structural design, it is necessary for a designer to have knowledge of stiffness and ultimate strength. These quantities can be estimated using testing, approximate method, and FEM analysis.
Considerable progress has been made recently in developing methods to predict the two important parameters that characterize a diaphragm assembly:

1. Shear stiffness.
2. Shear strength.

Tabulated values for specific connection assemblies are given in some of the references listed in the reference section and in the proprietary literature of deck panel manufacturer. As an alternative, these characteristics can be determined from the load deflection curve obtained from a simple beam or cantilever shear test. If the shear stiffness of a diaphragm is known, then the maximum shear strain that can be sustained by a diaphragm is a measure of its shear strength.

3.2.1 Shear Stiffness of PMDF

The diaphragm plays an important role in the overall behavior of structure, so it is important to have a clear understanding and knowledge of in-plane shear strength, shear stiffness, and the reliability of the system. As diaphragm strength is important, diaphragm stiffness is also a major consideration, because deflection compatibility must be maintained between the structural framing and the diaphragm. The total system reliability is primarily dependent on connections between panels along their edges and those from panels to supporting structural members. Panel shape, dimensions of the structure, material thicknesses, panel shape, and type of connections are factors affecting the system’s diaphragm shear strength and stiffness.

The diaphragm shear stiffness is important in assessing how forces are transferred through deck panels from one bridge girder to the other. This force transfer is important
to the stability of the bridge/girder system. Diaphragm shear stiffness can be defined as the ratio of average applied shear stress divided by the diaphragm’s shear strain.

When girders are braced by shear diaphragm, the parameter which is of most importance is shear rigidity, \( Q \), that has units of force per unit radian (KN/rad or kip/rad). The shear rigidity is calculated as the product of effective shear modulus \( (G') \) and tributary width of deck \( (S_d) \). The effective shear modulus can be calculated using the equation (3.1).

\[
G' = \frac{\tau'}{\gamma}
\]  
(3.1)

Where:

\( G' \) = Effective shear modulus

\( \tau' \) = Effective shear stress

\( \gamma \) = Shear strain

\( P \) = Load applied at the end of the testing frame

\( a \) = Panel width

\( b \) = Width of the deck panel

\( \Delta \) = Deflection at the tip of the testing frame

The tributary width of the deck \( (S_d) \) is the effective width of the deck bracing a single girder. In a bridge with \( n \) girders, \((n-1)\) metal deck forms would typically be used.

\[
S_d = (S_g - b_f)(n-1)/n
\]  
(3.2)

In the equation (3.2), \( n \) = number of girders in the system, \( S_g \) = spacing between girders, \( b_f \) = width of girder top flange.
Then shear rigidity of permanent metal deck form can be calculated using:

\[ Q = G' \cdot S_d \] \hspace{1cm} (3.3)

The building industry uses design tables and laboratory testing results to evaluate the effective shear modulus \((G')\). A shear test on the diaphragm can be performed to find the effective shear modulus. For design purposes, it is not practical to perform testing of a particular deck to measure the effective shear modulus. The SDI [1995] provides a series of equations that can be used to calculate the effective shear modulus \((G')\) for a given metal form. According to Currah [1993], if warping deformation in the corrugation are neglected SDI [1995], expressions provide a reasonable estimate of the effective shear modulus with laboratory test results for bridge deck forms.
Conventionally, shear modulus is defined as shear stress divided by shear strain; however, since the shear stress versus strain relationship of corrugated sheeting is generally not a linear function of material thickness SDI [1995], an effective shear stress should be utilized that is not dependent on metal thickness. Previous studies showed that following are the major factors that can affect diaphragm shear stiffness [Currah 1993]:

- Decks with closed ends possess substantially more shear stiffness due to more resistance to distortion of the deck form sheeting profile than do open-ended decks.
- The number of end fasteners connecting the deck panel to supporting member’s in every trough exhibits greater stiffness than panels in every other trough.
- There will be an increase in stiffness if we provide additional number of seam fasteners attaching adjacent deck sheets together in deck panel.
- Supporting angle flexibility.

3.2.2 Shear Strength of PMDF

The shear strength of a deck diaphragm can be determined experimentally by testing a deck panel assembly the same as the one shown in Figure 3.2. When the deck system reaches the ultimate capacity, the applied load to the deck panel becomes its maximum sustained value, $P_{ult}$. For this study, the diaphragm shear strength will be defined as the ultimate load the deck panel can sustain. The ultimate effective shear capacity of the diaphragm is computed as follows:

$$S'_{ult} = (P_{ult}L)/fw$$ (3.4)
As the bridge decks are fastened at the ends of the deck sheets to supporting members. This type of fastening leads to much larger forces in the end fasteners. The fasteners parallel to the deck span generate much larger forces than fastener forces generated perpendicular to span. These large-end fastener forces parallel to the span of the deck will generally control $P_{ult}$ and, consequently the shear strength of the deck panel.

The failure mode of PMDF systems is often characterized by the fracture of fasteners, bearing deformations on the deck at the fastener location or support angle deformation. For simplicity it can be assumed that bearing deformations occur along the side-lap seam and, when adjacent panels begin to separate, there is linear distribution of forces at the end fasteners along supporting angle.

Panel shape, dimensions of the structure, material thicknesses, panel shape, type of connections are factors affecting the system’s diaphragm shear strength and stiffness. The strength of a diaphragm is also governed by shear strength involving force transfer at the interior panel connection and the fastener across the ends of panels.

3.3 **Metal deck form contribution to brace bridge girders**

Cold-formed steel panels often are used as wall sheeting, roof decking, or floor decking in steel-framed buildings. These panels carry loads normal to their plane by virtue of their bending strength. In addition, adequately formed diaphragms connecting these panels can resist in-plane shear deformations. Because of this shear resistance, such diaphragms are used as wind bracing for buildings. Another use of this diaphragm action is as bracing against buckling for individual members of steel frames. If properly used,
they can eliminate the need for other types of bracing and thus contribute to economical design.

Figure 3.3 shows the possible modes of failure of beams with diaphragm bracing on the compression flanges. In Figure 3.3 (a), the diaphragm rigidity and strength are not adequate to prevent lateral buckling of the beams. In Figure 3.3 (b), the diaphragm is adequate, and the beams fail by yielding. Full bracing in this case is defined as that which has adequate rigidity and strength to prevent lateral buckling until the beam yields.

![Figure 3.3. Girder failure modes when braced with diaphragm on compression flanges](image)

Although current AASHTO provisions do not allow the use of permanent metal deck forms to be considered as a lateral bracing element for bridge girders during the design process, previous studies in the building industry have demonstrated that metal forming can significantly increase the buckling capacity of beams. There are numbers of previous research investigations on the bracing behavior of shear diaphragms. A closed form solution for beams braced by shear diaphragms resulted from these studies, and Errera [1976] presented the following energy-based expression for doubly symmetric
sections, which assumes that lateral displacement and twist of the cross-section were sine curve along the girder length.

\[
M_{cr} = \sqrt{\left(\frac{\pi^2 EI_y}{L^2} + Q\right)\left(\frac{\pi^2 EC_w}{L^2} + GJ + Qe^2\right)} + Qe
\]  (3.5)

Where:

- \(M_{cr}\) = Buckling moment of shear diaphragm braced girder
- \(E\) = Modulus of elasticity
- \(I_{yc}\) = Weak-axis moment of inertia
- \(L\) = Spacing between points of zero twist on the beam
- \(C_w\) = Warping coefficient of beam
- \(Q\) = Shear rigidity of diaphragm
- \(E\) = Distance from center of gravity of the girder to plane of the shear diaphragm

This equation proved very effective, and Helwig [1994] compared closed formed solution results with FEM results and showed that the difference is much less than 2%.

For doubly symmetric sections, the shear center and center of gravity are both located at mid-height of the cross-section. It is not clear what to use for \(e\) for singly symmetric sections for which the geometric and shear centers do not coincide. To account for the capacity of singly symmetric section, the Errera [1976] solution has been modified and is applicable to both doubly and singly symmetric sections, it is shown in equation (3.6).

\[
M_{cr} = \sqrt{(M_{AASHTO})^2 + \left[2Qe^2 \frac{\pi^2 E(2I_{yc})}{L_b^2} + QGJ + Q^2 e^2\right]} + Qe
\]  (3.6)
Where $M_{AASHTO}$ is given by the standard AASHTO specifications for estimating lateral torsional buckling capacity of singly and doubly symmetric girder cross-sections and can be calculated using the equation (3.7).

$$M_{AASHTO} = 91 \times 10^6 \left( \frac{I_{ye}}{L_b} \right) \sqrt{0.772 \left( \frac{J}{I_{ye}} \right) + 9.87 \left( \frac{d}{L_b} \right)^2}$$  \hspace{1cm} (3.7)

Where:

$J$ = Torsional constant

d = Depth of the girder

Helwig [1994] compared the finite element results with solutions presented by equation (3.7) and found that $e$ will provide best correlation with Timoshenko’s solution if it is taken as the distance from the plane of the deck to midheight of the girder i.e., 

$(e=d/2)$.

Errera and Apparao [1976] as well as Nethercot and Trahair [1975] have suggested that a simple approximation for the buckling capacity of the girder braced by a shear diaphragm on the top flange and uniform moment loading can be obtained with the equation (3.8).

$$M_{cr} = M_{AASHTO} + 2Qe$$ \hspace{1cm} (3.8)

Where, $M_{AASHTO}$ is the buckling capacity of the girder with no deck for bracing, while $Q$ and $e$ have been already defined. Therefore, pursuing the modified approximate solution form is much more desirable for design applications due to its simplicity. Another attractive feature of equation (3.8) is that it allows the designer to select a suitable solution for the girder buckling capacity ($M_{AASHTO}$). This is particularly attractive for singly-symmetric sections in which there are a variety of approximate solutions available.
These expressions are valid for girders subjected to a constant moment. In many cases, the buckling load for the cases with moment gradient are actually less than the buckling capacity for cases with uniform moment. For moment gradient cases, there is a noticeable reduction in the slope for increasing shear rigidity, which is different than constant moment cases where there is gradual reduction in the slope with an increase in shear rigidity. This reduction in slope for girder with moment gradient can be explained by the location of the center of twist. The center of the twist for girders subjected to moment gradient approaches the top flange, which would make the deck less effective as a bracing element. On the other hand, the center of the twist for girders subjected to the constant moment approaches the bottom flange, which results in a gradual reduction in slope.

