Deck Forms for Bridge Construction Safety

By

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UTCA

University Transportation Center for Alabama
The University of Alabama, The University of Alabama at Birmingham,
and The University of Alabama in Huntsville

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UTCA Theme: Management and Safety of Transportation Systems

University Transportation Center for Alabama

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16. Abstract

Because I-girders have inherently weak torsion resistance, cross-frames and/or diaphragms are placed 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 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 provide is not considered in construction-sequence engineering. Other researchers have investigated the stability that PMDF provides during the construction of bridges of moderate span length. 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 long-span deep steel plate 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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List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ALDOT	Alabama Department of Transportation
FEM	Finite Element Method
AISC	American Institute of Steel Construction
SDI	Steel Deck Institute
PMDF	Permanent Metal Deck Form
SSRC	Structural Stability Research Council
LRFD	Load and Resistance Factor Design

Executive Summary

Because I-girders have inherently weak torsion resistance, cross-frames and/or diaphragms are placed 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 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 provide is not considered in construction sequence engineering. Other researchers have investigated the stability that PMDF provides during the construction of moderate span length bridges. 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 long-span deep steel plate 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.

Section 1
Introduction

1.1 Research Overview

Lateral-torsional buckling, which can be roughly characterized as a phenomenon of sudden lateral displacement coupled with rotation of girder length that is not sufficiently braced, often controls the design of steel bridge girders during construction. 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 recent editions of the *AASHTO Standard Specifications for Highway Bridges* and *LRFD Bridge Design Specifications*, 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. The trend toward higher-strength steel will also tend to increase the likelihood of stability problems during construction. For these reasons, among others, alternative bracing systems and engineering methodologies are needed.

Deck forms are used to support wet concrete during construction of buildings and bridges. Before the concrete cures and composite action between the concrete slab and the steel girders is achieved, the girders alone 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 for lateral stability. However, AASHTO bridge specifications do not allow deck forms to be considered for providing stability.

The primary difference between the formwork used in the building construction and bridge construction industries are in 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 by 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 reported herein was 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 long-span plate girder steel girder bridges. Design calculations with improved connection details are presented that use 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 was 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 long-span steel plate girder bridges. Additional goals of the project include providing a systematic design procedure for considering a permanent metal deck form as bracing element, considering the effect of decking on vibration characteristics, and investigating opportunities for integrating improved construction stability practices.

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 finite element methods to investigate system behavior, and the dissemination of results. This study leverages recent testing and analytical research conducted by the University of Texas and the University of Houston, and a significant part of this report is essentially an orientation to the existing literature. This report will address safety aspects by investigating the stability during construction and the parameters necessary to understand the bracing behavior of permanent metal deck forms.

Section 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 out-of-plane displacement and twist of the beam cross-section. The 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 critical value. When the buckling moment is reached, lateral-torsional buckling is initiated by lateral deflection and 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 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 is 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.

$$M_{cr} = \frac{\pi}{L_b}\sqrt{EI_y GJ + \frac{\pi^2 E^2 I_y{}^2 h^2}{4L_b^2}}\ , \tag{2-1}$$

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 between flange centroids

AASHTO specifications provide the following expression for estimating the lateral-torsional bucking capacity of singly and doubly symmetric girder cross-sections:

$$M_{AASHTO} = 91\times10^6\left(\frac{I_{yc}}{L_b}\right)\sqrt{0.772\left(\frac{J}{I_{yc}}\right)+9.87\left(\frac{d}{L_b}\right)^2}\ , \tag{2-2}$$

where d is the depth of the girder. 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 as 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 toward increasing buckling resistance is somewhat linear and dependent upon the girder depth and is 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:

$$M_{cr} = M_{AASHTO} + Q\,d, \tag{2-3}$$

where Q is the shear rigidity of the metal deck form. The buckling load for cases with moment gradient are actually less than the buckling capacity for cases with uniform moment. To account for moment gradient effects, Helwig (1994) found that, instead of applying C_b to the entire expression, which produces unconservative results, the C_b factor should only be applied to M_{AASHTO}:

$$M_{cr} = C_b\, M_{AASHTO} + Q\,d\ , \tag{2-4}$$

where C_b is the moment gradient factor that accounts for the effects of moment gradient along the girder length and can be calculated using AISC (2005). In general, Helwig (1994) showed that the effectiveness of the deck is reduced when the girders are subjected to moment gradient.

Load height effects on doubly symmetric beams can be incorporated by modifying the C_b factor (Galambos 1998). The C_b value modified for the load height effects is 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:

$$M_{cr} = C_b^* M_{AASHTO} \tag{2-5}$$

This estimates the capacity of the girder without the metal deck form. The load height factor C_b^* is calculated as the ratio of C_b to B:

$$C_b^* = \frac{C_b}{B} \tag{2-6}$$

Two variables, A and B, are used to account for load height effects. The variable A is defined as traditional C_b value: 1.35 is used for a point load at midspan and 1.13 is used for a uniform distributed load. The variable B depends 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 following two expressions can be used 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 the beam parameter and is given by

$$W = \frac{\pi}{L}\sqrt{\frac{E\,C_W}{G\,J}}\,, \tag{2-7}$$

where

L = distance between discrete braces
E = modulus of elasticity
C_w = warping coefficient
G = shear modulus, and
J = torsional constant

This methodology provides 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 method similar to the equations for doubly symmetric sections would be useful for approximating load height effects in singly-symmetric girders. It is possible to check the accuracy of these equations on singly-symmetric sections. The method uses Equation 2-7 to calculate the beam parameter W. For singly symmetric sections, the warping term C_w is defined by:

$$C_w = I_y d^2 \rho (1 - \rho) \tag{2-8}$$

The variable ρ 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. A doubly symmetric section $\rho = 0.5$ results in a simpler expression: $C_w = I_y d^2/4$.

2.2 Metal Deck Forms Used in Building Construction Applications

In the building construction industry, 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 a planar system with a definite capacity to resist in-plane deformations caused by lateral loads. Previous studies have shown that permanent metal deck forms have substantial stiffness and strength in the plane of sheeting, which tends to provide significant bracing to the beams or girders to which they are attached. AASHTO specifications have historically required bracing in the form of cross frames or other discrete diaphragms at a maximum spacing of 25 feet. However, the 25-foot maximum spacing has been relaxed in recent AASHTO bridge specifications.

Although the building construction industry has long relied upon on the in-plane capacity of light metal sheeting, AASHTO does not permit PMDF to be utilized for bracing steel bridge girders, mainly due to the flexible connection details between the girders and deck forms. There are several other differences, however, in the type of metal deck forms that are used in the building construction 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 construction and bridge construction industries can be observed in the type and depth of deck profile, as well as the thickness of the sheet metal used to make the form. The PMDF used in the building construction 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 construction industry versus the bridge construction industry include the span length and shape of the deck panel, as well as the method by which it is attached to the girders. In the building construction 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 span over several beams. Figure 2-1 shows a typical PMDF panel arrangement for building construction 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.

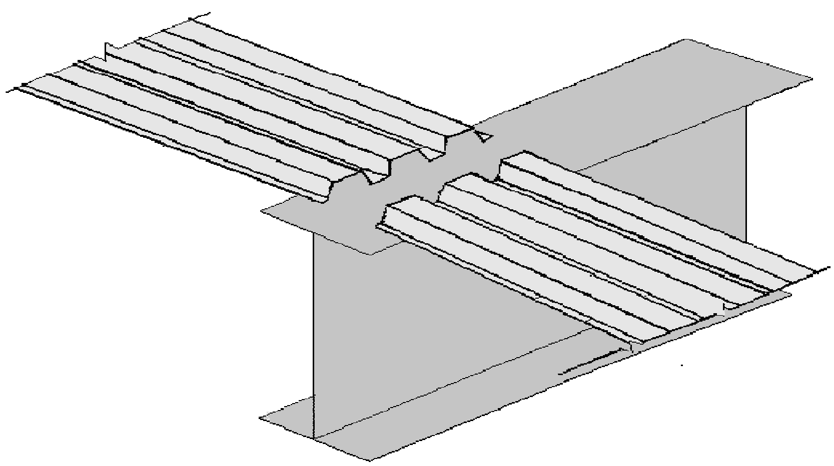

Figure 2-1. Metal deck form building application

2.3 Potential Bridge Application

The most significant difference between the use of PMDF in the building construction and bridge construction industries is in the connection details. Some building construction industry applications use a deck panel supported on all four sides, whereas there is only one arrangement of deck sheeting possible for bridge deck construction. Instead of running continuously over the top of girders at the same elevation, the steel deck must span between the bridge girders that are not typically at the same elevation. 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.

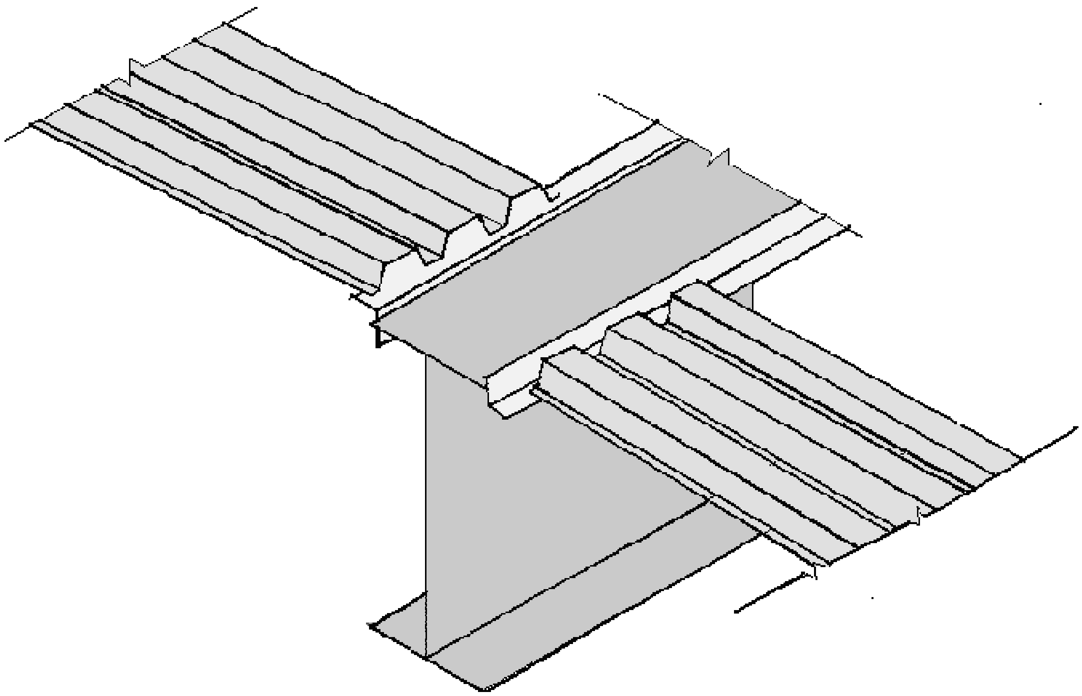

Figure 2-2. Metal deck form bridge application

Furthermore, 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 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 to provide 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 construction deck forms, in which the stiffness can be reduced due to the warping deformation of corrugations. 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 used in the building construction industry. The larger difference in the system stiffness is due to the connection details that are utilized in bridges.

Metal deck forms used in bridge applications 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, the eccentricity produced by this connection can substantially reduce the stiffness and strength of the deck form system.

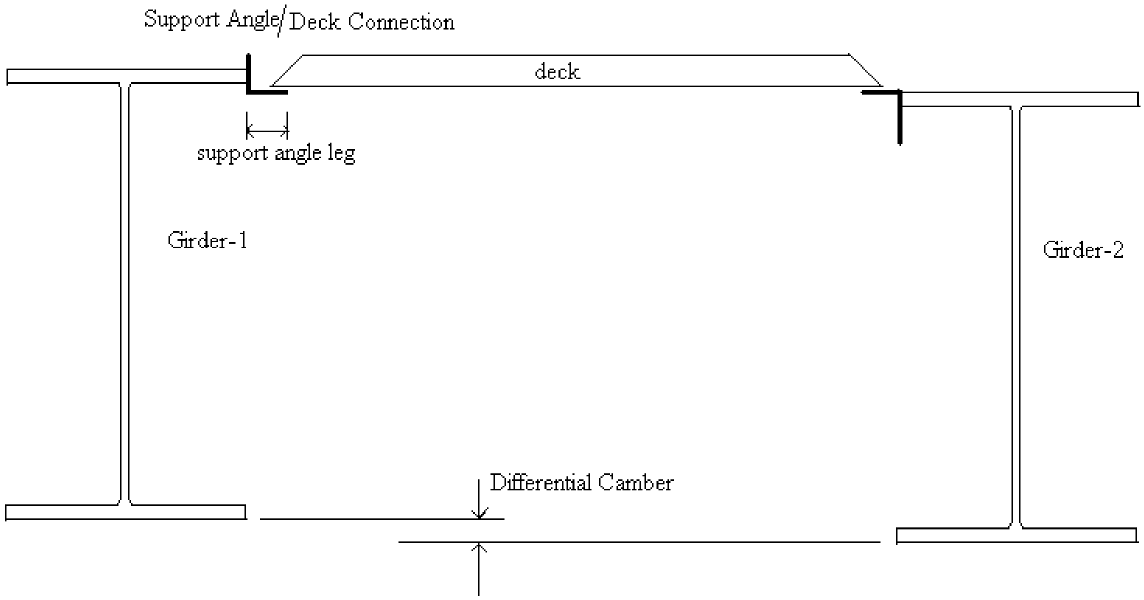

Figure 2-3. Typical bridge application with differential camber between adjacent girders

Although AASHTO does not allow permanent metal deck forms to be considered for bracing in steel bridge girders, design and construction engineers have an increased awareness of compression flange stability provided 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 a viable structural solution to spanning long distances. Therefore, in recent years, the use of long-span steel bridge girders has increased for 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 susceptible to instability 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. Wind load effects complicate not only design but also erection, which may be overlooked by contractors.

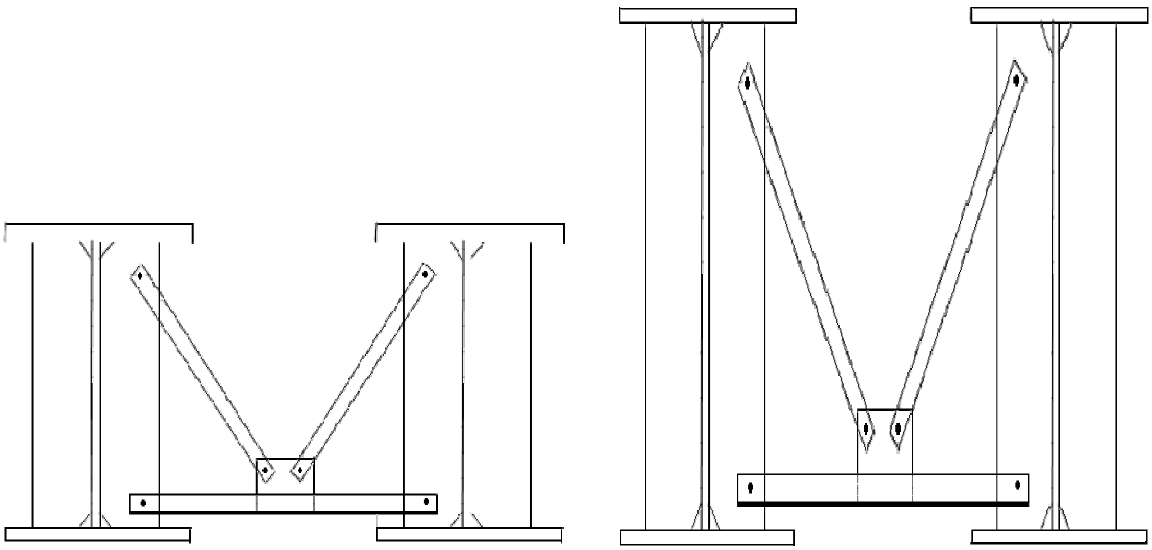

Figure 2-4. Short-span shallow and long-span deep bridge plate girder assembly

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 that 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 some other rational manner. Bracing members are placed to provide support against buckling. These members are assumed to be elastic and are therefore 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 become 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 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. 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 compression flange. For uniform moment, lateral bracing applied at the bottom flange of a simply supported beam is almost totally ineffective because the center of twist is located at a point near or outside of the tension flange. 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 cross-frames or diaphragms has been proven to be 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 moderate span bridge girders have been investigated by others. Previous studies have shown that permanent metal deck forms have substantial stiffness and strength in the plane of sheeting, which provides significant bracing to 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 investigated. Initially, work done on shallow, short-span bridge girders by prior researchers is reviewed. Design equations proposed by others are then applied to long-span, deep bridge plate girder stability by incorporating bracing contributions from metal deck forms. Finite element analyses are then used to check the applicability of these design equations to long-span, deep plate girder bridges.

2.5 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 material availability, plywood forms and concrete panels were the first choice to support wet concrete during construction. However, these forms have major drawbacks. Panels have limited spans (maximum 8 ft), which may reduce girder spacing and increase the number of girders required. Also, the task of removing temporary forms is difficult since the contractor must remove the forms from underneath the bridge. In areas where form removal is expensive or hazardous, the use of permanent deck forms is desirable. Several permanent deck form systems have been developed that eliminate the need for temporary form removal.

Precast concrete panels are one of the deck form systems that have been used over the past few decades. Although these panels are economical, there are major drawbacks, as these forms are very heavy and placement is labor intensive, usually requiring an external crane or trolley. Another major drawback is that, since the camber between adjacent girders differs significantly and 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 deck form system is the permanent metal deck form, 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 inch 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 used to attach the metal deck forms to girders.
The primary advantage of the 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 and 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. However, 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 polymer foam or other 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 described above. 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.

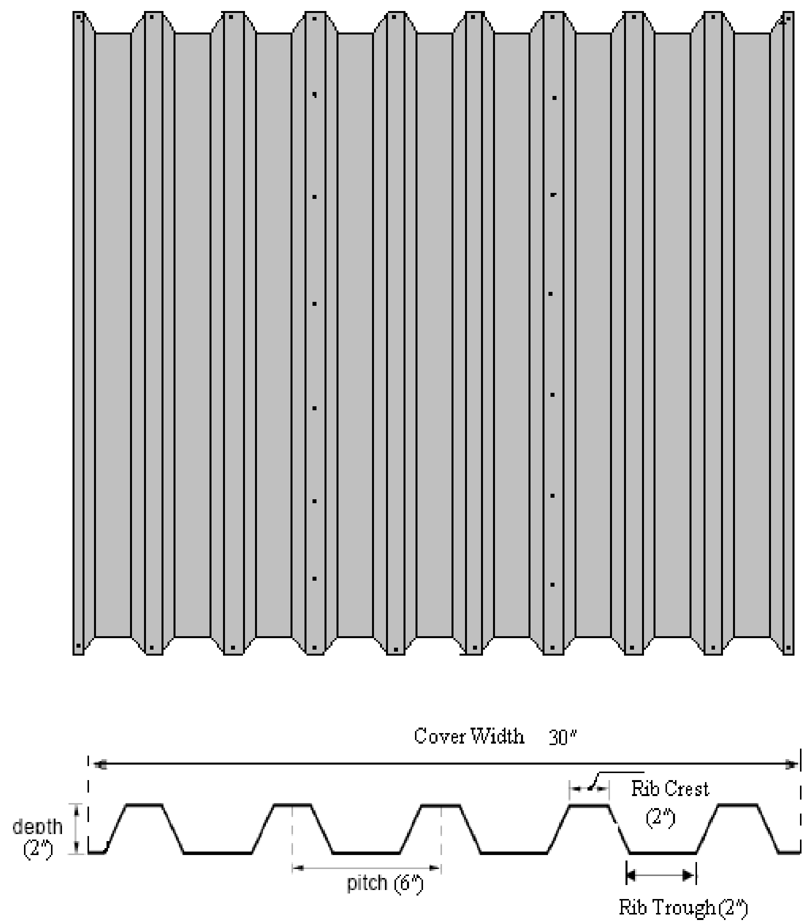

Figure 2-5. Metal deck form plan view and cross-section view

2.5.2 Deck Attachments

There are several methods of fastening PMDF to their supporting girders. The two used most commonly in the bridge industry today are welded connections and strap connections.

<u>2.5.2.1 Welded Connection Details</u> The configuration illustrated in Figure 2-6 is used when welding supporting angles directly to the top of the girder top flange is permitted. Once the support angles are welded to the girders, the deck panels can be fastened to the angles with end fasteners.

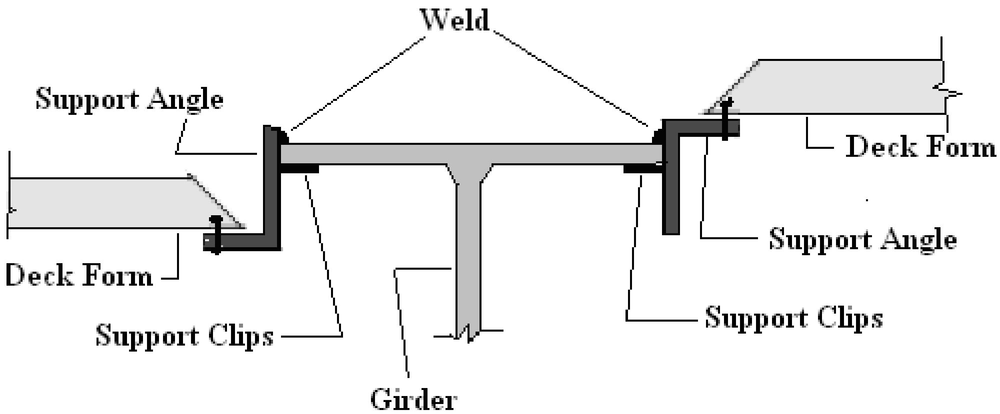

Figure 2-6. Typical welded connection detail in bridge industry

Welding directly to the top flange may not be permitted because of the potential fatigue problems in tensile stress regions. A minimum distance of ½ inch maintained between the end fastener centerline and both the deck end and the angle edge is typically required.

<u>2.5.2.2 Strap Connection Details</u> When welding to the girder is not allowed, a more complicated method of deck support angle attachment is used. This method consists of welding deck support angles to strap angles that are spaced at approximately one foot on center along the girder span (Figure 2-7). These strap angles are not welded to the girder; hold-down clips are used to prevent any uplift of the deck panels. The deck panels are then fastened to deck support angles.

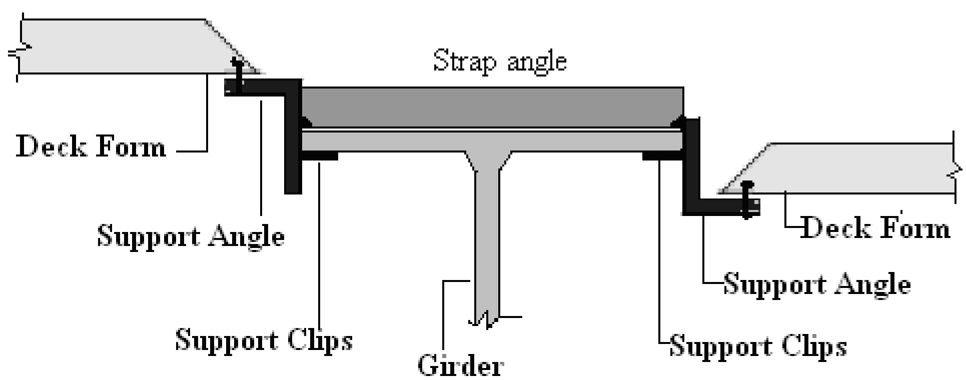

Figure 2-7. Typical strap angle connection detail in bridge industry

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

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

In shear tests performed at the University of Houston, PMDF showed a tendency to produce fields of tension and compression within the panel system as illustrated 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 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:

$$\frac{1}{\beta_{sys}} = \frac{1}{\beta_{deck}} + \frac{1}{\beta_{conn}} \qquad (2\text{-}9)$$

To determine the shear rigidity of the connection, the values of the normalized connection shear rigidities should be divided by half the span of the deck. The shear rigidity of the system, Q_{sys}, can then be calculated using

$$\frac{1}{Q_{sys}} = \frac{1}{Q_{deck}} + \frac{1}{Q_{conn}}, \qquad (2\text{-}10)$$

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}.