Lawson and Nethercot [1985] applied traditional $C_b$ value on the entire Errera expression and presented the equation (3.9) for a beam braced by a shear diaphragm subjected to a moment gradient.

$$M_{cr} = C_b(M_{AASHTO} + 2Qe)$$  \hspace{1cm} (3.9)

Helwig [1994] found that applying $C_b$ to the entire modified solution is un-conservative and the equation (3.9) does not estimate the buckling capacity precisely or accurately due to drastic changes in the buckled shape that the deck causes. It was shown that using $C_b$ factor only on the girder capacity was the most logical design approach, and the equation to account for the moment gradient is presented in equation (3.10).

$$M_{cr} = C_bM_{AASHTO} + Qd$$  \hspace{1cm} (3.10)

Previous researchers have shown that there is a significant effect of transverse load height applied to the buckling capacity of the girder cross-section. Top flange
loading and long-span girders make the deck significantly less effective as a bracing element. To account for load height effects, Lawson and Nethercot [1985] presented the energy-based equation (3.11), assuming that the twist and lateral displacement of the beam follows the sine curve along the beam.

\[
M_{cr} = C_b d \left[ -\frac{P_e g}{2} + \frac{Q(1 - g)}{2} + \left( \frac{-\frac{P_e g}{2} + \frac{Q(1 - g)}{2}}{2} \right)^2 - \frac{Q^2}{4} + \left( \frac{P_e}{2} + \frac{Q}{2} \right) \left( \frac{P_e}{2} + 2P_T + \frac{Q}{2} \right) \right] \quad (3.11)
\]

Where:

\( d \) = Depth of the girders

\( P_e \) = Weak axis Euler load = \( \left( \frac{\pi^2 EI}{L^2} \right) \)

\( C_b \) = Moment gradient factor

\( G \) = Load height factor

The term \( P_T \) is given by \( (GJ/d^2) \)

\( G \) = Shear modulus of beam material

\( J \) = Torsional constant of the beam

The \( C_b \) factor outside the square bracket accounts for moment gradients, while factor \( g \) accounts for load height effects. Lawson and Nethercot [1985] have recommended using traditional \( C_b \) and \( g \) values. The AISC (American Institute of Steel Construction) specification uses the equation (3.12) for the \( C_b \) of a girder buckling between points of full bracing (cross-frames)

\[
C_b = \frac{12.5M_{\text{max}}}{2.5M_{\text{max}} + 3M_2 + 4M_{cl} + 3M_4} \quad (3.12)
\]
Where:

\[ M_{\text{max}} = \text{Maximum moment between full braces} \]

\[ M_2 = \text{Moment at the quarter point between full braces} \]

\[ M_{cl} = \text{Moment at the midway between full braces} \]

\[ M_4 = \text{Moment at the three quarters point between full braces} \]

For a point load at the mid-span top flange, the value of the \( g \) is 0.55, while the value of \( g \) for a distributed load is 0.45. As energy-based solution depend on the buckled shape assumed in the derivation, Helwig [1994] showed that the buckled shape for girders subjected to moment gradient is significantly different than the sine curve.

Helwig and Frank [1999] presented finite element results that demonstrate the effects of moment gradient and load height on the bracing behavior of shear diaphragms. To make these results applicable to general loading conditions, they proposed the equation (3.13) to approximate the ideal stiffness requirements.

\[
M_{cr} = C_b^* M_{\text{AASHTO}} + mQd
\]  

(3.13)

Where:

\[ C_b^* = \text{Moment gradient factor that considers load height effects} \]

\[ d = \text{Depth of the cross-section} \]

\[ M_{cr}, M_{\text{AASHTO}}, \text{ and } Q \text{ are as defined in equation (3.5 and 3.6)} \]

The term \( mQd \) in equation (3.13) represents the contribution from the PMDF/support angle connection. The component of the deck in equation (3.13) is a function of the girder depth and the deck shear rigidity as well as the constant \( m \) that depends on the type of loading, the presence of intermediate bracing, and the web slenderness.
Helwig and Frank [1999] demonstrated that $e = (d/2)$, for both singly and doubly-symmetric girders, is a good approximation. For the uniform moment, the value of $m = 1$ is used. In most practical applications, the transverse loading on the beams is applied at the top flange. For uniformly distributed loads applied at the top flange, $m$ can be obtained from Table 3.1. The references to torsional bracing in Table 3.1 apply to the presence of cross-frames or diaphragms. The values for $m$ are also applicable for concentrated loads applied at the top flange; however, if the load point is also a braced point (no twist), $m = 1.0$ may be used. Web buckling due to bending or shear stresses needs to be considered when bracing girders with slender webs. The buckling capacity of diaphragm-braced beams should be limited to the lowest value, based on the limited states of web bend buckling, shear buckling or lateral-torsional buckling given by equation (3.10). This expression can be rearranged to solve the ideal effective shear modulus in terms of the maximum moment, $M_{cr}$, and the buckling capacity of the girder without diaphragm bracing, $C_b^*M_{AASHTO}$, between points of zero twist:

$$G'_{ideal} = \left( M_{cr} - C_b^*M_{AASHTO} \right) / (s_d md) \left( \text{equation 3.14} \right)$$

The expressions presented so far represent the capacity for perfectly straight beams braced by a shear diaphragm. For a particular maximum moment, the diaphragm stiffness

<table>
<thead>
<tr>
<th>Web Slenderness</th>
<th>Top Flange Loading w/o Midspan Torsional Brace</th>
<th>Top Flange Loading with Midspan Torsional Brace Helwig and Frank [1999]</th>
<th>Helwig and Frank and Yura [2003]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h/t_w &lt; 60$</td>
<td>0.5</td>
<td>0.85</td>
<td></td>
</tr>
<tr>
<td>$h/t_w &gt; 60$</td>
<td>0.375</td>
<td>0.64</td>
<td></td>
</tr>
</tbody>
</table>
derived from these expressions would represent the ideal stiffness. Helwig and Frank [1999] conducted large displacement FEM analysis on girders with initial imperfections. They found that providing four times the ideal stiffness could effectively control deformations and brace forces. For design considerations, equation (3.14) becomes:

$$G'_{req,d} = 4(M_{cr} - C^b M_{AASHTO})/(s_d m_d)$$ (3.15)

The stiffness requirement for the shear diaphragm given is based on an analysis of beams with an initial twist, $\theta_0 = L/(500 d)$, where $d$ = section depth.

### 3.4 Transverse Stiffening Angle Contribution to the Bracing Behavior of PMDF System

The AASHTO specifications do not allow the use of permanent metal deck forms for bracing in steel girder bridges, mainly due to the softening effect of eccentric support angles; therefore, Egilmez [2007] continued the work of Helwig [1994] and tested a number of modified connection details to control the support angle deformation. However, the concept of the use of a transverse stiffening angle that spans between adjacent girder flanges proved most effective and practical. These stiffening angles were positioned to coincide with a sidelap seam so that the deck could be screwed directly to the angle with several fasteners. As a result, these systems provide more stability bracing than conventional diaphragms connected on two sides. The spacing between stiffening angles were kept between 8 ft and 16 ft.

Using this modification to connection detail, Egilmez [2007] performed all laboratory tests, and the results were validated using FEM technique. The results from laboratory and FEM presented in the next chapter will focus on the behavior of PMDF systems with stiffening angles. The comparison revealed that the stiffened deck provides
a substantial increase in the buckling capacity of girders. The lateral load tests done with the proposed connection detail by Egilmez [2007] showed that the stiffened connection system provided a larger lateral stiffness than the unstiffened connection system. Considering the cases with a lateral load at mid-span for the 20-gage deck, the difference in the lateral stiffness between the unstiffened and stiffened systems ranged between 2 k/in and approximately 5.3 k/in. This increment in the lateral stiffness due to the stiffening angles can provide a significant increase in the buckling capacity; however, accounting for the bracing is somewhat difficult. For many problems, this amount of bracing acting alone may be adequate to substantially reduce the unbraced length of the beams.

To consider contribution made by a stiffening angle, Egilmez [2007] modified the expression provided by Helwig [1994]. Equation (3.14) was developed for girders braced by a shear diaphragm that was fastened on only two sides (i.e., at the support angle). The stability bracing contributions from the stiffening angles provide a quite different type of bracing than the restraint coming from the PMDF connection through the support angle. Since these systems have panels that are connected on four sides, they tend to be more effective than a shear diaphragm supported on only two sides.

The PMDF with stiffened connections provides restraint of two points relative to one another, which is very similar to a relative bracing system. As a simple estimate, the bracing of the stiffening angles will be approximated by using 50% of the buckling moment computed by using $L_b/2$ to evaluate the buckling capacity, where $L_b$ is the spacing between the cross-frames. This approach should be conservative for most problems since the stiffening angles probably provide significantly higher bracing;
however, this provides a simple solution that can be graphed along with the FEM solutions. Therefore, equation (3.14) becomes:

\[ M_{cr} = \frac{C_s^* M_{AASHTOL} (L_s / 2)}{2} + mQd \]  

(3.16)

It should be noted that the effective shear stiffness of the decks used in the field will usually be greater than the smallest value obtained from laboratory test results. The tests in the laboratory used the largest possible eccentricity all along the girder length. In the field the eccentricities will often be smaller at several locations along the girder length.

3.5 Strength Requirement for Shear Diaphragm Bracing

The strength requirement for shear diaphragm bracing is a function of the span and depth of the beam. If a diaphragm with stiffness \( G'_{req,d} \) is provided, the required bracing moment \( M'_{br} \) per unit length of the beam can be approximated by Egilmez (2007) which is shown in equation (3.17).

\[ M'_{br} = k \left( \frac{M_{cr} L}{d^2} \right) \]  

(3.17)

Where:

\[ L = \text{Total beam span, } d = \text{Girder depth} \]

The brace moment represents the warping restraint provided to the top flange of the girder per unit length of the span and can be resolved into forces on the diaphragm. Brace moment expression can be used to determine the forces in the fasteners used to connect the shear diaphragm to the beams. However, in many instances, the stresses that result from the fastener forces predicted by expression (3.17) can be large, particularly
since the fasteners are relatively small. Although a shear diaphragm model predicts relatively large fastener forces, the magnitude of fastener forces in actual PMDF-braced systems are probably not this high, because deck contributions to bracing comes from both shear flexural behaviors. The connection forces between beams and the diaphragm must be obtained as the resultant of values $M_{br}$ and $V_{br}$.