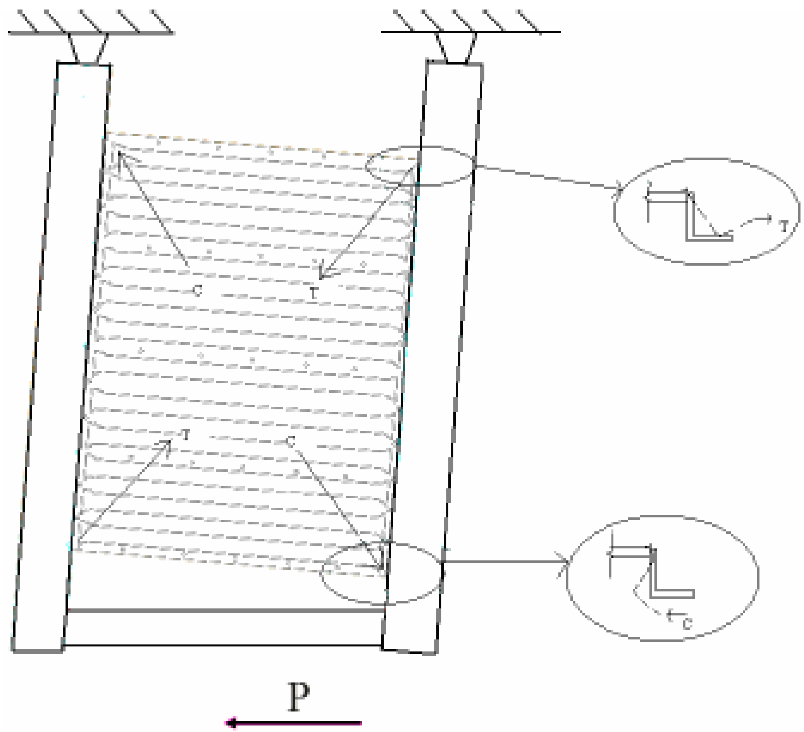

Figure 2-8. Fields of tension and compression within the panel system

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 Steel Diaphragm Design Manual design method (SDI 1995) 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 panels provide the greatest proportion of the shear resistance.

Fasteners required in the erection of permanent metal deck forms consist of end fasteners that connect the deck sheets to the girders and seam fasteners, referred to as side lap fasteners, that connect individual sheets together at 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 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 screws, or button-punched material.

Past studies have shown that fastener stiffness generally does not have much effect on overall stiffness. Using rigid fasteners increases the system stiffness by 5% to 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 that the magnitude of fastener forces becomes approximately half the corresponding values for the unstiffened PMDF systems.

Presently, self-drilling screws are the dominant method of attachment of bridge deck forms for both end and side-lap fastening. Equations 2-11 and 2-12 define the flexibility of the screws for the deck to support angle flexibility, S_f, and deck-to-deck (side-lap) flexibility, S_s, respectively (SDI 1995):

$$S_f = 0.0013/t^{0.5} \quad \text{(in/kip)} \tag{2-11}$$

$$S_s = 0.003/t^{0.5} \quad \text{(in/kip)} \tag{2-12}$$

The background and importance of each component of the bridge superstructure and substructure is explained in subsequent sections. Research done on shallow, short-span bridge girders by others is presented in Section 3. Then design methodology proposed by others is applied to long-span, deep plate girder stability by incorporating bracing contributions from metal deck forms.

Section 3
Literature Synthesis

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 construction and bridge construction industries to support wet concrete during construction. A considerable amount of work on shear diaphragm action was done during the 1960s and 1970s.

3.1.1 Buildings

Nilson*, et al.* (1960) laid the foundation of the work by 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 was extended by Lutrell and Apparao (1969), who investigated the effect of panel configuration, material properties, span length, and particularly the method of fastening the diaphragm. Lutrell and Apparao (1969) developed a semi-empirical formula for estimating the shear stiffness of standard corrugated panels and noted that shear stiffness was mainly dependent upon 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 diaphragms used in the building construction industry. 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 finite element models of shear diaphragms to calculate the effective shear modulus and strength and compared the results to experimental data. Plane stress 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 lightweight diaphragms. He considered the different modes of failure and assumed a suitable distribution of internal forces within the diaphragm 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 at that time. The results were verified by comparisons 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 depends on 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's study was to determine the shear stiffness of permanent metal deck form panels without any effects from the supports used to attach the forms to the girders. Currah 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 connected using strap angles that saddle the top flanges. Currah noted that both connection details can introduce an eccentricity in the transfer of the loads from the permanent metal deck forms to the top flanges. Currah concluded that the connection stiffness was a controlling factor in the stiffness of the metal deck form diaphragm system and that the flexibility of the supporting angles should be carefully considered if permanent decking is to be considered as a lateral bracing 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 explained that shear strength is controlled by a combination of one or more failure modes. Currah 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.

Soderberg (1994) continued the work of Currah by further investigating the connection stiffness of permanent metal deck forms and ways to improve the connection. He proposed a modified strap connection detail that showed improved connection stiffness but required significant fabrication and placement efforts. Throughout the testing, various modifications were made to the diaphragm system to determine their effects on the system response to lateral and vertical loads. The responses of the system were compared to analytical results to determine the diaphragm stiffness. Three sets of experimental tests were conducted. The 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 used the shear frame test set-up that was constructed and used by Currah (1993) to determine the diaphragm shear stiffness improvements provided by improved connection details. Also, strength and ductility issues were addressed concerning the improved strap detail. The "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. 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 that the buckling capacity of the twin girder system agrees with Helwig's bracing formula.

Helwig (1994) studied the lateral bracing ability of permanent metal deck forms commonly used in steel bridge construction. It was noted 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 also noted that permanent metal deck forms provide continuous bracing against lateral movement along the girder, thus behaving as a shear diaphragm. Rigorous FEM analyses were conducted 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 results were compared to existing solutions for beams braced by shear diaphragms, which were then used to develop a design approach for single-span and continuous girders braced by 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 the 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 the C_b equation to account for the effect of the reverse-curvature bending for singly symmetric sections. Helwig and Frank 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 center of gravity of the beam.

Texas researchers continued the work of Helwig (1994) that studied the lateral bracing ability of permanent metal deck forms in steel plate girder bridges. These investigations were particularly focused on improving the connection detail between the top girder flanges with the permanent metal deck forms and proposed improved connection details that involve a transverse stiffening angle that spans between adjacent girders to control the support angle deformation. The effects of parameters—such as the metal gage of the deck forms, span of the forms, connection details, and panel aspect ratio—were 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 that the deck could be fastened directly to the angle that spans between adjacent girders. While the main purpose of the stiffening angle is to control deformation of the support angles, it also provides deck support at the end of the panel. The PMDF/stiffening angle assembly is shown in Figure 3-1.

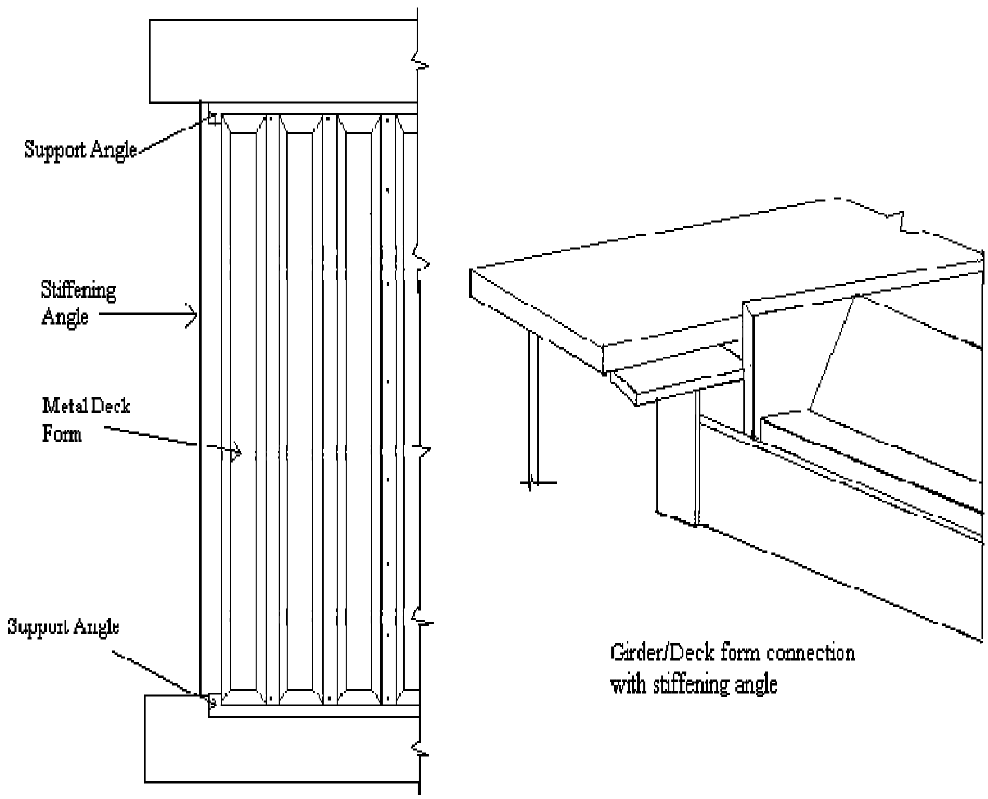

Figure 3-1. Girder/metal deck form connection with stiffening angle

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.

3.2 Permanent Metal Deck Form Characteristics

Before making use of shear diaphragms in structural design, it is necessary for the designer to have knowledge of stiffness and ultimate strength characteristics. These quantities can be estimated using testing, approximate methods, and FEM analyses. Considerable progress has been made recently in developing methods to predict the two important parameters that characterize a diaphragm assembly: shear stiffness and shear strength. Tabulated values for specific connection assemblies are given in the references listed in the reference section and in literatures provided by deck panel manufacturers. As an alternative, these characteristics can be determined from the load deflection curves 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 structures, so it is important to have a clear understanding and knowledge of in-plane shear strength, shear stiffness, and system reliability. 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 upon connections between panels along their edges and the connections from panels to supporting structural members. Panel shape, structure dimensions, material thicknesses, and type of connections are factors that affect the system's 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 diaphragms, the shear rigidity, Q, which is in units of force per unit radian (KN/rad or kip/rad), is the most important parameter. Shear rigidity is calculated as the product of effective shear modulus (G') and tributary width of the deck (S_d). Effective shear modulus can be calculated using the Equation 3-1.

$$G' = \frac{\tau'}{\gamma}, \tag{3-1}$$

where G' = effective shear modulus, τ' = effective shear stress, and γ = shear strain.

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:

$$S_d = (S_g - b_f)(n-1)/n, \tag{3-2}$$

where n = number of girders in the system, S_g = spacing between girders, and b_f = width of girder top flange. Then shear rigidity of the permanent metal deck form can be calculated using

$$Q = G' \cdot S_d \tag{3-3}$$

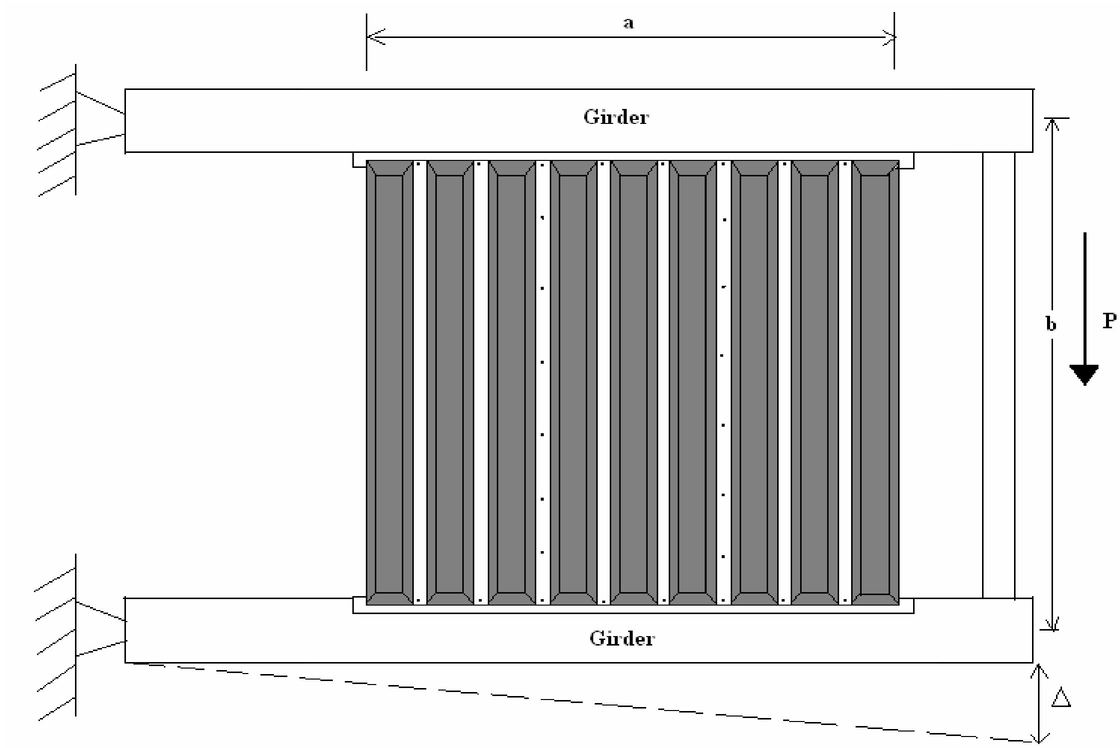

Figure 3-2. Shear stiffness determination of girder/metal deck form system

The building construction 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. SDI (1995) provides 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 is 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 must be utilized that is not dependent on metal thickness. Previous studies showed that the 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 members in every trough exhibits greater stiffness than panels in every other trough.
- There will be an increase in stiffness if additional seam fasteners that attach adjacent deck sheets together in deck panel are provided.
- 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 as shown in Figure 3-2. When the deck system reaches its ultimate capacity, the applied load 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:

$$S'_{ult} = (P_{ult} L) / f_w \tag{3-4}$$

The bridge decks are fastened to supporting members at the ends of the deck sheets. 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 the 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, and type of connections affect the diaphragm shear strength and stiffness of the system. The strength of a diaphragm is also governed by force transfer at the interior panel connection and the fastener across the ends of the panels.