The majority of recommendations in the literature concerning bracing by shear diaphragm have dealt with simply supported girders. When girders are continuous, both top and bottom flanges have regions subjected to compression so the buckling mode is more complex and can involve large lateral translations of both flanges. The shear diaphragm can brace the top flange; however, the diaphragm has no effect on the bottom flange. The only work that dealt with continuous type girders with top flange bracing was conducted by Yura [1995]. The expression (3.18) was presented by Yura [1995] for girders with a top flange braced continuously and transverse loading applied at the top flange.

$$C_b = 2.25 - \frac{1}{2} \left( \frac{M_1}{M_0} \right) + \frac{2M_{cl}}{(M_0 + M_1)}$$

(3.18)

It was also shown that, in many cases, girders braced by a shear diaphragm behave in a manner similar to girders, with the top flange fully restrained from lateral displacement along the girder. In these instances, the expression for reduced $C_b$ may be useful in predicting the buckling capacity of these girders.
3.6 Effect of Fastener Spacing, Panel Width, and Girder Spacing

Previous studies showed that the shear characteristic of the PMDF/diaphragm system depends upon the spacing and number of the side-lap and end fasteners. The shear stiffness of the PMDF system will substantially decrease if the number and spacing of end and side fasteners are absent. To design a permanent metal deck form system as a lateral brace, Currah [1993] recommended fastening the deck form panel ends in every rib trough and keeping side-lap fasteners as close as possible.

An experimental study was done by Currah [1993] to check the effect of deck panel width on the shear characteristics of PMDF/girder system. The strap and welded angle connection details were considered, and the comparison revealed that shear stiffness and strength increases as the deck panel width increases.

Egilmez and Helwig [2005] showed that the shear rigidity of the metal deck form system increases as girder spacing increases. For bridges with multiple girders, the shear rigidity for each girder will tend to go up, since there are more metal deck forms per girder that can provide bracing. However, the effect of girder spacing (deck span) on the contribution to bracing provided by the stiffening angles was not investigated.
CHAPTER 4
PROPOSED DESIGN METHODOLOGY

4.1 Overview

The previous chapters provide a thorough background and overview of experimental and analytical test results done by others. Computational and experimental studies that concentrated on the large displacement analyses were carried out by previous researchers to investigate the strength and stiffness requirements of the metal deck form system. The goal of these parametrical studies was to improve the understanding of the bracing behavior of the PMDF systems with stiffened and unstiffened connections. The general approach that was adopted in the parametric studies of these systems was to maintain a conservative model. In addition, expressions that estimated the stiffness and strength requirements for the PMDF systems are also conservative.

The primary objective of this chapter is to propose a design methodology that considers a permanent metal deck form as a bracing element to stabilize long deep-bridge girders against lateral loading/wind and possible loading during construction. In the appendix, design calculations demonstrate the use of these modified expressions in actual existing bridge design examples. Although a girder/deck forms system with new, modified stiffened connection details showed impressive results, it was never the objective of this study to understand the effect of a stiffened connection details for long-
span deep plate girder bridges. For the purpose of understanding, design method considering stiffening angle contribution bracing is presented at the end.

4.2 Design Recommendations

4.2.1 Recommendations for Stiffness

Recommendations by Helwig [1994] and Egilmez [2007] have been presented for the required stiffness of the permanent metal deck form system to use as a lateral brace during construction. The modified equation (4.1) should be used to calculate the ideal deck stiffness for a given moment level with and without stiffening angle connection detail.

\[ G'_{\text{ideal}} = \frac{(M_{\text{cr}} - M_{\text{recommended}})}{s_d m_d} \]  

(4.1)

Expression for \( M_{\text{recommended}} \) can be taken as follows,

Case 1: Without Stiffening angle [Helwig 1994]

\[ M_{\text{recommended}} = C_b \frac{M_{\text{AASHTO}}}{L_b} \]

Case 2: With Stiffening angle [Egilmez 2007]

\[ M_{\text{recommended}} = \frac{1}{2} C_b \frac{M_{\text{AASHTO}}}{L_b / 2} \]

The modification consisted of using 50% of the buckling moment corresponding to \( L_b / 2 \), where \( L_b \) is the spacing between cross-frames. This solution gives conservative results relative to the FEM analysis results. Table 3.1 presents the recommended m values for stiffened-deck braced girders. With more than one intermediate cross-frame, the m-values provide conservative estimates of the buckling capacity. Where \( G'_{\text{ideal}} \) = ideal deck stiffness, \( M_{\text{cr}} \) = maximum design moment, \( S_d \) = tributary width of deck bracing a single
girder, $M_{AASHTO}$ is the buckling capacity of the girder using half the spacing between cross-frames, $C_b^*$, $m$ and $d$ have been defined previously. To control the deflections, the required deck shear stiffness $G'_{req'd}$ should be taken as Four times the ideal stiffness.

4.2.2 Recommendations for Strength

The magnitudes of brace moments tend to increase with the depth of the individual girder and $L/d$ ratio for specific girder depth. The recommended value of brace stiffness of $4Q_i$ was used to establish the strength requirements. The equation (4.2) is presented by Egilmez [2007] for estimating the required bracing moment per unit length of a girder for a diaphragm with stiffness is given by,

$$M'_{br} = k \left( \frac{M_{cr}L}{d^2} \right)$$  \hspace{1cm} (4.2)

Where, $L = \text{total beam span}$ and $d = \text{beam depth}$. The brace moment represents the warping restraint provided to the top flange of the girder per unit length of the span. The equation (4.2) can be used to determine the forces in the fasteners used to connect the metal deck form.

For unstiffened connections, permanent metal deck form bracing is supported only along two sides; therefore the recommended value for $k$ is 0.0011. For stiffened connections, equation (4.2) shows very conservative estimates. Based upon the large displacement solutions, the values of $k$ for the stiffened-deck braced girders can be chosen from Table (4.1).
### Table 4.1: Design k values

<table>
<thead>
<tr>
<th>Web Slenderness</th>
<th>Top Flange Loading w/o Midspan Torsional Brace</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h/t_w &lt; 60$</td>
<td>0.00015</td>
</tr>
<tr>
<td>$h/t_w &gt; 60$</td>
<td>-</td>
</tr>
</tbody>
</table>

### 4.3 Design Method

This section presents a methodology that demonstrates the use of the design recommendations. Using the recommended deck form system stiffness and strength requirements, the design buckling load can be determined from the method developed by Helwig [1995] and Egilmez [2007].

**Step-1**

In this design method, the first step is to check the web shear and bend buckling capacity of the bare girders against any factored dead and live loads that would exist prior to placement of the decking system diaphragm. The capacity of the girders should be limited to web shear and/or bend buckling capacity. Equations for both of these phenomenons are covered in AASHTO specifications. The bend buckling causes an increase in the stress in the compression flange. The web bend buckling capacity can be calculated using the equation (4.1).

$$ M_r = \left[ \frac{\lambda^2}{\left( \frac{D_c}{\tau_w} \right)^2} \right] S_{xc} \quad (4.3) $$
This equation is used to solve for the maximum moment allowed to prevent bend buckling. Since the girders are doubly symmetric, the value of \( \lambda \) will be taken as 15400, as opposed to 12500 for singly symmetric girders. The small flange in compression or the parameter \( \lambda \) from above equation can have one of the following values:

- 12500 for members with compression flange area less than tension flange area,
- 15400 for members with compression flange area equal to or greater than tension flange area.

The girder cross-sectional properties can be found in the AISC manual specifications. The shear buckling capacity is controlled by AASHTO specifications and can be checked using the following equation:

\[
V_w = 0.5D t_w F_y
\]  
\[(4.4)\]

\( F_y \) = yield capacity of the web material

\( t_w \) and \( D \) = web thickness and depth, respectively

As web buckling is caused by shearing stresses or bending stresses, closely spaced stiffeners and a thick web are used to eliminate web buckling resulting from shear and/or bending stresses.

**Step-2**

Lateral Buckling Check:

There are several loading stages that must be considered when designing composite plate girder bridges. The critical stage for the bending capacity of the steel section usually occurs during the placement of the concrete deck. During this critical stage, the girder with the metal deck form system for bracing is relied on to support the
entire construction load. The construction load consists of the weight of the steel girder, the fresh concrete, the screed, the forms, and the other equipment and personnel used to place the concrete. Two separate stages must be considered during construction.

In the first stage, the girders must be able to support their own weight and a small portion of the construction load. The cross-frame was required based upon a check of the erection/construction condition; Therefore, the total applied load that the girders must be able to support for enough stability can be estimated as the sum of the loads due to the self weight of the girders and a small portion of the construction live load (5—10 lb/ft²). This entire load must be carried by the steel section alone. These loads and length is used to calculate the required girder erection moment or the moment that girder must support.

\[ M_e = L \cdot F \frac{w_e L^2}{8} \]  

(4.5)

One must check if the girder can carry this moment with no intermediate braces using Timoshenko’s solution or the AASHTO Standard specification equation. (i.e., check \( C_b^* M_{AASHTO} M_e \))

The capacity of the girder for lateral torsional buckling of a doubly symmetric section using girder section properties is calculated either by Timoshenko’s solution or the AASHTO equation. Timoshenko’s solution for lateral torsional buckling capacity is explained briefly in the chapter Three and is not applicable for a singly symmetric section, since it produces unconservative estimates of buckling load when the small flange is in compression and conservative estimates when the larger flange is in compression. Therefore, the capacity of the girder using AASHTO specifications for lateral torsional buckling, as shown in the equation (4.6).
Making use of the girder section properties, buckling capacity of girders can be calculated using AASHTO equation. There are three approaches included here to solve for the bracing of bridge girders during construction against loading from construction and wind. They are included in subsequent section.

### Step-3

**Moment Capacity Equations:**

Calculate resulting moment capacity for girders when,

1. Girders braced without contribution of deck form

   For a girder subjected to moment gradient, the buckling capacity is calculated as the product of the corresponding $C_b^*$ values, and the buckling moment is predicted by one of the lateral torsional buckling formulas.

   \[
   M_{cr} = C_b^* M_{AASHTO}
   \]  

   (4.7)

   This provides the estimated capacity of the girder without the metal deck form. The effect of load height on buckling capacity must be considered; therefore, the load height factor $C_b^*$ is calculated as the ratio of $C_b$ to $B$.

   \[
   C_b^* = \frac{C_b}{B}
   \]  

   (4.8)

   The factor $C_b$ is accounted for moment gradient along the girder length and can be calculated using AISC specification. The variable $B$ can be calculated using expressions provided in chapter Three.
2. Girders braced with contribution of metal deck forms

The buckling moment formula for girder systems with the contribution of a decking system as bracing diaphragms was developed by Helwig [1994].

\[
M_{cr} = C^*_b M_{AASHTO} + mQd
\]  

(4.9)

Where:

- \( M_{cr} \) = Buckling moment of girder and deck system
- \( M_{AASHTO} \) = Buckling moment calculated using the AASHTO formula
- \( Q \) = Shear rigidity of decking system
- \( C^*_b \) = Moment gradient factor that considers load height effects
- \( m \) = Constant that depends on loading
- \( d \) = Depth of the girder cross-section

The term \( mQd \) represents the contribution from the PMDF/support angle connection.

3. Bracing behavior of metal deck form system with stiffening angle connection

To account for the effect of the stiffening angle, according to Egilmez [2007], the bracing of the stiffening angles is approximated by using 50\% of the buckling moment computed by using \( L_b/2 \) to evaluate the buckling capacity, where \( L_b \) is the spacing between the cross-frames.

This approach provides conservative estimates for most problems, since the stiffening angles probably provide significantly higher bracing, and also gives a simple solution that can be compared along with the FEA solutions. Therefore, equation (3.13) becomes as follows:
\[ M_{ce} = \frac{C_b^* M_{AASHTO(L_e/2)}}{2} + mQd \]  

(4.10)

Step-4

For case 2 and 3, the recommended additional moment that could be carried due to metal deck form bracing is then calculated.

The actual design problem can be solved in the following two ways.

1. **When Shear rigidity of the deck system is unknown**

   For this case, calculate the loading due to girder self-weight, concrete slabs, and construction live loads and estimate the factored dead load moment for the given bridge example. With girder capacity known, check the brace stiffness and strength requirements as follows:

   1. **Check Brace Stiffness Requirement**

      A. Ideal Deck Shear Stiffness \((G'_{\text{ideal}})\)

         1. Tributary width of the deck bracing a single girder, \(S_d\):

            The tributary width will be equal to a clear span of the PMDF per girder and can be found by the equation (4.11):

            \[ S_d = \frac{(S_g - t_f)(n_g - 1)}{n_g} \]  

            (4.11)

         2. Ideal deck shear stiffness \((G'_{\text{ideal}})\) can be estimated using the following expression

            \[ G'_{\text{ideal}} = \frac{(M_u - M_{\text{recommended}})}{s_g md} \]  

            (4.12)

            Expression for \(M_{\text{recommended}}\) can be taken as follows:

            **Case 1: Without stiffening angle:** \(C_b^* M_{AASHTO}\)
Case 2: With stiffening angle: $(1/2)C_0^*M_{AASHTO(L_w/2)}$

The bridge example covered in the appendix C will cover the design calculation for both cases, and results will be compared to evaluate the effect of a stiffening angle on bracing behavior.

B. Required Deck Shear Stiffness

To control the girder deformations provide four times the ideal stiffness.

$$G'_{req'd} = 4G'_{ideal}$$ \hspace{1cm} (4.13)

To use a metal deck form system for the bracing of bridge girders during construction, provide a metal deck form system that has shear stiffness more than $G'_{req'd}$. The brace must satisfy stiffness and strength requirements; therefore check the strength requirement.

2. Check Brace strength Requirement

The strength requirements are measured in terms of force exerted on the braces by the structure. Previous studies have shown that stability brace forces are reduced if a larger value of brace stiffness is used (AISC or Yura 1995). For a deck form system with the brace stiffness of $4Q_{ideal}$, the required bracing moment per unit length of a girder is as follows:

$$M'_{reqd} = k \frac{(M_{cr} L)}{d^2}$$ \hspace{1cm} (4.14)

Though the recommendations presented in this section are only for sections with an $h/t_w$ ratio less than 60, for the case where $h/t_w$ is greater than 60, we can still use this value of $k$ as 0.00015 for the strength requirements, as proposed in the strength recommendation.
section. Since the web slenderness gets to be a problem for cases when the web may be near stresses for shear buckling or web bend buckling, with higher web slenderness and a relatively long-span, this problem can be minimized.

2. When Shear rigidity of the deck system is known

In this case, calculate the design load to show the increase in the buckling capacity of the girder system after considering a metal deck form diaphragm system as a bracing system. To calculate the total design buckling capacity, take the addition of the recommended AASHTO moment capacity equation of the bare girders and the additional moment contribution from the shear diaphragm bracing. This additional moment contribution from shear diaphragm bracing consists of deck form system rigidity; i.e., \( mQ_{sysd} \). The metal deck form system stiffness is comprised of the deck’s stiffness and the connection’s stiffness.

Deck Shear Rigidity

The recommended shear rigidity of the deck is calculated by multiplying the effective shear modulus by the effective or tributary width shown as follows:

\[
Q_{\text{deck}} = \frac{G' \times (\text{deckspan})}{2}
\]  

This is very similar to an actual design in which the effective shear modulus \( G' \) would be calculated using the formulas in the manual from the (SDI)/procedure developed by Currah or perhaps recommended values from deck manufacturers.

Connection Shear Stiffness

In order to determine the shear rigidity of the connection, the values of the normalized connection shear rigidities should be divided by half span of the deck as follows:
The recommended connection stiffness for stiffened and unstiffened strap details connections can be estimated with the use of FEM or the method proposed by Soderberg [1994]/Currah [1993].

The shear rigidity of the deck system would then be calculated by combining the shear rigidity of the deck with that of the connection. Since the decking system stiffness is comprised of the deck’s stiffness and the connection stiffness, combining the deck and connection stiffness to determine the recommended stiffness of the diaphragm system is given by the equation (4.17):

\[
\frac{1}{Q_{\text{sys}}} = \frac{1}{Q_{\text{deck}}} + \frac{1}{Q_{\text{conn}}}
\]  

Where:

\(Q_{\text{deck}}\) = Shear rigidity of the deck

\(Q_{\text{conn}}\) = Connection shear rigidity

It should be noted that \(Q_{\text{sys}}\) must be less than or equal to the smallest of either \(Q_{\text{deck}}\) or \(Q_{\text{conn}}\).

For different loading conditions, the resulting increase in buckling capacity provided by deck system bracing can be calculated using \(mQ_{\text{sys}}d\). The total design moment recommended is then calculated by adding the recommended AASHTO buckling capacity of the bare girders with the additional moment capacity calculated by the PMDF bracing system. The calculations are presented in the bridge example covered in Appendix C.
CHAPTER 5
FINITE ELEMENT ANALYSIS TECHNIQUE AND RESULTS

5.1 Executive Summary and Introduction

Current FEM analysis methods do not have the ability to accurately predict the stability criteria for long-span deep steel plate girders during construction; therefore, in this study, three-dimensional finite element bridge models have been developed to more accurately predict bridge behavior during construction. Before going into detail, we briefly review the finite element work done on a similar subject.

Helwig [1994] studied the effect of permanent metal deck forms on the buckling capacity of straight girders using the finite element program ANSYS. Typical connections of the forms to the girders are with self-tapping screws to angles attached to the top flange of the girder. Helwig [1994] used four-node shell elements to represent the forms. Coupling the translational degrees of freedom of the corner nodes of the form elements to the centerline nodes of the top flanges would allow only shearing deformation. To avoid local buckling problems that occurred in preliminary models, the forms were given a unit thickness, and the modulus of elasticity was varied to achieve the desired shear rigidity. In addition to varying the elastic modulus, local buckling was controlled by modeling the corrugations in the metal forms with beam elements that would stiffen the forms out-of-plane. Helwig [1994] used existing closed form solutions
of prior researchers for “fully braced” beams to check the accuracy of his models. The finite element results were compared to existing solutions for beams braced by shear diaphragms. These solutions were used to develop a design approach for single-span and continuous girders braced by the permanent metal deck forms. It was found that this design approach reduces the number of cross-frames required to laterally brace the girders.

Egilmez et al. [2003] continued the work of Helwig [1994] and proposed an improvement in the FEM technique in order to perform parametric studies on the behavior of the steel I-girders braced with permanent metal deck forms. A combination of shell, beam, and truss elements was used to model the structural components of the twin-girder system. The improved modeling technique involved creating a shear diaphragm truss panel consisting of two-node truss elements spanning between two girders. The truss panel was connected to the top girder flange by coupling the translational degrees of freedom between the nodes along the centerline of the top flange and the ends of the truss panel. The models were calibrated by adjusting the areas of the truss members in the panels to match laboratory test results of a real two-girder system subjected to lateral displacement and buckling tests. Comparisons of laboratory and FEM result revealed that simple modifications to the connection can greatly improve the shear strength and stiffness of the permanent metal deck form system. After comparing the laboratory and FEA values of effective shear modulus $G'_{exp}$ for eccentric unstiffened and stiffened connections, it was concluded that providing stiffening angles to this system increases the stiffness by more than factor of four. In chapter Four, the applicability of the design equations developed by others for very long-span, deep plate girder bridges while
considering contributions from metal deck form is presented. Demonstration of these design equations on an actual bridge example, which consists of very long-span, deep girders is also presented in Appendix C.

In this chapter, FEM is carried out for an actual bridge to study the effect of the metal deck forms on the stability of very long-span, deep bridge girders against wind load. The primary purpose of this FEM was to compare and correlate the shear rigidity of the girder system with and without metal deck form to those calculated using SDI [1995] manual. The 3D static and dynamic FEM analysis results and comparisons are presented to study the implication of prior works as applied to the construction of long-span plate girder bridges. Since the ultimate goal of this project is to study the effect of metal deck form on natural frequencies and time period, an introduction to dynamic FEM analyses is also presented to study the effects of PMDF on the vibration characteristics of the bridge superstructure. In-depth study of the effect of metal deck form on vibration characteristics of bridge girder system will be presented in the future.