3.3 Metal Deck Form Contribution to Bridge Girder Bracing

Cold-formed steel panels are often 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 illustrates the possible modes of failure of beams with diaphragm bracing at the compression flanges. In Figure 3-3a, the diaphragm rigidity and strength are not adequate to prevent lateral buckling of the beams. In Figure 3-3b, 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.

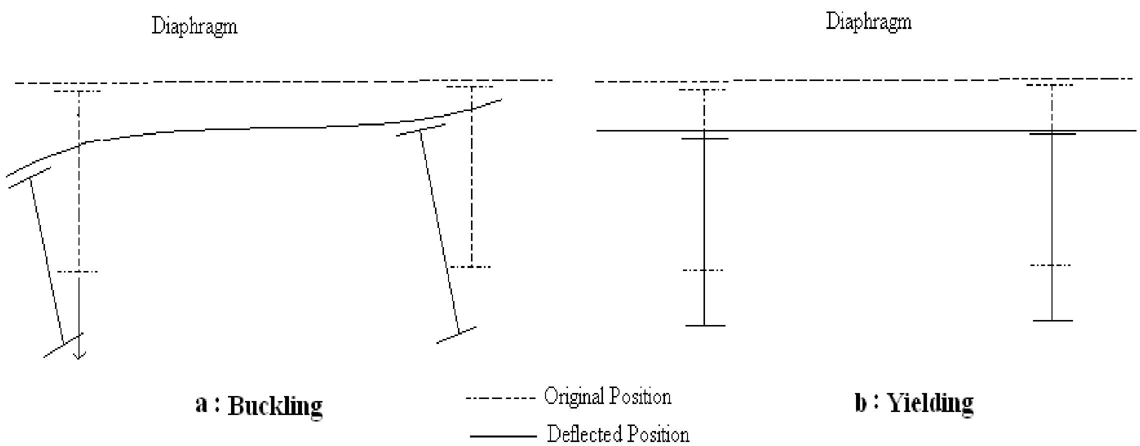

Figure 3-3. Girder failure modes when braced with diaphragm on compression flanges

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, studies by the building construction industry have demonstrated that metal forms can significantly increase the buckling capacity of beams. 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. The development assumes that lateral displacement and twist of the cross-section along the girder length are in the form of a sine curve.

$$M_{cr} = \sqrt{\left(\frac{\pi^2 EI_y}{L^2} + Q\right)\left(\frac{\pi^2 EC_w}{L^2} + GJ + Qe^2\right)} + Qe\,, \tag{3-5}$$

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-form solution results with FEM results and showed that the difference is less than 2%.

For doubly symmetric sections, the shear center and center of gravity are both located at mid-height of the cross-section. To account for the capacity of singly symmetric sections, the Errera

(1976) solution was modified and is applicable to both doubly and singly symmetric sections, as shown in Equation 3-6:

$$M_{cr} = \sqrt{(M_{AASHTO})^2 + \left[2Qe^2 \frac{\pi^2 E(2I_{yc})}{L_b^2} + QGJ + Q^2e^2\right]} + Qe \tag{3-6}$$

M_{AASHTO} is given by AASHTO specifications for estimating lateral-torsional bucking capacity of singly and doubly symmetric girder cross-sections and can be calculated using Equation 3-7:

$$M_{AASHTO} = 91\times10^6 \left(\frac{I_{YC}}{L_b}\right)\sqrt{0.772\left(\frac{J}{I_{yc}}\right) + 9.87\left(\frac{d}{L_b}\right)^2}\,, \tag{3-7}$$

where J = torsional constant and d = girder depth.

Helwig (1994) compared the finite element results with solutions presented by Equation 3-7 and found that e will provide good 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) 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 Equation 3-8:

$$M_{cr} = M_{AASHTO} + 2Qe\,, \tag{3-8}$$

where M_{AASHTO} is the buckling capacity of the girder with no deck for bracing and Q and e have been previously defined. Therefore, using 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. 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 may be 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 girders 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 the traditional C_b value to the entire Errera expression and presented Equation 3-9 for a beam braced by a shear diaphragm subjected to a moment gradient:

$$M_{cr} = C_b(M_{AASHTO} + 2Qe) \tag{3-9}$$

Helwig (1994) found that applying C_b to the entire modified solution is not conservative and that Equation 3-9 does not estimate the buckling capacity accurately due to drastic changes in the buckled shape resulting from the restraint effects provided by the deck. It was shown that using the C_b factor only on the girder capacity was the most logical design approach and that Equation 3-10 properly accounts for the moment gradient.

$$M_{cr} = C_b M_{AASHTO} + Qd \tag{3-10}$$

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, which assumes that the twist and lateral displacement of the beam follows a sine curve along the beam.

$$M_{cr} = C_b d\left[\frac{-P_e g}{2} + \frac{Q(1-g)}{2} + \sqrt{\left(\frac{-P_e g}{2} + \frac{Q(1-g)}{2}\right)^2 - \frac{Q^2}{4} + \left(\frac{P_e}{2} + \frac{Q}{2}\right)\left(\frac{P_e}{2} + 2P_T + \frac{Q}{2}\right)}\right], \tag{3-11}$$

where

- d = depth of the girders
- P_e = weak axis Euler load = ($\pi^2 EI/L^2$)
- C_b = moment gradient factor
- G = load height factor
- P_T = GJ/d^2
- G = shear modulus of beam material
- J = torsional constant of the beam

The C_b factor accounts for moment gradients, while g accounts for load height effects. Lawson and Nethercot (1985) recommended using traditional C_b and g values. The AISC design specification uses Equation 3-12 for the C_b of a girder buckling between points of full bracing (cross-frames):

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_2 + 4M_{cl} + 3M_4}, \tag{3-12}$$

where

- M_{max} = maximum moment between full braces
- M_2 = moment at the quarter point between full braces

M_{cl} = moment at the midway between full braces
M_4 = moment at the three quarters point between full braces

For a point load at the midspan top flange the value of g is 0.55, while for a distributed load it is 0.45. As energy-based solutions 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 assumed 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 Equation 3-13 to approximate the ideal stiffness:

$$M_{cr} = C_b^* M_{AASHTO} + mQd \,, \tag{3-13}$$

where C_b^* = moment gradient factor that considers load height effects and d = depth of the cross-section. M_{cr}, M_{AASHTO}, and Q are as defined in Equations 3-5 and 3-6. The term mQd represents the contribution from the PMDF/support angle connection. The component representing the deck in Equation 3-13 is a function of the girder depth and the deck shear rigidity, and the constant m depends on the type of loading, the presence of intermediate bracing, and the web slenderness.

Helwig and Frank (1999) demonstrated that $e = d/2$ is a good approximation for both singly and doubly symmetric girders and that $m = 1.0$ should be used for uniform moment. 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.

Table 3-1. Design Values of *m*

Web Slenderness	Top Flange Loading w/o Midspan Torsional Brace Helwig and Frank (1999)	Top Flange Loading with Midspan Torsional Brace Helwig and Yura (2003)
$h/t_w<60$	0.5	0.85
$h/t_w>60$	0.375	0.64

Web buckling must 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 limit 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:

$$G'_{ideal} = (M_{cr} - C_b^* M_{AASHTO}) / (s_d \, md) \tag{3-14}$$

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

$$G'_{req'd} = 4(M_{cr} - C_b^* M_{AASHTO}) \big/ (s_d\, md) \tag{3-15}$$

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 the PMDF System

Egilmez (2007) continued the work of Helwig (1994) and tested a number of modified connection details developed to control the support angle deformation. However, the concept 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 side-lap 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 connection detail modification, Egilmez (2007) performed laboratory tests and finite element modeling. 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 developed 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 a 20-gage deck, the difference in the lateral stiffness between the unstiffened and stiffened systems ranged between 2 kip/in and 5.3 kip/in. This increment in the lateral stiffness due to the stiffening angles can provide a significant increase in buckling capacity; however, accounting for 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 stiffness contributed by a stiffening angle, Egilmez (2007) modified the expression by Helwig (1994). Equation 3-14 was developed for girders braced by a shear diaphragm fastened on only two sides (i.e., at the support angle). The stability bracing contributed by the stiffening angles provide a different type of bracing than the restraint from the PMDF connection through the support angle. Since these systems involve panels 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 similar to a relative bracing system. As a simple estimate, the bracing of the stiffening angles will be approximated using 50% of the buckling moment computed with $L_b/2$ 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 provide significantly higher bracing. Therefore, Equation 3-14 becomes:

$$M_{cr} = \frac{C_b^* M_{AASHTO(L_b/2)}}{2} + mQd \tag{3-16}$$

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 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):

$$M'_{br} = k\frac{(M_{cr}L)}{d^2}, \tag{3-17}$$

where L = total beam span and d = 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. The 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 Equation 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 as high because deck contributions to bracing comes from both shear and 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 girders with top flange bracing was conducted by Yura (1995). Equation 3-18 was presented for girders with a top flange braced continuously and transverse loading applied at the top flange:

$$C_b = 2.25 - \frac{1}{2}\left(\frac{M_1}{M_0}\right) + \frac{2M_{cl}}{\left(\mathrm{M}_0 + \mathrm{M}_1^*\right)} \tag{3-18}$$

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 characteristics of the PMDF/diaphragm system depend on the spacing and number of side-lap and end fasteners. The shear stiffness of PMDF systems 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. This experimental study also investigated the effect of deck panel width on the shear characteristics of PMDF/girder systems. 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.

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 span, deep bridge girders. This first phase of the project largely leverages recent stability research conducted by others, but with interest toward application to very long span, deep bridge girders. Next phases of the work focus on the use of FEM to validate application to very long span, 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.

Section 4
Proposed Design Methodology

4.1 Overview

The previous chapters provide a thorough background review of relevant 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 parametric 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 these parametric studies was to maintain a conservative model; expressions developed for estimating the stiffness and strength requirements for 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-span deep bridge girders against lateral loading (wind) and other construction load considerations. The appendix presents design calculations that demonstrate the use of these modified expressions in existing bridge design examples. Although a girder/deck forms system with new, modified stiffened connection details showed impressive results, it was not the objective of this study to understand the effect of stiffened connection details for long-span deep plate girder bridges. For the purpose of understanding, design methods that consider stiffening angle contribution to 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:

$$G'_{ideal} = \frac{(M_{cr} - M_{recommended})}{s_d md} \tag{4-1}$$

$M_{recommended}$ can be taken as follows:

Case 1: Without stiffening angle (Helwig 1994)

$$M_{recommended} = C_b^* M_{AASHTO}$$

Case 2: With stiffening angle (Egilmez 2007)

$$M_{recommended} = (1/2)\, C_b^* M_{AASHTO(L_b/2)}$$

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'_{ideal} = ideal deck stiffness, M_{cr} = maximum design moment, S_d = tributary width of deck bracing a single girder, and M_{AASHTO} = the buckling capacity of the girder using half the spacing between cross-frames, C_b^*, *m*, and *d* have been defined. 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 strength requirements. Equation 4-2 was presented by Egilmez (2007) for estimating the required bracing moment per unit length of a girder for a diaphragm with stiffness is given by

$$M'_{br} = k\frac{(M_{cr}L)}{d^2}, \tag{4-2}$$

where L = total beam span and d = beam depth. The brace moment represents the warping restraint provided to the top flange of the girder per unit length of the span. 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 results in 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

Web Slenderness	Top Flange Loading w/o Midspan Torsional Brace
$h/t_w < 60$	0.00015
$h/t_w > 60$	-

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

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. Bend-buckling causes an increase in the stress in the compression flange. The web bend-buckling capacity can be calculated using the Equation 4-3:

$$M_r = \left[\frac{\lambda^2}{\left(\frac{D_c}{t_w} \right)^2} \right] S_{xc} \tag{4-3}$$

This equation is used to solve for the maximum moment allowed to prevent bend-buckling. The parameter λ 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.