The study is focused on creating a modeling method for both single-span and two-span continuous bridges. Parameters such as cross-frame, stiffeners, and permanent metal deck forms were considered in the development of the FEM. By calculating the deflections of the girder system with and without metal deck form due to wind load during the construction, the shear rigidity of the bridge was captured and compared to the results from FEM.

To understand the bracing behavior of the PMDF system during construction, it is necessary to have an idea about the effect of wind loads on the PMDF system. The main objective is to determine the lateral stiffness of PMDF systems subjected to deformations
similar to the deflected top flange profile of buckled girders. Using the relationship proposed in the SDI [1995] diaphragm design manual, calculate the lateral deflection of PMDF system which is given as follows:

$$\Delta = \frac{P \cdot H}{G'_s \cdot B}$$

(5.1)

Where:

- $P$ = Axial force
- $H$ = Panel width
- $B$ = Panel length
- $G'_s$ = PMDF system shear stiffness

Detailed finite element models of steel plate girder bridges have been created using the commercially available finite element analysis program ANSYS, a powerful commercially available software package for both computer-aided design (CAD) and FEM analysis. Linear/static analysis is performed, which is used when determining structural displacements, stresses, strains, and forces that result from loading that does not generate significant inertia and damping effects. Dynamic modal analysis is also carried out to study the change in the mode shapes and natural frequencies of the girder/metal deck form system due to wind during construction. All components in the bridge superstructure were modeled with linear elastic material models. For the elements representing the structural steel, the elastic modulus used was 29,000 ksi, and the Poisson’s ratio used was 0.3.

The finite element models developed in this research include specifically detailed bridge components. These components include the plate girders, cross-frames/diaphragms, metal deck forms, load applications, and stiffeners. The subsequent
sections in this chapter will outline the modeling techniques used and the development of each component of the finite element models.

5.2 Plate Girders

The modeling of girder webs, flanges, stiffener plates, and supporting angles was carried out using four-node structural shell elements (SHELL93) in ANSYS. The SHELL93 element has six degrees of freedom at each node, with both bending and membrane capabilities that include shearing deformation (ANSYS 2003). The basic geometry of the girder cross-section was created by using keypoints (Figure 5.1). The positions of the keypoints were obtained from the bridge construction drawings and connected with lines. Using these lines, areas were created that were meshed with SHELL93 elements. Figure 5.1 contains perspective views and a cross-sectional view of a single-span, single-girder model meshed with SHELL93 elements.

Instead of very long continuous spans, single-span girder model was created in ANSYS. The single-girder models were given constant cross-sections and subjected to uniformly distributed loads, which were applied laterally as uniform pressures to the girders. The girder cross-section is as shown in Figure (5.1). For the case of the single-span girder, deflections based on traditional beam theory were obtained from hand calculations and compared to the ANSYS results. These preliminary ANSYS models produced accurate results (<1%) as compared to the hand calculations and, therefore, the girder modeling method used for the ANSYS models was deemed adequate to use for an entire bridge model.
5.3 **Stiffeners and Supporting Angles**

Bearing stiffeners, intermediate web stiffeners, and supporting angles are typical of the studied bridges. Bearing stiffeners are present to stiffen the web at support bearing locations, intermediate web stiffeners are used for web stiffening along the span, and support angles are used as connections between girder top flange and metal deck forms that were tied together with the help of welding or TEK screws. The bearing stiffeners, intermediate web stiffeners, and supporting angles are modeled by creating areas between web keypoints and keypoints at the flange edge. On the actual girders, stiffeners and plates are of constant width and rarely extend to the flange edge. This is confirmed by Figure 5.2, which displays oblique and cross-sectional views of bearing and intermediate web stiffeners. Actual plate thicknesses are attained directly, from the bridge construction.
Figure 5.2. Oblique and front view of girder with stiffeners

plans and applied appropriately during the FEM. Web stiffening plates and supporting angles were modeled into the finite element models using four node SHELL93 elements. All web stiffeners were spaced 25 ft away from each other. In the actual bridge being modeled, these plates are fully welded along the height of the web and are also welded to the top and bottom girder flanges. Modeling this detail can be found difficult if the top and bottom flanges are not of equal width, and the plates may not extend exactly to the edge of the flanges.

Although there were difficulties while modeling web stiffeners, it was later determined that this approach showed the proper behavior between connected girders and flanges. The bearing stiffeners were modeled by creating keypoints on the top and bottom
flanges and connecting these keypoints to the girder along top and bottom web keypoints. This technique allowed the girder cross-section to deflect/rotate, truly representing the connection between the plates and the girder flanges. Generating the model was noticeably tedious when using the nominal plate geometry, because the stiffeners tend to vary in width depending on their function. For example, the bearing stiffeners are not typically the same width as the connector plates for the diaphragms or vertical web stiffening plates. This problem was avoided by keeping constant widths of the stiffener plates. This method allowed the girder cross-section to deflect/rotate and was considerably easier to implement in the models, since it did not involve the creation of nodes in addition to the ones already in place for the girder. Figure 5.2 contains a perspective and end view of an ANSYS girder with bearing and web stiffeners modeled using this approach.

5.4 Permanent Metal Deck Forms

Based on loading during construction and SDI [1995] design specifications, a metal deck form was designed for this particular bridge example. A metal deck form with 20-gauge thickness was selected with a 30-inch cover width. Shear properties, i.e., shear stiffness and shear strength, were calculated using SDI [1995] design specifications, and these values were cross-checked with the manufacturer’s values.

Helwig [1994] and Egilmez [2007] performed comprehensive analytical studies involve the modeling of metal deck forms/girder assembly. For modeling of metal deck forms, these researchers employed a method that involves the use of two-node three-dimensional LINK8 truss elements (struts and diagonals) spanning between the top girder flanges by coupling all of the translational DOF’s (global x, y, and z directions) between
the edges of the top flanges and the truss elements. Although the number of degrees of freedom this method uses is fewer, it takes tedious trial and error to actually determine the cross-sectional areas of diagonals and struts of truss until displacement matches that of truss analogy. Therefore, instead of the modeling method implemented by Egilmez [2007], which was found to be efficient in terms of the amount of degrees of freedoms it added to the models, a three-dimensional metal deck form model was created using the four-node Shell93 element shown in Figure 5.3. The cross-section of metal deck forms was modeled in ANSYS using keypoints and then connected by lines.

![Figure 5.3. 3D Metal deck form model](image-url)

As a typical bridge uses metal deck forms that rest directly on supporting angles, modeling of this connection detail proved very tedious. Therefore, to make modeling easier, the supporting angle cross-section was modeled using keypoints, and then areas were plotted with the help of lines that were created by connecting these keypoints. Four-
node shell93 elements were used to model both supporting angle and metal deck forms because of bending and membrane capabilities that include shearing deformation ANSYS [2003]. This element may have variable thickness. The thickness is assumed to vary smoothly over the area of the element, with the thickness input at the corner nodes ANSYS [2003].

![Diagram of typical girder connection detail]

Figure 5.4. Typical girder connection detail

For proper behavior of the support angle/metal deck form connection, necessary precaution was taken so that the metal deck forms share common nodes with the supporting angle nodes. Although this method used more numbers of degree of freedom, it depicts the behavior as being very similar to the actual bridge condition. This method of connecting the metal deck form shell93 elements to the supporting angle elements was believed to more accurately represent the true geometry of the connection. Figure 5.5 illustrates a close-up oblique and plan view for a four-girder bridge of an ANSYS finite element model, including the metal deck form/support angle system.
5.5 Cross-Frames and Diaphragms

Diaphragms and cross-frames are common to all bridges. According to the AASHTO specifications, the aforementioned cross-frames must transfer lateral wind loads from the bottom of the girder to the deck to the bearings, support the bottom flange in negative moment regions, stabilize the top flange before the deck has cured, and distribute all vertical dead and live loads applied to the bridge. The cross-frame components are typically steel angles or structural tees between three and five inches in size and are bolted to the connector plates. The K-type of cross-frame bracing is used in the studied bridge and is illustrated in Figure 5.6.

Initially these cross-frames are modeled with truss elements so that each member of the cross-frame represents one single element. Hence the cross-frame members became very stiff, which showed bizarre results; therefore, each cross-frame member is modeled with the help of the beam element. The cross-frame member section properties were computed using the AISC Manual of Steel Construction and implemented into ANSYS [2003].
Figure 5.6. K-type cross-frame finite element model

Two approaches have been tried to model cross-frame bracing members. In the first approach, every cross-frame was modeled by creating lines between the girder keypoints existing at the intersection of the web and flange centerlines. On the actual girders, the cross-frame connections are offset from the flange to web intersection to allow for the connection bolts. Because of modeling difficulties, which consist of 3D permanent metal deck form geometry, it was not possible to create lines at the intersection of the web and flange centerlines.

To overcome this modeling difficulty, instead of plotting keypoints at the intersection of the web and flange centerlines, keypoints were plotted 8 inches below the top flange and 8 inches above the bottom flange at each respective girder. This simplifying assumption had very little effect on the predicted girder deflection.
The following procedure is followed to perform a specific modeling procedure:

- Material property set is defined for the steel.
- Finite element types and analysis is defined (SHELL/BEAM, static analysis, etc.).
- Real constant sets are defined, including deck form and support angle thicknesses, beam moments of inertia, etc.
- Keypoints are created for the girders, web stiffeners, supporting angle, and metal deck forms.
- Areas are generated between the keypoints to represent the girders, web stiffeners, supporting angle, and metal deck forms.
- Attributes are applied to all of the modeled areas (attributes include the element type and real constant set); then they are sized appropriately and meshed to create the girder and slab elements.
- Lines are created between existing and newly originated keypoints to generate all three cross-frame types, as applicable.
- Attributes are applied to the modeled lines; then they are sized and meshed to create the rigid link and cross-frame elements.
- Nodes of the metal deck forms share common nodes with the support angle to ensure model finite element compatibility.