Shear buckling capacity is defined by AASHTO specifications and can be checked using the following equation:

$$V_u = 0.5 D t_w F_y \tag{4-4}$$

where F_y = yield capacity of the web material, t_w = web thickness, and D = girder depth. As web buckling is caused by shearing stresses or bending stresses, closely spaced stiffeners and a thick web are used to control web buckling.

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 upon to support the entire construction load. The construction load consists of the weight of the steel girder, fresh concrete, screed, 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. Therefore, the total applied load that the girders must be able to support 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 to 10 lb/ft^2). This entire load must be carried by the steel section alone. These loads and length are used to calculate the required girder erection moment or the moment that girder must support. The ability of the girder to carry this moment with no intermediate braces must be checked using Timoshenko's solution or the AASHTO specification equation (i.e., check $C_b^* M_{AASHTO} > M_e$). Timoshenko's solution for lateral-torsional buckling capacity is explained briefly in Chapter 3 and is not applicable for singly symmetric sections since it produces conservative estimates when the larger flange is in compression but not when the flange is in compression. Therefore, the capacity of the girder must be calculated using AASHTO specifications for lateral-torsional buckling:

$$M_{AASHTO} = 91 \times 10^6 \left(\frac{I_{yc}}{L_b} \right) \sqrt{0.772 \left(\frac{J}{I_{yc}} \right) + 9.87 \left(\frac{d}{L_b} \right)^2} \qquad (4\text{-}5)$$

STEP 3

Moment Capacity Equations:

1. **Girders braced without contribution of deck forms**

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:

$$M_{cr} = C_b^* M_{AASHTO} \qquad (4\text{-}6)$$

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:

$$C_b^* = \frac{C_b}{B} \tag{4-7}$$

C_b accounts for moment gradient along the girder length and can be calculated using AISC specifications. The variable *B* can be calculated using expressions provided in Chapter 3.

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):

$$M_{cr} = C_b^* M_{AASHTO} + mQd \,, \tag{4-8}$$

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, the bracing of the stiffening angles should be approximated by using 50% of the buckling moment computed using $L_b/2$, where L_b is the spacing between the cross-frames (Egilmez 2007). This approach provides a conservative estimate for most problems because the stiffening angles provide significantly higher bracing, and it also gives a simple solution that can be compared to the FEA solutions. Therefore, Equation 3-13 becomes:

$$M_{cr} = \frac{C_b^* M_{AASHTO(L_b/2)}}{2} + mQd \tag{4-9}$$

<u>STEP 4</u>

For cases 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.

I. 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'_{ideal})

The tributary width of the deck bracing a single girder, S_d, will be equal to a clear span of the PMDF per girder and can be found by:

$$S_d = \frac{\left(S_g - t_f\right)\left(n_g - 1\right)}{n_g} \tag{4-10}$$

Ideal deck shear stiffness (G'_{ideal}) can be estimated using the following expression:

$$G'_{ideal} = \frac{(M_u - M_{recommended})}{s_d md} \tag{4-11}$$

$M_{recommended}$ can be taken as follows:

Case 1: Without stiffening angle: $C_b^* M_{AASHTO}$

Case 2: With stiffening angle: $(1/2)C_b^* M_{AASHTO(L_b/2)}$

The bridge example provided in Appendix C illustrates the design calculation for both cases, and results are 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.

$$G'_{req'd} = 4G'_{ideal} \tag{4-12}$$

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 (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:

$$M'_{reqd} = k\frac{(M_u L)}{d^2} \tag{4-13}$$

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

II. When shear rigidity of the deck system is known

When the shear rigidity of the deck system is known, the design load is calculated 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, the recommended AASHTO girder moment capacity equation is added to the moment contribution provided by the shear diaphragm bracing. This additional moment contribution from shear diaphragm bracing represents the deck form system rigidity, i.e., $mQ_{sys}d$. The metal deck form system stiffness is comprised of the combined deck and connection stiffness.

Deck Shear Rigidity

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

$$Q_{deck} = \frac{G' \times (deckspan)}{2} \tag{4-14}$$

This is similar to an actual design in which the effective shear modulus G' would be calculated using the formulas provided in the SDI manual, procedure developed by Currah, or deck manufacturers recommended values.

Connection Shear Stiffness

To determine the shear rigidity of the connection, the values of the normalized connection shear rigidities should be divided by half span length of the deck:

$$Q_{connection} = \frac{Q_{recommended}}{(span/2)} \tag{4-15}$$

The recommended connection stiffness for stiffened and unstiffened strap connection details can be estimated with FEM or the method proposed by Currah (1993) and Soderberg (1994). 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 deck system stiffness is comprised of the deck stiffness and the connection stiffness, combining the deck and connection stiffness to determine the recommended stiffness of the diaphragm system is given by:

$$\frac{1}{Q_{sys}} = \frac{1}{Q_{deck}} + \frac{1}{Q_{conn}} \tag{4-16}$$

Where Q_{deck} = shear rigidity of the deck and 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}. For different loading conditions, the resulting increase in buckling capacity provided by deck system bracing can be calculated using $mQ_{sys}d$. 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 provided in Appendix C.

Section 5
Finite Element Analysis Technique and Results

5.1 Executive Summary and Introduction

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 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 allowed 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. Existing closed form solutions of prior researchers for "fully braced" beams were used to check the accuracy of the 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 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 results 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.

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 in Chapter 4.

Demonstration of these design equations on an actual bridge example that consists of very long-span, deep girders is 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 study was to compare and correlate the shear rigidity of the girder system with and without metal deck forms to those calculated using the SDI manual (SDI 1995). 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 one of the goals of this project was to study the effect of metal deck forms on vibration characteristics of the system, dynamic FEM analyses were also conducted.

The study is focused on creating a modeling method for both single-span and two-span continuous bridges. Parameters such as cross-frames, 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 FEM results.

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 diaphragm design manual, the lateral deflection of the PMDF system can be calculated as follows:

$$\Delta = \frac{P}{G'}\frac{H}{B}, \tag{5-1}$$

where

P = axial force

H = panel width

B = panel length

G'_{sys} = PMDF system shear stiffness

Detailed finite element models of steel plate girder bridges were created using the finite element analysis program ANSYS, a powerful commercially available software package for both computer-aided design (CAD) and FEM analysis. Linear static analyses were performed to determine the displacements, stresses, strains, and forces that result from static loading. Dynamic modal analyses were 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 of 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 detailed bridge components, such as 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). 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 points that define the shape of the structure (keypoints) (Figure 5-1). The positions of the keypoints were obtained from 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.

A single-span model was initially created. 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 shown in Figure 5-1. For the case of the single-span girder, deflections based on linear 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 full bridge models.

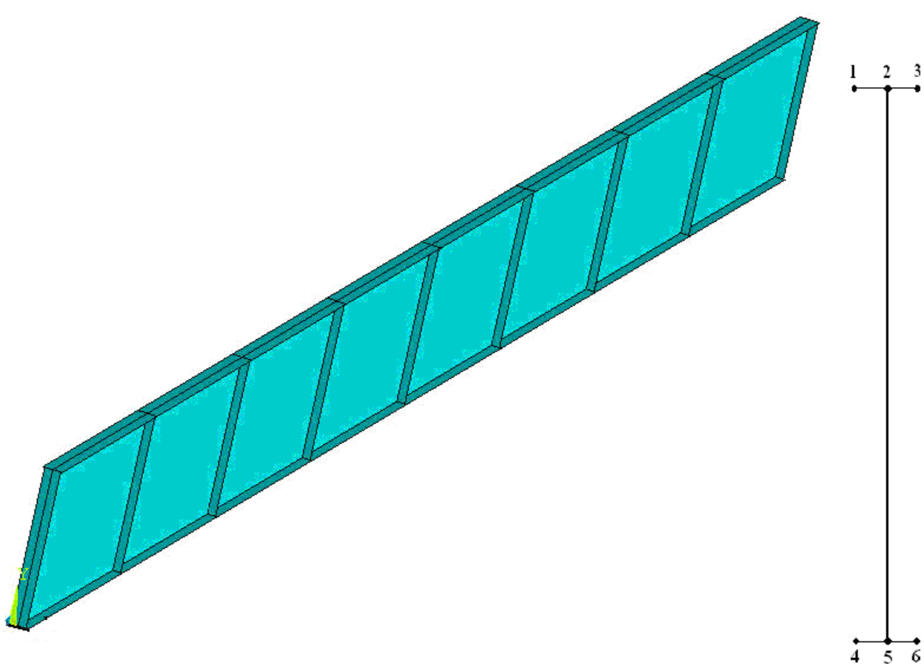

Figure 5-1. Single Girder Model

5.3 Stiffeners and Supporting Angles

Bridge plate girders typically include bearing stiffeners, intermediate web stiffeners, and supporting angles. Bearing stiffeners 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 screws. The bearing stiffeners, intermediate web stiffeners, and supporting angles were 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 were attained from the bridge construction plans and applied appropriately during the FEM. Web stiffening plates and supporting angles were modeled using four-node SHELL93 elements. All web stiffeners were spaced at 25 ft increments. In the actual bridge being modeled, these plates are fully welded along the height of the web and 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.

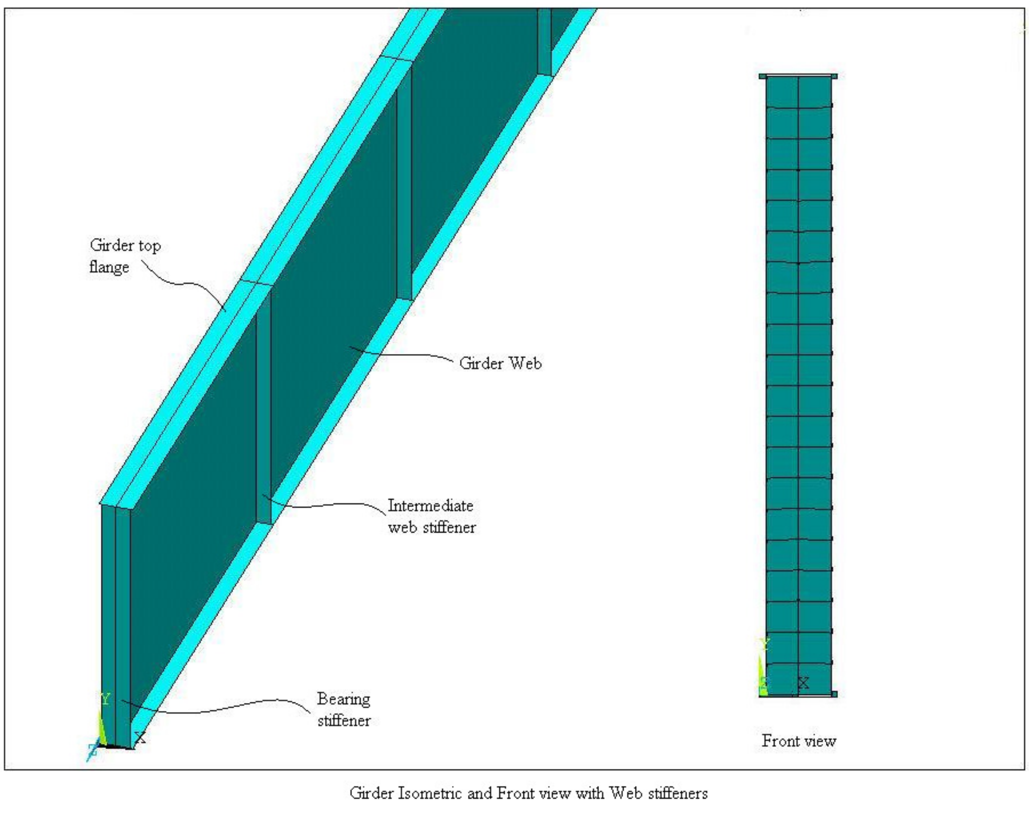

Figure 5-2. Oblique and front view of girder with stiffeners

Although there were difficulties in modeling the 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 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 issue was overcome using constant widths of the stiffener plates, which allowed the girder cross-section to deflect/rotate, and it 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 the mesh with bearing and web stiffeners modeled using this approach.