Due to restrictions on the use of the number of DOF in ANSYS software, the most critical and the maximum span length girders were selected. Using cross-section properties of these girders and other bridge components, a model was created that consists of four girders spaced at 120 inches, web stiffeners and cross-frames at 300 inches apart from each other. As the bridge was modeled as a simply supported single-
span bridge, simply supported boundary conditions were used. All translations, i.e., Ux, Uy, Uz, are restrained in X, Y, and Z directions respectively, whereas all members are allowed to rotate.

5.6 Load Calculation and Application

As the main goal of this research is to study the effect of wind load on very long deep-bridge girders during construction a critical wind speed of 70 to 100 mph is considered during the FEM of structure. This load is converted into uniform pressure and then applied laterally to the rightmost exterior girder. Efforts were made to emulate actual bridge conditions. Calculation of the lateral load due to the wind load was performed based on the height of the girder. Loads were calculated for the rightmost exterior girder based on the girder depth obtained from the construction plans. Only wind load was considered. After checking the accuracy of the structure, elastic static analysis was carried out on the four-girder bridge model. This analysis was performed for two different cases:

1. Girders attached without permanent metal deck form, i.e., bracing with cross frame only.

2. Girders attached with permanent metal deck form.

These cases are essential since it was demonstrated analytically and experimentally by Helwig [1994] and Egilmez [2007] that a metal deck can be used for the bracing of short-span, shallow bridge girders during construction. It is important to understand what effect (if any) metal deck forms have to stabilize deep, long-span bridge girders during construction. The applicability of previous researcher’s design equations, which include
contributions from metal deck forms, for long-span, deep bridge girders must be validated. The applicability of these equations to deep, long span bridge girders, during construction is addressed in this study.

First of all, finite element models with two and four girders were run without a captivating contribution from metal deck forms. The isometric view of the bridge model is shown in Figure 5.7.

![Figure 5.7. 3-D finite element model of four girders with cross-frames](image)

Loading was applied in a lateral direction, and maximum deflection results were obtained. After the completion of analysis without metal deck forms, two and four-girder bridge models with metal deck forms were analyzed. This analysis focuses on determining the lateral stiffness of PMDF systems subjected to deformations similar to
the deflected top flange profile of buckled girders; therefore, deflection for the top flange middle section is measured instead of maximum deflection. The lateral stiffness of the system was obtained by dividing applied force by the corresponding lateral deflection at that point.

Figure 5.8. 3-D finite element model of four girders with metal deck form

As stated earlier, effective bracing must satisfy stiffness and strength criteria. Therefore, during this analysis, lateral stiffness is a prime parameter to study the effect of the stiffness of PMDF on the stiffness of the total system stiffness (in other words, effect on bracing). Lateral stiffness represents distortion produced under the influence of shear forces in its own plane. The need to understand the lateral movement is important for assessing the transfer of forces through metal deck forms between adjacent girders. Figure 5.8 shows a finite element model built in ANSYS for a four-girder system with metal deck forms.
5.7 Modelling Comparisons and Results

During this research phase, the bridge was modeled with and without metal deck forms. Only the midspan top flange deflections are included for the simple span bridge structure. A complete deflection summary for both models is tabulated and graphed and is included in the following subsections.

5.7.1 Summary of ANSYS Results without PMDF contribution

Table 5.1 contains the summary of the mid-span top flange deflection predicted by ANSYS with the modeling methods is presented. Figure 5.9 shows the location where the lateral deflection in X-direction has been measured. All of the relevant results, pictures, and graphs for the studied bridge are presented as the section progresses.

Deflection at keypoints is more important than the maximum deflection of the girder system; therefore, deflections at these points were observed. The deflections at the rightmost and leftmost exterior girders for these keypoints in a four-girder system without metal deck forms is also tabulated and graphed in the Table 5.1.

Hand calculated displacement results based on simple beam theory were obtained for a single girder and then compared to the ANSYS results. These ANSYS models showed accurate results less than 1% as compared to the hand calculations and, therefore, the girder modeling method used for the ANSYS models was deemed adequate for use in an entire bridge model. During FEM for two and four girder systems, it was seen that both systems showed displacement values less than that of hand calculated displacement results. This is because hand calculation involves the use of simple beam theory for calculating lateral displacements, which does not account for the bracing effects provided by cross-frames, and therefore shows conservative results.
Table 5.1: Lateral displacements for girders without PMDF

<table>
<thead>
<tr>
<th>Number of Girders</th>
<th>Girder displacement without PMDF in inches</th>
<th>Theory</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FEM</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Left Girder</td>
<td>Right Girder</td>
</tr>
<tr>
<td></td>
<td>Top Flange (Avg. of three points)</td>
<td>Bottom Flange (Avg. of three points)</td>
</tr>
<tr>
<td>2</td>
<td>1.9678</td>
<td>1.9725</td>
</tr>
<tr>
<td></td>
<td>1.9676</td>
<td>1.9724</td>
</tr>
<tr>
<td>4</td>
<td>0.98290</td>
<td>0.98355</td>
</tr>
<tr>
<td></td>
<td>0.98315</td>
<td>0.98379</td>
</tr>
</tbody>
</table>

5.7.2 Summary of ANSYS Results with PMDF contribution

To study the effect of metal deck forms on the stiffness characteristics of whole metal deck form/girder systems, an analytical model the same as that shown in Figure 5.8.
for two-girder and four-girder systems was developed. Similar uniform lateral loading was applied to the exterior rightmost girder, and lateral displacement due to this loading was monitored. With known lateral displacement ($\Delta$), metal deck form panel length ($B$), width ($H$), and applied lateral force ($P$), the lateral stiffness of permanent metal deck form/girder systems can be calculated using procedure presented in the appendix B. Sample lateral stiffness calculation for the studied bridge is shown in the appendix B. Girder displacements seen for case with metal deck forms were measured at the same keypoints chosen before. Deflected profile of four girder bridge system with deflection results is presented in Figure 5.10 and Table 5.2, respectively.

Although lateral displacement of the girder system is presented here, the main focus of this section was to study the effect of a metal deck form system on the stiffness of the whole bridge system. During the modeling of a girder/metal deck form system, special attention was provided for connection detail between metal deck forms and support angles. The worst connection detail in a bridge system is used, which is the unstiffened detail when the decking is below the flange of the girder at its extreme eccentricity. This connection was used to emulate the exact behavior which the actual bridge connection possesses. This is necessary since previous researchers showed that, for short-span, shallow bridge girders, the presence of this kind of eccentric connection detail, results in substantial decrease in the total stiffness of the bridge system. Therefore, to study the effect of metal deck form/support angle connection details on the stiffness of long-span, deep bridge girder systems, lateral displacement analytical tests were conducted. Based on the obtained displacements from this analysis, the lateral stiffness of the bridge system will be calculated and then compared to theoretically calculated results.
discussion based on these comparisons will be presented.

Therefore, first of all, a complete summary of lateral displacement results for both models is tabulated. The description girder labels will be the same as described previously. Table 5.2 shows the average displacement results at particular keypoint position. It can be seen that girder top and bottom flange displacement reduces substantially when a metal deck form is installed on girders with the help of support angle connections. Based on these displacement results, the lateral stiffness of the metal deck form/girder system is estimated and presented in Table 5.3.
Table 5.2: Lateral displacements for girders with PMDF

<table>
<thead>
<tr>
<th>Number of Girders</th>
<th>Left Girder</th>
<th>Right Girder</th>
<th>Theory</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top Flange (Avg. of three points)</td>
<td>Bottom Flange (Avg. of three points)</td>
<td>Top Flange (Avg. of three points)</td>
</tr>
<tr>
<td>2</td>
<td>0.14067</td>
<td>0.19125</td>
<td>0.14018</td>
</tr>
<tr>
<td>4</td>
<td>0.04151</td>
<td>0.047035</td>
<td>0.040371</td>
</tr>
</tbody>
</table>

Table 5.3: Lateral stiffness for girders with PMDF

<table>
<thead>
<tr>
<th>No. of Girders</th>
<th>FEM (inches)</th>
<th>Theory (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Girders No deck</td>
<td>Girders With Metal Deck Form</td>
</tr>
<tr>
<td>2</td>
<td>1.97</td>
<td>0.141</td>
</tr>
<tr>
<td>4</td>
<td>0.983</td>
<td>0.0459</td>
</tr>
</tbody>
</table>

The theory lateral stiffness calculation procedure is based on the classic equation for springs in series given by the equation (5.2).

$$\frac{1}{G_{sys}} = \frac{1}{G_{deck}} + \frac{1}{G_{conn}}$$  \hspace{1cm} (5.2)

The full lateral stiffness calculation procedure for studied bridge is demonstrated in the Appendix A.
The lateral displacement from hand calculated results was compared with FEA model results, and the comparison revealed that the metal deck form system controlled deformation substantially as compared to a girder system without metal deck forms. It can be seen from Table 5.3 that for two and four-girder systems, FEM lateral stiffness results show very small difference (around 2%) with theoretical results. For a full bridge finite element model, obtained results for the lateral stiffness of the bridge system with extreme eccentric support angle connection shows good agreement with theoretical results. The difference between results is less than 2%, which indicates that metal deck forms provide sufficient stiffness to the deep-plate girders during construction. This shows that the PMDF system possesses sufficient in-plane stiffness capacity and that it can be more efficient to treat the PMDF system as a bracing element.

The metal deck form systems studied in this case satisfy the stiffness criteria, which is one of the major criteria for bridge girder stability bracing. Based on better agreement as to lateral stiffness results between FEM and theory calculations, this case study provides a solid platform to make preliminary conclusion that, because of sufficient in-plane flexural stiffness, a metal deck provides sufficient lateral stability bracing to long-span, deep plate girder bridges during construction.

During this bridge case study, many parameters were included that might affect bridge stability, such as cross-frames, metal deck forms, and stiffeners. The predicted results have a very good agreement with the theoretically calculated results; however, it is apparent that the finite element bridge modeling has many details and can be very complicated. As only one bridge case study was studied, it can not be proposed that design equations formulated by others are applicable for deep, long-span bridge girders,
but this study will definitely provide sufficient background information on the effect of metal deck forms on the bracing behavior of deep, long-span plate bridge girders. Since the ultimate goal of this research project is to study the possible vibration effects of metal deck forms on bridge girder system stability during construction, introduction to dynamic finite element analysis is presented in the subsequent section.