5.4 Permanent Metal Deck Forms

Based on construction loading and the SDI 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 the SDI design specifications, and these values were cross-checked with the manufacturer's values.

Helwig (1994) and Egilmez (2007) performed comprehensive analytical studies that involved 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 determine the correct cross-sectional areas. Therefore, instead of the modeling method implemented by Egilmez (2007), a three-dimensional metal deck form model was created using the four-node SHELL93 element (Figure 5-3). The cross-section of metal deck forms was modeled using keypoints and then connected by lines.

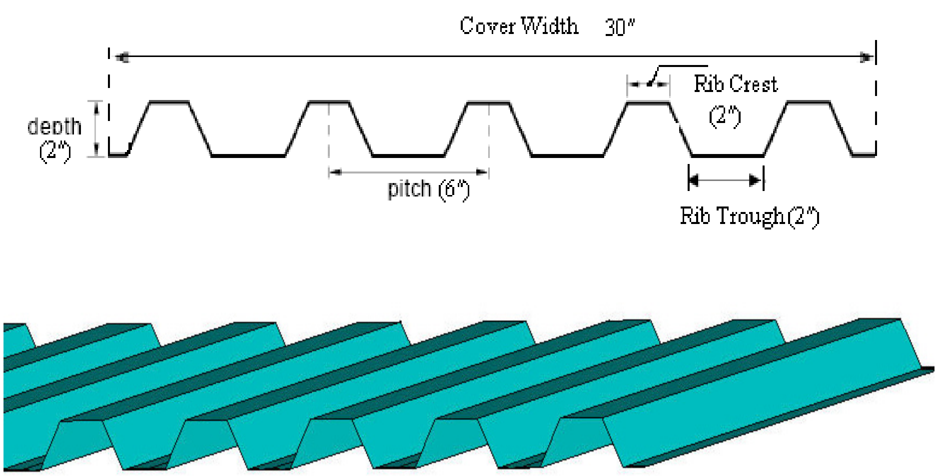

Figure 5-3. 3D Metal deck form model

Because a typical bridge uses metal deck forms that rest directly on supporting angles, modeling of this connection detail proved tedious. Therefore, to facilitate modeling, 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 angles and metal deck forms because of bending and membrane capabilities that include shearing deformation.

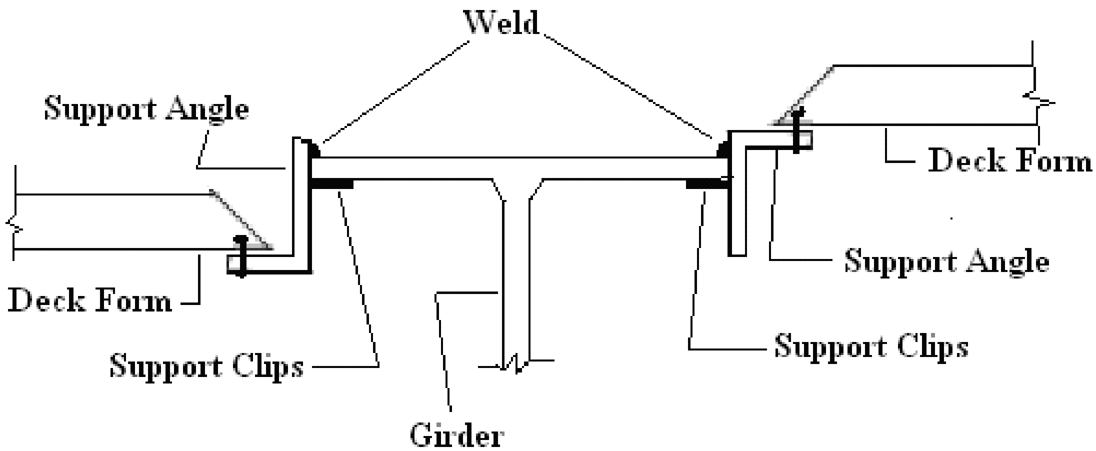

Figure 5-4. Typical girder connection detail

For proper behavior of the support angle/metal deck form connection, precaution was taken so that the metal deck forms share nodes with the supporting angle nodes. 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.

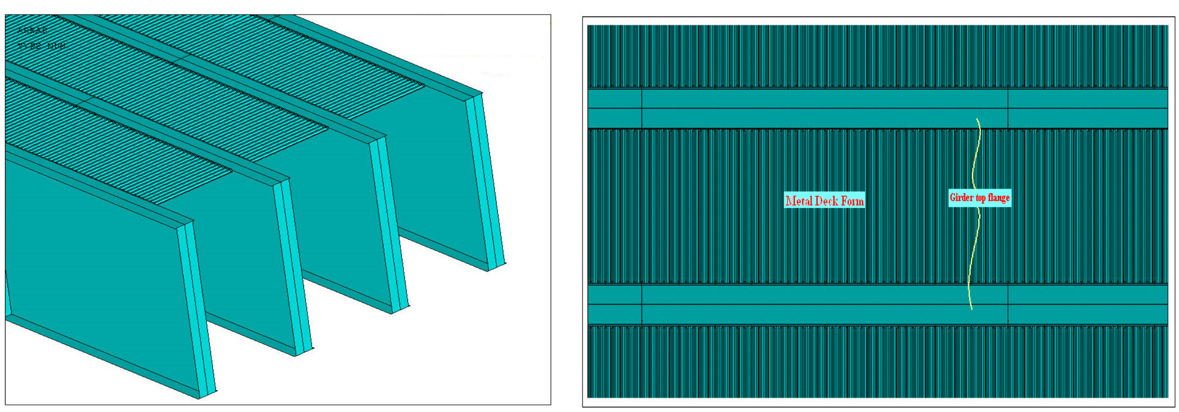

Figure 5-5. Isometric and plan view of Girder/PMDF system model

5.5 Cross-Frames and Diaphragms

Diaphragms and cross-frames are necessary for all I-girder bridges. According to AASHTO specifications, the cross-frames must transfer lateral wind loads from the bottom of the girder to the deck and bearings, support the bottom flange in negative moment regions, and stabilize the top flange before the deck has cured. 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. K-type cross-frame bracing was used in the studied bridge and is illustrated in Figure 5-6.

Initially these cross-frames were modeled with truss elements so that each member of the cross-frame represents a single element. However, the cross-frames modeled in this manner resulted in overly stiff results. Subsequently, each cross-frame member was modeled with beam elements so that flexural action would be simulated in addition to axial resistance. The cross-frame member section properties were computed using the AISC Manual of Steel Construction and implemented into the model.

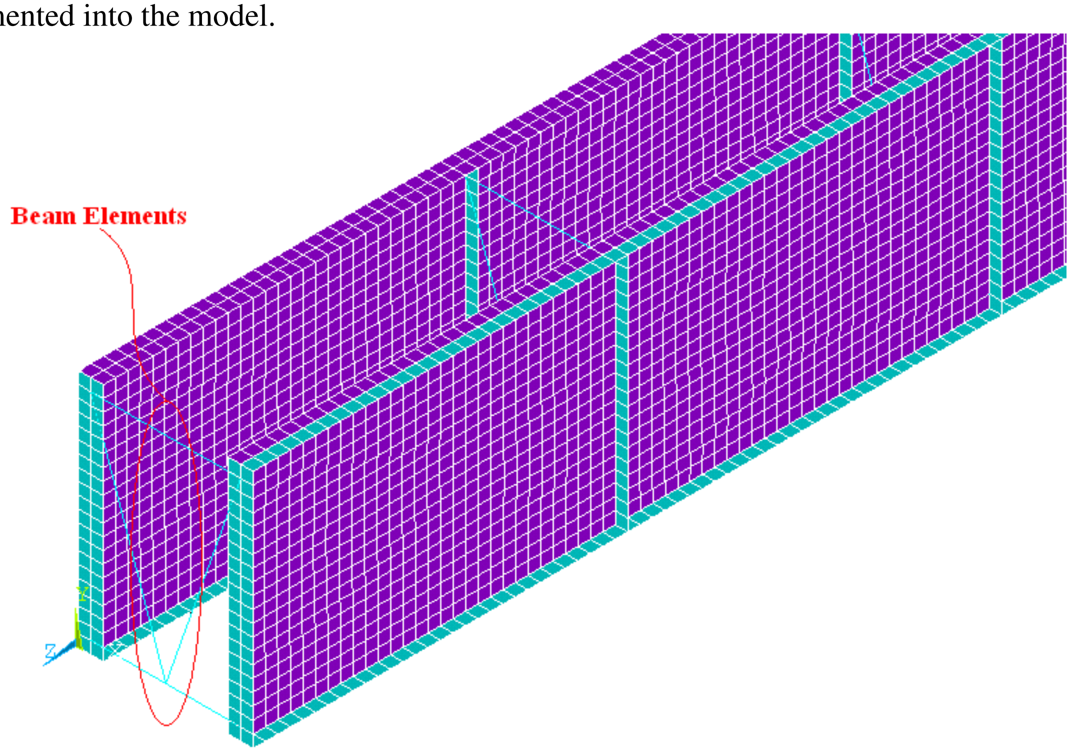

Figure 5-6. K-type cross-frame finite element model

Two approaches were 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 little effect on the girder deflection.

The following modeling procedures were followed:

- Material property set is defined for the steel.
- Element types and analysis type is defined.
- 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 the software's degree-of-freedom restrictions, the most critical and the maximum span length girders were selected. Using the cross-section properties of these girders and other bridge components, a model was created that consists of four girders spaced 10 ft apart and web stiffeners and cross-frames 25 ft apart. Since the bridge was modeled as a simply supported single-span bridge, simply supported boundary conditions were used.

5.6 Load Calculation and Application

Wind speeds of 70 to 100 mph were considered. This load was converted into uniform pressure and then applied laterally to the rightmost exterior girder. Calculation of the lateral load due to the wind load was performed based on the height of the girder. After checking the accuracy of the structure, elastic static analyses were carried out on the four-girder bridge model. These analyses were performed for two different cases: girders attached without permanent metal deck forms—i.e., bracing with cross frame only—and girders attached with permanent metal deck forms.

These cases are essential because it was demonstrated analytically and experimentally by others that a metal deck can be used for bracing 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. The isometric view of the bridge model is shown in Figure 5-7. 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 was 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.

Since effective bracing must satisfy stiffness and strength criteria, lateral stiffness is a prime parameter to study the effect of the stiffness of PMDF on the stiffness of the total system stiffness. Lateral stiffness represents displacement 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 of a four-girder system with metal deck forms.

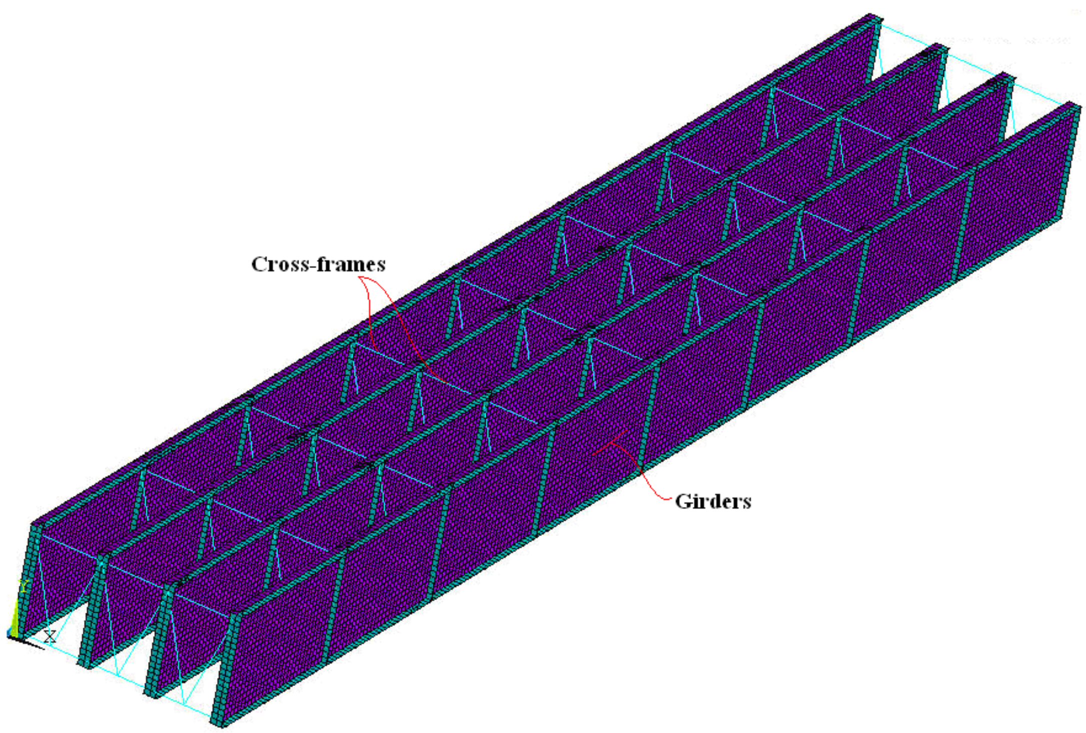

Figure 5-7. 3-D finite element model of four girders with cross-frames

5.7 Modeling Comparisons and Results

The bridge was modeled with and without metal deck forms. Only the midspan top flange deflections were evaluated for the simple span bridge structure. A complete deflection summary for both models was tabulated and graphed, and is discussed in the following sections.

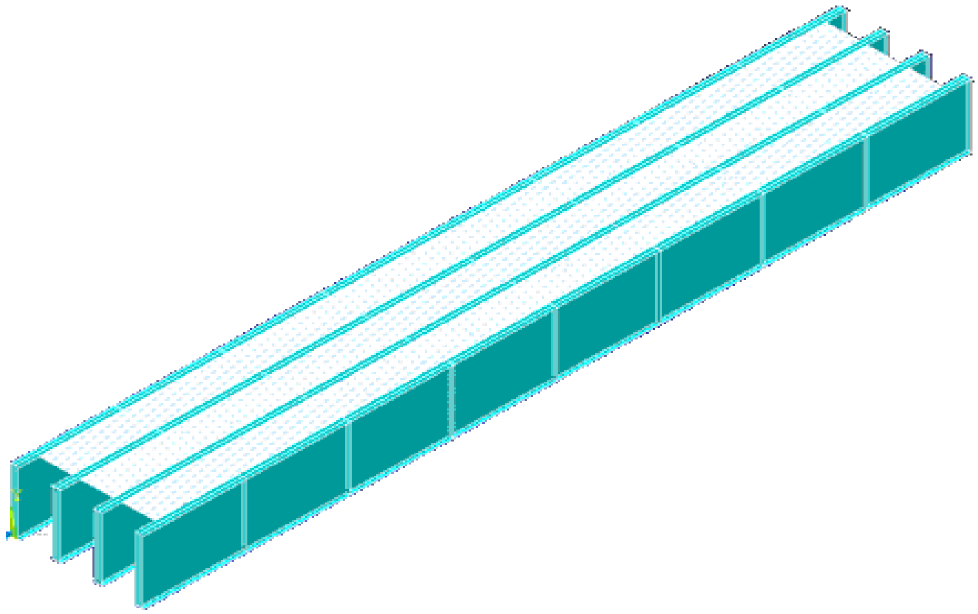

Figure 5-8. 3-D finite element model of four girders with metal deck form

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. Figure 5-9 shows the location where the lateral deflection in the X-direction was measured. Deflections at keypoints were observed. The deflections at the rightmost and leftmost exterior girders in the four-girder system without metal deck forms was also tabulated and provided in Table 5-1. Hand calculated displacement results based on simple beam theory were obtained for a single girder and then compared to the FEM results. These models showed results within 1% as compared to the hand calculations and, therefore, the girder modeling method used for the models was deemed adequate for use in an entire bridge model. During the modeling of the two and four girder systems, it was observed 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

Girder displacement without PMDF in inches					
Number of Girders	FEM				Theory
	Left Girder		Right Girder		
	Top Flange (avg. of three points)	Bottom Flange (avg. of three points)	Top Flange (avg. of three points)	Bottom Flange (avg. of three points)	
2	1.9678	1.9725	1.9676	1.9724	2.46
4	0.9829	0.98355	0.98315	0.98379	1.22

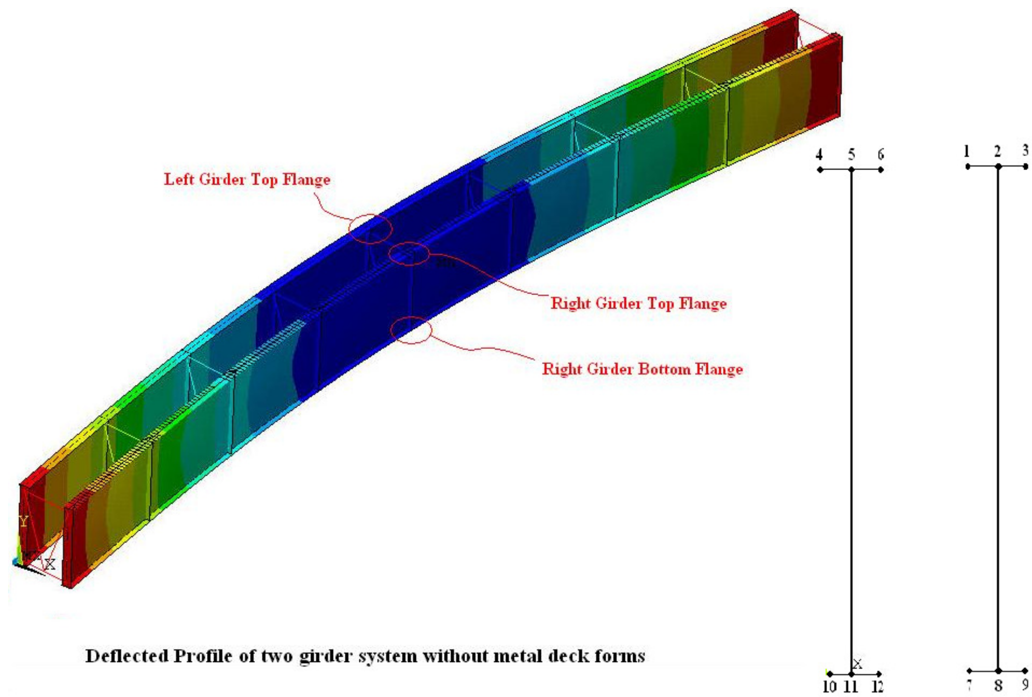

Figure 5-9. Deflected finite element model of the twin-girder system

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, the analytical model shown in Figure 5-8 for two-girder and four-girder systems was analyzed. Similar uniform lateral loading was applied to the exterior rightmost girder, and lateral displacement due to this loading was monitored. With known lateral displacement (Δ), 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 Appendix B. Sample lateral stiffness calculations for the studied bridge are shown in Appendix B. Girder displacements seen for the case with metal deck forms were measured at the same keypoints. Deflected profiles of the four-girder bridge system and the displacement values are presented in Figure 5-10 and Table 5-2 respectively.

During the modeling of the girder/metal deck form system, special attention was given to connection details between metal deck forms and support angles. The most flexible connection detail—the unstiffened detail when the decking is below the flange of the girder at its extreme eccentricity—was used. This connection was used to emulate actual bridge behavior, and is necessary since previous researchers showed that, for short-span, shallow bridge girders, the presence of this type of eccentric connection detail results in a 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 displacements obtained from the analyses, the lateral stiffness of the bridge system was calculated and then compared to theoretically calculated results. Therefore, first a complete summary of lateral displacement results for both models is tabulated. Table 5-2 shows the average displacement results at keypoint positions. It can be seen that girder top and bottom flange displacements reduce substantially when a metal deck form is installed on girders with the help of support angle connections.

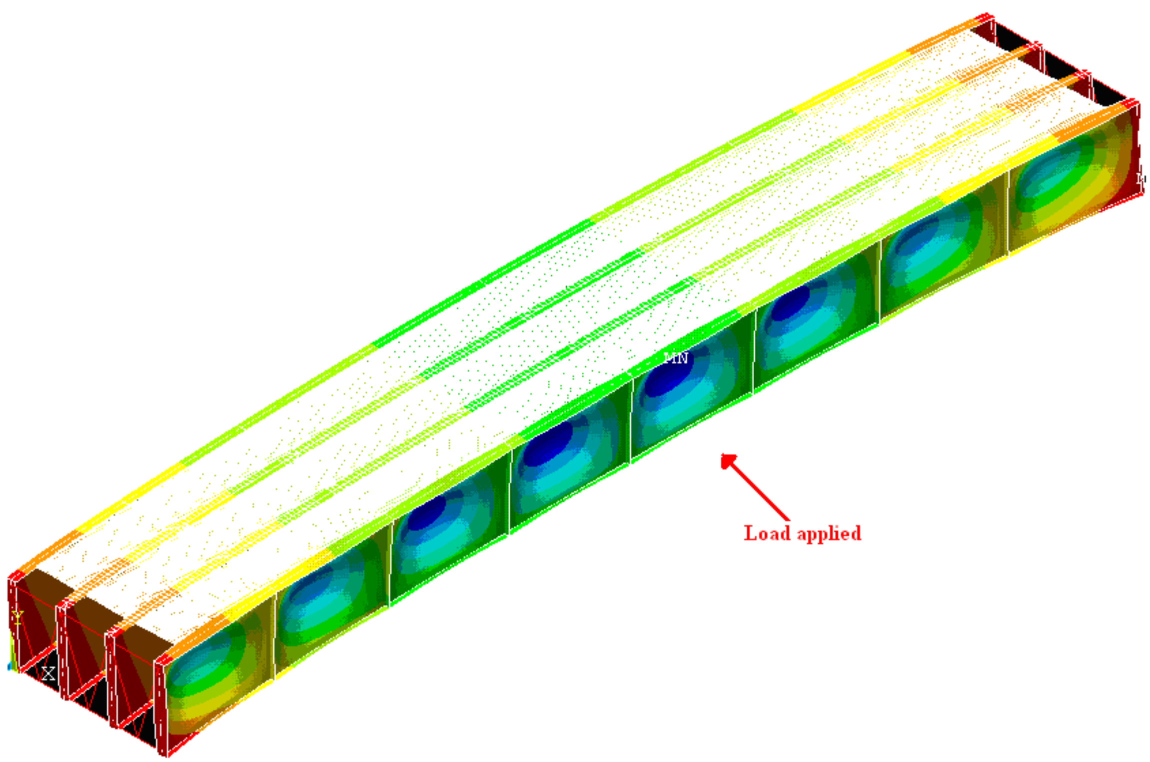

Figure 5-10. Deflected profile for girders with PMDF

Table 5-2. Lateral displacements for girders with PMDF

Girder displacement without PMDF in inches					
Number of Girders	FEM				Theory
	Left Girder		Right Girder		
	Top Flange (avg. of three points)	Bottom Flange (avg. of three points)	Top Flange (avg. of three points)	Bottom Flange (avg. of three points)	
2	0.14067	0.19125	0.14018	0.19115	0.137
4	0.04151	0.047035	0.040371	0.047294	0.0458

Based on these displacement results, the lateral stiffness of the metal deck form/girder system is estimated and presented in Table 5-3.