5.8 Dynamic Behavior of Plate Girders with PMDF

Long-span steel plate girder bridges during construction conditions are very susceptible to wind action due to their great flexibility, so wind stability is becoming a major concern in the design and construction phrases. Due to higher fabrication and maintenance costs, ALDOT is replacing existing truss bridges with long-span steel plate girder bridges. Engineering projects across straits or rivers are being planned around Alabama, and many long and super long-span steel plate girder bridges are proposed. During construction of deep, long-span bridge plate girder bridges, ALDOT engineers encountered vibration problems due to high wind, which leads to the instability of the compression flanges of plate girders. As compared with the service condition, although the period of construction is not too long, the structural stiffness of plate girder bridges under construction is reduced greatly, and consequently they become very susceptible to the dynamic wind action.

To overcome these difficulties during construction, ALDOT invited researchers at the University of Alabama at Birmingham to investigate and deliver simple design method/checks to reduce wind instability during construction. Efforts are underway to study the dynamic vibration behavior of bridge girders against very high wind. In this study, only the introduction to vibration characteristic of bridge girders has been
presented. Dynamic modal analysis is performed to observe the change in mode shapes and natural frequency of the bridge girder system with the presence of metal deck forms. Another student will perform full dynamic vibration analysis research on bridge plate girders. Therefore the overall purpose of the proposed effort is therefore to improve bridge design efficiency and construction safety by developing strength definition and engineering methodology.

Since ALDOT engineers encountered vibration problems even with the presence of such discrete bracing systems as cross-frame, an alternative bracing system such as a metal deck form is needed. In this study, dynamic modal analysis is carried out on the previously developed bridge models. The following assumptions were used during the dynamic modal analysis of girders:

- The beams are composed of a linear, homogeneous, isotropic elastic material.
- No loads are applied to the girders.

Initially, the accuracy of the structure was checked by performing modal analysis on a single girder, and natural frequency results from this analysis showed very good agreement when compared with theoretically calculated results. Natural frequency was calculated theoretically using a simple beam equation as follows:

\[ f = \frac{\lambda^2}{2\pi^2} \left( \frac{EI}{m} \right)^{\frac{1}{2}} \]  

(5.3)

Where:

\[ m = \text{mass per unit length of beam}, \quad EI = \text{Flexural rigidity} \]

Two and four-girder bridge systems with and without metal deck forms have been analyzed, and the results for natural frequency and different mode shapes for these girder
systems are presented in Table 5.4 and Table 5.5. The necessary input data, such as material properties, i.e., density required, is assigned for dynamic modal analysis. Any structure with mass and elasticity possess one or more natural frequencies of vibration. To study the vibration effect of metal deck forms on bridge girders, overall changes in natural frequencies in hertz and period in second is observed. Therefore Table 5.5 enlists the natural period of the girders with and without metal deck forms for two and four-girder system.

It can be seen from Table 5.5 that the natural periods for two-girder and four-girder system remain very close to (4 second compared to 3.98 second). Natural period for a two-girder system decreases from 4 second to 1.19 second, and the natural frequency of the girder system is decreases from 0.84 Hz to 0.25 Hz. This means that as the stiffness of the whole structure is decreases we can conclude that the overall stiffness of the system is also decreases since natural frequency of any linear structure is directly proportional to system overall stiffness.

Although metal deck forms have sufficient in-plane stiffness, the overall reduction in stiffness is because of the eccentric connection between metal deck forms and girder flanges. Although the overall stiffness of the system is decreased, this reduced stiffness is still on the higher side compared to girder systems with discrete bracing, such as cross-frames or diaphragms.

As effectiveness of brace system is primarily dependent on the change in mode shape. Mode shapes were therefore plotted to understand the bracing behavior of metal deck form/girder systems. It was seen that, when only cross-frames, i.e., discrete bracing, is used, with the change in mode shape, the braces become less effective.
Table 5.4: Natural frequencies for girders with and without PMDF

<table>
<thead>
<tr>
<th>Mode</th>
<th>Two-girder system</th>
<th>Four-girder system</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without PMDF</td>
<td>With PMDF</td>
</tr>
<tr>
<td>1</td>
<td>0.84</td>
<td>0.25</td>
</tr>
<tr>
<td>2</td>
<td>2.021</td>
<td>0.98</td>
</tr>
<tr>
<td>3</td>
<td>2.863</td>
<td>2.079</td>
</tr>
<tr>
<td>4</td>
<td>3.359</td>
<td>3.04</td>
</tr>
<tr>
<td>5</td>
<td>3.463</td>
<td>3.204</td>
</tr>
</tbody>
</table>

Table 5.5: Natural period for girders with and without PMDF

<table>
<thead>
<tr>
<th>Mode</th>
<th>Natural Period (second)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Two-girder system</td>
</tr>
<tr>
<td></td>
<td>Without PMDF</td>
</tr>
<tr>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>1.021</td>
</tr>
<tr>
<td>3</td>
<td>0.481</td>
</tr>
<tr>
<td>4</td>
<td>0.328</td>
</tr>
<tr>
<td>5</td>
<td>0.312</td>
</tr>
</tbody>
</table>

Figure 5.11 shows mode shapes corresponding to the first and fifth modes of the two-girder system without metal deck forms.
Analysis was performed on both four and two-girder systems with metal deck forms connected at the top with the help of support angles. The mode shapes corresponding to the first and second modes of the structure are shown in Figure 5.12.

In this study, during the analysis of girders with metal deck forms, not many changes in the mode shape were seen until the girder buckles between the braces. Girder buckling was not observed for four and two-girder metal deck form systems. The girder will generally buckle in a mode shape for which the bracing system is the least effective. During this analysis it was seen that that girders braced by a metal deck forms do not demonstrate a succession of changes in mode shape with increasing deck shear rigidity.

During dynamic modal analysis of bridge girders with and without metal deck forms, parameters were included that might have effects on the bridge girders vibration behavior, such as cross frames and metal deck forms. The natural frequencies and mode shapes of these structures were studied and the results presented. During this analysis, natural frequencies of the structure were seen decreasing gradually and metal deck forms do not demonstrate a succession of changes in mode shape with increasing deck shear rigidity.
Although the overall stiffness of the system seen decreased because the girder metal deck forms eccentric connections, this reduced stiffness is still on the higher side compared to girder systems with discrete bracing, such as cross-frames or diaphragms. Although dynamic behavior of metal deck form systems was carried out to study the vibration phenomenon, detail vibration dynamic analysis is necessary to understand the actual dynamic behavior of girder/PMDF systems during construction.
CHAPTER 6
SUMMARY AND FUTURE WORK

Due to higher fabrication and maintenance costs, ALDOT is replacing existing truss bridges across rivers around Alabama, with deep, long-span, steel plate girder bridges. During construction of these bridges, ALDOT engineers encountered vibration problems due to high wind, which leads to instability of the compression flanges of plate girders. The wind stability of deep, long-span steel plate bridge girder is becoming a major concern during the design and construction phases. As compared with the service condition, although the period of construction is not too long, the structural stiffness of plate girder bridges under construction is greatly reduced because of the girder top flange connection with metal deck forms. Cross-frames/diaphragms have generally been used for lateral wind stability of girders during construction, but because of complications in erection and fabrication due to cross-frames and continuous fatigue problems, researchers have demonstrated that metal decking can provide significant lateral stability to the short-span, shallow girder system. This investigation was initiated to better understand and define these construction stability issues and the role that the metal decking can play in enhancing stability during construction of deep, long-span plate girder bridges.

Because of the amount of literature available, this study was divided into four different phases:
1. Review work of others on PMDF for stability.

2. Study applicability of design equations proposed by others to deep, long span plate girder bridges

3. Use static and finite element analyses to study the implication of prior works as applied to the construction of long-span plate girder bridges.

4. Use dynamic finite element analyses to study the effects of PMDF on vibration characteristics of the bridge superstructure

6.1 Literature Synthesis

During this study, efforts were taken to understand the total literature done and published. Research on the use of metal deck forms as a bracing element for the building industry was started during the 1960 and 1970’s. But the PMDF system in the bridge industry differs substantially from the form system used in the building industry. Therefore, Texas researchers performed a comprehensive research program that includes experimental and analytical studies of the bracing of shallow, short-span bridge girders using permanent metal deck forms during construction.

To increase the overall stiffness of the bridge girder system, which decreases substantially due to flexible/eccentric connection details between girder flange and metal deck forms, Texas researchers proposed new modified connection details which improved the stiffness of the existing metal deck form system. Researchers developed design equations to implement with actual bridge girders. One of the goals of this study was to thoroughly understand these prior formulated design equations, since those equations were used for long-span, deep plate girder bridges and transfer/recommend
design methodology to Alabama stakeholders for future possible development of the concepts to improve engineering and construction practice.

6.2 Applicability

The primary objective of this chapter was to propose a design methodology that considers permanent metal deck forms as a bracing element to stabilize long-span, deep bridge girders against wind and possible construction loading during construction. Design equations developed by others were presented for considering the contribution of PMDF systems to lateral bracing during construction. These studies have demonstrated stability for short-span highway overpass bridges, but additional work was needed to implement this design method for very long-span, deep bridge girder cross-sections. This was necessary since the magnitudes of brace moments tend to increase with the depth of the individual girder and the $L/d$ ratio for specific girder depth. To study the applicability of this method, a demonstration of the use of these expressions in existing very long-span, deep plate girder bridge structure, to provide wind bracing during construction is presented. Shear stiffness and strength criteria for considering metal deck forms as wind brace, which is primary requirement of any bracing system presented by others, is also explained.

It was observed that, for the particular bridge being studied, moment capacity for an unbraced length $L_b$ of 25 ft is much greater than the factored dead load moment. Stiffness calculations are therefore not necessary, and decking with and without modified connection detail (i.e., stiffening angle) with relative low stiffness will be able to brace the girders. This is generally the case in most long-span, deep bridge girders, since the
magnitudes of moments tend to increase with the depth of the individual girder. Finally, from a design prospective, it can be concluded that metal deck forms with relative low stiffness can be used as wind braces for long-span, very deep bridge girders during construction.

Next phases of the research were focused on using finite element analyses to validate applicability to very long span bridges and to investigate the effect of PMDF on the vibration characteristics of bridge superstructures during construction. Finite element analyses were performed that focused on the ability of metal deck forms with modified connection details to brace deep, long-span bridge girders during construction.