Table 5-3. Lateral stiffness for girders with PMDF

No. of Girders	FEM (inches)			Theory (inches)		
	No deck	Girders With Metal Deck Form	Lateral Stiffness G' (kips/in)	No deck	Girders With Metal Deck Form	Lateral Stiffness G' (kips/in)
2	1.97	0.141	71.2	2.46	0.137	72.7
4	0.983	0.0459	74.1	1.22	0.0458	72.7

The theoretical lateral stiffness calculation procedure is based on the classic equation for springs in series given by the Equation 5-2:

$$\frac{1}{G_{sys}} = \frac{1}{G_{deck}} + \frac{1}{G_{conn}} \tag{5-2}$$

The full lateral stiffness calculation procedure is demonstrated in 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 substantially restrained deformation 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 (approximately 2%) with theoretical results. For a full bridge finite element model, results obtained 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%. This indicates that the PMDF system possesses sufficient in-plane rigidity to treat the PMDF system as a bracing element for 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 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 cannot be proposed that design equations formulated by others are applicable for deep, long-span bridge girders, but this study will 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 was 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

During construction, long-span steel plate girder bridges are susceptible to instability caused by wind action due to their flexibility. Due to maintenance costs, DOTs have been systematically replacing older truss bridges with long-span steel plate girder bridges. Engineering projects across significant rivers and waterways are being planned around Alabama, and long and super long-span steel plate girder bridges are being considered. During construction of several plate girder bridges, ALDOT engineers encountered vibration problems due to high wind, which leads to the instability of the compression flanges. The structural stiffness of plate girder bridges under construction is much less than the structure in service conditions, and consequently they become susceptible to the dynamic wind action. In this study, only a brief investigation on the vibration characteristics of bridge girders has been concluded. Dynamic modal analysis was performed to observe the change in mode shapes and natural frequency of the bridge girder system with the presence of metal deck forms. 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.

Two- and four-girder bridge systems with and without metal deck forms were analyzed, and the natural frequency results and mode shapes are presented in Table 5-4 and Table 5-5. To study the vibration effect of metal deck forms on bridge girders, overall changes in natural frequencies were observed. Therefore Table 5-5 lists the natural period of the girders with and without metal deck forms for the two and four-girder systems. It can be seen from Table 5-5 that the natural periods for the two-girder and four-girder systems are close for like modes of vibration (4 seconds compared to 3.98 seconds). The mode shapes were plotted to observe the bracing effects of metal deck form/girder systems. It was seen that, when only cross-frames are present, the braces have less effect on the mode shape. Figure 5-11 shows mode shapes corresponding to the first and fifth modes of the two-girder system without metal deck forms. The mode shapes corresponding to the first and second modes of the structure with PMDF are shown in Figure 5-12.

Table 5-4. Natural frequencies for girders with and without PMDF

Mode	Natural Frequency (Hz)			
	Two-girder system		Four-girder system	
	Without PMDF	With PMDF	Without PMDF	With PMDF
1	0.84	0.25	1.13	0.251
2	2.021	0.98	2.392	0.984
3	2.863	2.079	3.153	2.088
4	3.359	3.04	3.438	3.058
5	3.463	3.204	3.465	3.39

Table 5-5. Natural period for girders with and without PMDF

Mode	Natural Period (sec)			
	Two-girder system		Four-girder system	
	Without PMDF	With PMDF	Without PMDF	With PMDF
1	4	1.19	3.98	0.884
2	1.021	0.494	1.0162	0.418
3	0.481	0.349	0.478	0.317
4	0.328	0.297	0.326	0.2908
5	0.312	0.288	0.294	0.288

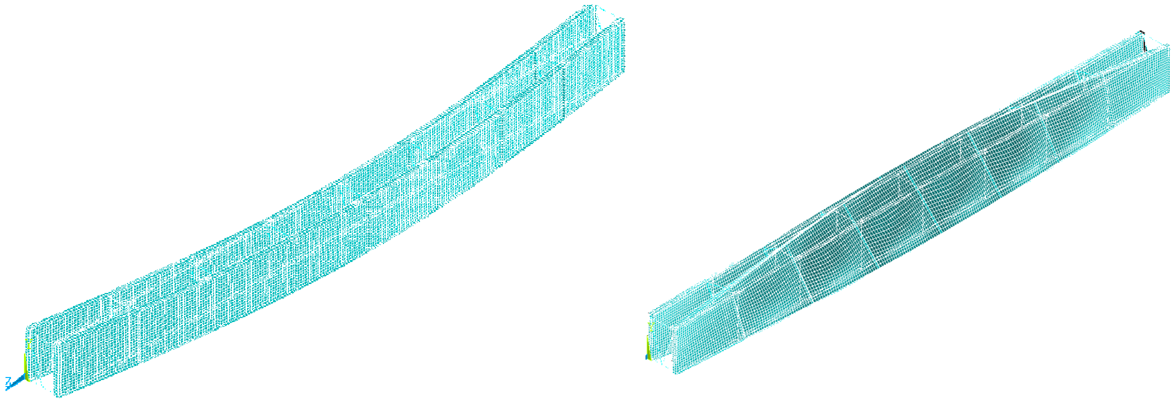

Figure 5-11. Mode shapes for two-girder system without PMDF

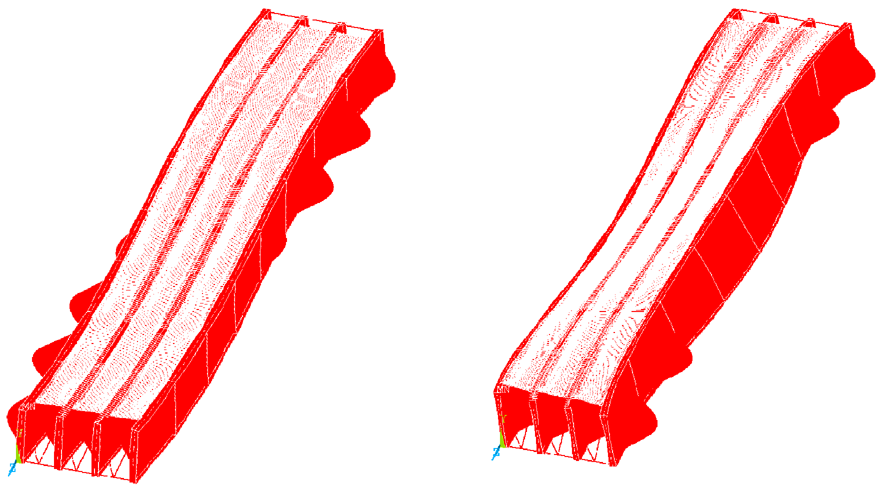

Figure 5-12. Mode shapes for four-girder system with PMDF

Section 6
Summary and Future Work

Vibration problems due to high winds have been encountered during the construction of deep, long-span, steel plate girder bridges. As compared with the service condition, the structural stiffness of plate girder bridges under construction is significantly less. Cross-frames and diaphragms have generally been used for lateral wind stability of girders during construction, but there is a need to minimize discrete bracing because of complications in erection and fabrication and fatigue problems. Therefore researchers have studied and demonstrated that metal decking can provide significant lateral stability to short-span, shallow girder systems. 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. The study was divided in-to 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

Research on the use of metal deck forms as a bracing element for the building construction industry was started during the 1960s. However, the PMDF systems used in the bridge construction industry differs substantially from the form system used in the building construction industry. Therefore, researchers have performed comprehensive research programs that include 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, researchers have developed and proposed modified connection details that improved the stiffness of the existing metal deck form system, along with the necessary design equations. One of the goals of this study was to thoroughly understand these prior formulated design equations, and investigate their applicability to long-span, deep plate girder bridges.

6.2 Applicability

The primary objective of the applicability section 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 construction loading. Design equations developed by others were presented for considering the contribution of PMDF systems to lateral bracing during constriction. 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 increase with the depth of the individual girder and the *L/d* ratio. To study the applicability of this method, a demonstration 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 bracing, which is a primary requirement of any bracing system, is also explained.

It was observed that, for the particular bridge studied, moment capacity for an unbraced length of 25 ft is much greater than the factored dead load moment. Stiffness calculations are therefore unnecessary, and decking with and without stiffening angles 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.

6.3 Finite Element Analysis

Detailed finite element models of steel plate girder bridges were created. The same bridge that was used for design calculation was analyzed. Three-dimensional models were used to 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 analyses were performed and all components of the bridge superstructure were modeled with linear elastic material models. Parameters such as cross-frames, stiffeners, and permanent metal deck forms were considered in the development of the finite element models. Stiffness results for girder systems 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 significantly decreased deformation over girder systems without metal deck forms. A full bridge finite element model with eccentric support angle connections showed good agreement with theoretical results (around 2% difference).

Metal deck form systems studied satisfies the stiffness criteria. Based on agreement between FEM results and theory calculations, this case study indicates 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. Since the ultimate goal of this research project was 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 was presented. Mode shapes for two- and four-girder bridge systems with and without metal deck forms were analyzed. The effect of the stiffening angle was not included in the dynamic finite analysis. During this analysis, it was seen that that girders braced by a metal deck forms do not demonstrate 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.

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 must be validated using different bridge cross-sections. The 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, additional detailed dynamic analyses are also necessary.

Section 7
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Appendix A
Lateral Stiffness Calculations from ANSYS Deflection Results

When girders are braced by metal deck forms, the most important parameter is shear rigidity, Q_{sys}, which 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 the deck (S_d):

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

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

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

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

Shear rigidity (Q_{sys}) 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_{deck}

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

$$Q_{deck} = G'_{deck} \times deck\ span$$

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

$$Q_{deck} = \frac{G'_{deck} \times deck\ span}{2}$$

Q_{conn}

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:

$$Q_{connection} = \frac{Q_{recommended}}{(span/2)}$$

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 resulted in Q_{system} equal to 3490 kip/rad. Finally, the shear rigidity of the metal deck form/girder system for studied bridge is

$$G'_{sys} = \frac{Q_{sys}}{S_d} = \frac{3490}{48} = 72.7\ kip / inch$$

Appendix B
Lateral Deflection for Girders with and without PMDF

The SDI manual uses the following deflection equations for girders with and without a metal deck form:

Lateral deflection calculation without PMDF:

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

$$Deflection\ (\Delta) \ = \frac{5}{384} \times \frac{0.5 \times 0.00694 \times 192 \times (200 \times 12)^4}{2.9 \times 10^7 \times 2 \times 2016} = 2.46 \ \ inches$$

Lateral deflection calculation with PMDF:

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

$$Deflection\ (\Delta) \ = \ \frac{0.016 \times 200^2}{8 \times 8 \times 72.7} \ = \ 0.137\ inches.$$

These theoretically calculated values were compared with finite element results and presented in Table 5-3.

Appendix C
Bridge Design Example

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

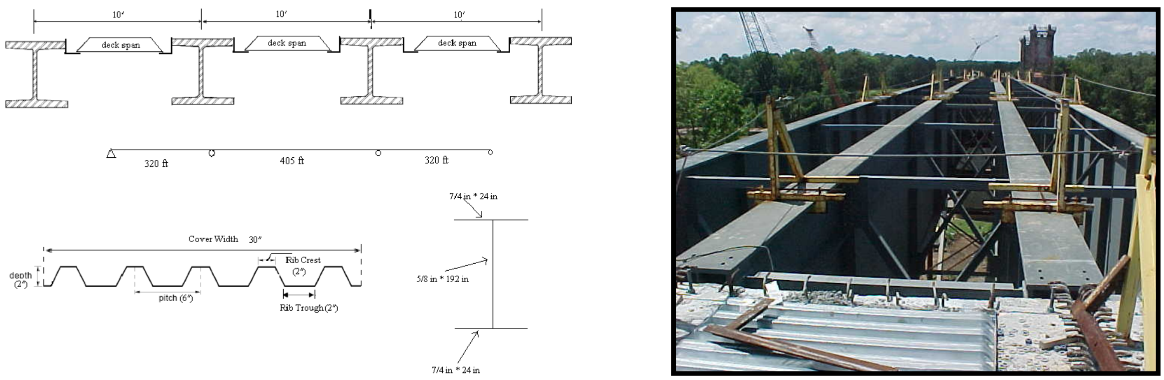

Figure C-1. Bridge over Tombigbee River, Layout and Components

This example focuses 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.

<u>Girders braced with the contribution of metal deck forms</u>
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* 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:

$$G'_{required} = \frac{4(M_{cr} - C_b^* M_{AASHTO})}{s_d md},$$

where s_d is tributary width of the deck bracing a single girder:

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

L_b = 25 ft; C_b^*= 1.0; d = 192 in

Here $C_b^* M_{AASHTO(25\,ft)}$ =102807.9 K-ft > 55654 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.

<u>Bracing behavior of metal deck form/girder system with stiffening angle connection</u>
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, the 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:

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

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

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

L_b = 50 ft; C_b^* = 1.0; d = 192 in.

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

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

$$G'_{required} = \frac{4(55654 \times 12 - 51404 \times 12)}{72 \times 0.375 \times 192} = 39.35 K / in$$

The stiffened metal deck form system with effective shear stiffness of 39.35 K/in provides sufficient bracing 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 spacing of intermediate cross-frames.

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