6.3 Static and Dynamic Finite Element Analysis

With the help of the ANSYS finite element analysis program, detailed FEM of steel plate girder bridges have been created. The same bridge that was used for design calculation was analyzed. Three-dimensional, finite element bridge models have been developed to more accurately predict PMDF bracing effects on bridge girder systems during construction. To understand the PMDF effects on the overall stiffness of the bridge girder system, linear/static analysis was performed, and all components of the bridge superstructure modeled with linear elastic material models. Parameters such as cross-frame, stiffeners, and permanent metal deck forms were considered in the development of the finite element models. Stiffness results for the girder system with and without metal deck forms due to wind load during construction were presented, and these results were compared to the theoretically calculated stiffness results. Comparison of stiffness results revealed that metal deck form systems controlled deformation more
substantially than girder systems without metal deck forms. A full bridge finite element model results for lateral stiffness of bridge system with extreme eccentric support angle connection showing good agreement with theoretical results. (around 2% difference)

Metal deck form systems studied in this case study satisfies the stiffness criteria, which is one of the major criteria for bridge girder stability bracing. Based on better agreement as to results between FEM and theory calculation, this case study provides a solid platform to make preliminary conclusions that, because of sufficient in-plane flexural stiffness, a metal deck provides sufficient lateral stability bracing to long-span, deep plate girder bridges during construction. During this bridge case study, the predicted FEM results showed very good agreement with the theoretically calculated results.

As only one bridge case study was studied, we cannot propose that design equations formulated by others are applicable for deep, long-span bridge girders, but this study will definitely provide a significant contribution to understanding the effect of metal deck forms on the bracing behavior of deep, long-span plate bridge girders. Since the ultimate goal of this research project is to study the possible vibration effects of metal deck forms on the stability of bridge girder systems during construction, an introduction to dynamic modal analysis is presented to study the vibration characteristics of the girder/metal deck form system due to wind during construction.

As the effectiveness of the brace system is primarily dependent on the change in mode shape, natural frequencies. Mode shapes for two and four-girder bridge systems with and without metal deck forms have been analyzed and the results presented. The effect of modified connection details i.e., stiffening angle has not been included in the static and dynamic finite analysis. During this analysis, it was seen that that girders
braced by a metal deck forms do not demonstrate a succession of changes in mode shape with increasing deck shear rigidity. Although the overall stiffness of the PMDF/girder system decreased because the girder/metal deck forms an eccentric connection, this reduced stiffness is still on the higher side compared to girder systems with discrete bracing, such as cross-frames or diaphragms.

6.4 Future Work

Although this study makes a significant contribution understanding the effect of metal deck forms on the bracing behavior of deep, long-span plate bridge girders, the applicability of these design expressions needs to be validated on different cross-sections of bridge girders. Stiffening angle connection detail must be captured in future finite element models. To thoroughly study the vibration phenomenon effect on the girder/PMDF system during construction, detail vibration dynamic analysis is necessary.
LIST OF REFERENCES


APPENDIX A

LATERAL STIFFNESS CALCULATIONS FROM ANSYS DEFLECTION RESULTS

When girders are braced by metal deck forms, the parameter that is of the most importance to be consider is shear rigidity, $Q_{sys}$, that has units of force per unit radian (KN/rad or kip/rad). The shear rigidity of the metal deck form/girder system is calculated as the product of effective shear modulus ($G''_{sys}$) and tributary width of deck ($S_d$)

$$S_d = \frac{(s_g - b_f)(n-1)}{n}$$

$$S_d = \frac{(10 \times 12 - 24)(2 - 1)}{2} = 48 \text{ inches}$$

The shear rigidity of metal deck form systems can be computed from the following equation:

$$Q_{sys} = G''_{sys} \cdot S_d$$

The shear rigidity ($Q_{sys}$) value for a fastened decking system is comprised of deck shear rigidity ($Q_{deck}$) and connection bending rigidity ($Q_{conn}$) and can be calculated using the following expression:

$$\frac{1}{Q_{system}} = \frac{1}{Q_{deck}} + \frac{1}{Q_{conn}}$$
\( Q_{\text{deck}} \)

The shear rigidity of the deck forms, can be calculated using the formulas in the manual from the SDI [1995] perhaps the recommended values from deck manufacturer. Shear strain imposed on the decking diaphragm results in shear force equal to the span of the decking multiplied by the average shear stiffness of the decking:

\[
Q_{\text{deck}} = G'_{\text{deck}} \times \text{deck span}
\]

Since two girders are braced by each metal deck form, half of the resulting shear force braces each girder, resulting in,

\[
Q_{\text{deck}} = \frac{G'_{\text{deck}} \times \text{deck span}}{2}
\]

\( Q_{\text{conn}} \)

In order to determine the shear rigidity of the connection, the values of the normalized connection shear rigidities should be divided by the half span of the deck:

\[
Q_{\text{connection}} = \frac{Q_{\text{recommended}}}{(\text{span}/2)}
\]

The recommended connection stiffness for stiffened and unstiffened strap details connection can be estimated with the use of finite element analysis/design software or the method proposed by Soderberg [1994].

Therefore, for this particular bridge study, computing the shear rigidity of the deck and connection separately using the previous expression, we got \( Q_{\text{system}} \) equal to 3490 kip/rad.

Finally, the shear rigidity of the metal deck form/girder system for studied bridge is,

\[
G'_{\text{sys}} = \frac{Q_{\text{sys}}}{S_d} = \frac{3490}{48} = 72.7 \text{ kip/inch}
\]
APPENDIX B

LATERAL DEFLECTION FOR GIRDERS WITH AND WITHOUT PMDF

The SDI [1995] manual uses the following simple deflection equations for girders with and without a metal deck form contribution:

Lateral deflection calculation without PMDF:

\[ \text{Deflection} (\Delta) = \frac{5}{384} \times \frac{wL^4}{EI} \]

Lateral deflection calculation with PMDF:

\[ \text{Deflection} (\Delta) = \frac{qL^2}{8BG'_{\text{System}}} \]

These theoretically calculated values were compared with finite element results, which are shown in Table 5.3.
APPENDIX C
BRIDGE DESIGN EXAMPLE

The given example considers a bridge with continuous girders. The bridge lay-out is shown in the following figure. This is a four girder bridge over the Tombigbee River on relocated state route 114 at Naheola station in Choctaw and Marengo counties. The bridge had three spans, with exterior spans of 320 ft and a center span of 405 ft. Therefore, determine the required metal deck form system stiffness to adequately brace the girders during construction?

Figure (bridge over Tombigbee River): Bridge Layout and its Components

This example will concentrate on the design of bracing for the center 405 ft span. The original design made use of 15 intermediate cross-frames spaced at 25 ft. From the design calculations factored dead load moment $M_{cr}$ is calculated as 55654 K-ft. Due to the presence of intermediate braces $C_b^*$ value is taken as 1. There are four girders laterally spaced at 10 ft and connected by metal deck forms.
Girders braced with the contribution of metal deck forms

The buckling moment formula for girder systems with the contribution of a decking system as bracing diaphragms was developed by Helwig [1994] and is shown in the following equation:

\[ M_{cr} = C^*_b M_{AASHTO} + mQd \]

The term \( mQd \) in equation represents the contribution from the PMDF/support angle connection. Due to the application of loading on the top flange, the value of \( m \) will be taken as 0.375. Rearranging the equation to estimate required shear stiffness, we get,

\[ G'_{\text{required}} = \frac{4(M_{cr} - C^*_b M_{AASHTO})}{s_d m d} \]

\( s_d \) is tributary width of the deck bracing a single girder.

\[ s_d = \frac{(s_g - b_f)(n-1)}{n} = \frac{(10 \times 12 - 24)(4-1)}{4} = 72 \text{ in} \]

\( L_b = 25 \text{ ft} \); \( C^*_b = 1.0 \); \( d = 192 \text{ in} \)

Here \( C^*_b M_{AASHTO(25 \text{ ft})} = 102807.9 \text{ K-ft} > 55654 \text{ K-ft} \)

It was observed that, for unbraced length \( (L_b) \) of 25 ft, \( C^*_b M_{AASHTO} \) is much greater than the factored dead load moment \( (M_{cr}) \); therefore stiffness calculations are not necessary and decking with relatively low stiffness will be able to brace these girders. This is a general case in most long-span, deep bridge girders, since the magnitudes of moments tend to increase with the depth of the individual girder. Metal deck forms with relatively low stiffness will be enough for bracing girders during construction. Here the distance between cross-frames is kept at 25 ft.
Bracing behavior of metal deck form/girder system with stiffening angle connection

To account for the effect of the stiffening angle, according to Egilmez (2007), the bracing of the stiffening angles will be approximated by using 50% of the buckling moment computed by using $L_b/2$ to evaluate the buckling capacity, where $L_b$ is the spacing between the cross-frames. Therefore, moment capacity equation becomes,

$$M_{cr} = \frac{C^*_b M_{AASHTO(L_b/2)}}{2} + mQd$$

Due to the application of loading on the top flange, the value of $m$ will be taken as 0.375.

Rearranging the equation to estimate the required shear stiffness, we get,

$$G'_{required} = \frac{4(M_{cr} - (1/2)C^*_b M_{AASHTO(L_b/2)})}{s_d md}$$

$s_d$ is tributary width of the deck bracing a single girder.

$$s_d = \frac{(s_e - b_f)(n-1)}{n} = \frac{(10\times12 - 24)(4-1)}{4} = 72 \text{ in}$$

$L_b = 50 \text{ ft}; C^*_b = 1.0; d=192 \text{ in}$

Here $(1/2)C^*_b M_{AASHTO(L_b/2)} = (1/2)C^*_b M_{AASHTO(50/2)} = 51404 \text{ K-ft}$

Therefore, to control deformations, the required shear stiffness of metal deck form system will be,

$$G'_{required} = \frac{4(55654\times12 - 51404\times12)}{72 \times 0.375 \times 192} = 39.35 \text{ K/in}$$

The stiffened metal deck form system with effective shear stiffness of 39.35 K/in provides enough bracing to girders to eliminate 8 cross frames along the 405 ft girder length. The distance between cross frames is now therefore equal to 50 ft, which is twice the original design of intermediate cross-frames spaced at 25 ft.