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Maintaining Deck Profile in Steel I-Girder Bridges During Deck Placement

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MAINTAINING DECK PROFILE IN STEEL I-GIRDER BRIDGES DURING DECK PLACEMENT

by

Ali Naif Inceefe

A thesis submitted to the Graduate College
in partial fulfillment of the requirements
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MAINTAINING DECK PROFILE IN STEEL I-GIRDER BRIDGES DURING DECK PLACEMENT

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Western Michigan University, 2018

Conventional steel I-girder highway bridges utilize cast-in-place concrete deck. Maintaining the deck profile as per project specifications might be a challenging task. Deformation of girders and formwork used for deck placement need to be controlled during deck placement. Differential deflection between girders, exterior girder web out-of-plane deformation, exterior girder warping, or a combination thereof could impact the deck profile. These deformations challenge maintaining deck profile and need to be considered as a part of constructability evaluation. Determining deck profile due to potential girder deformations is necessary before deck placement so that the remedial actions can be implemented when deck finish tolerances are violated. For non-complex steel I-girder bridges, cost- and time-effective procedures are needed for this evaluation. Simplified analysis tools were adopted and modified to analyze girder differential deflection and exterior girder warping. A procedure was developed to address the effect of web out-of-plane deformation on deck profile and demonstrated using an example. Remedial measures were suggested for maintaining the intended deck profile.
Dedicated to Aysel Akpınar. She will be missed forever.
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Ali Naif Inceefe
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a = \frac{E c_w}{G J}

b = factor

b_f = flange width

b_{fc} = top flange width

b_{ft} = bottom flange width

b_{ov} = deck overhang width

b_{sm} = screed machine wheel spacing

b_{sr} = width of screed rail platform

b_{t} = transverse stiffener width

b_{wa} = walkway width

C_w = warping constant of cross-section

D = web depth

d_o = transverse stiffener spacing

E = modulus of elasticity

= 29,000 ksi for steel

f_{sm} = factor for screed machine load per bracket

G = shear modulus of elasticity

= 11,200 ksi for steel

H_{br} = distance between the bracket bearing point and the bottom flange

I_C = connectivity index

I_S = skew index

I_x = major axis moment of inertia

I_y = minor axis moment of inertia

J = torsional constant of cross-section

L = span length

L_b = unbraced length, distance between the consecutive cross-frames

m = constant, equals to 1 for simple-span bridges and 2 for continuous-span bridges

N_b = number of girders

n_{cf} = number of intermediate cross-frames

n_w = number of screed machine wheels

P_c = concrete load per bracket

P_{CLL} = construction live load per bracket

P_{of} = combined overhang formwork and bracket load per bracket

P_{sm} = screed machine load per bracket
$P_{sm}$ = total weight of screed machine

$P_{wa}$ = walkway load per bracket

$R$ = radius of curvature of bridge centerline

$R_{A,x}$ = lateral load acting on the top flange due to overhang bracket

$R_{B,x}$ = lateral load acting on the exterior girder web at bracket bearing point

$S$ = girder spacing

$s_b$ = tributary area of the bracket

= bracket spacing when brackets are placed uniformly

$t_{fc}$ = top flange thickness

$t_{ft}$ = bottom flange thickness

$t_p$ = transverse stiffener thickness

$s$ = deck thickness

$T_u$ = uniformly distributed torsional moment

$W$ = bridge width

$w_c$ = weight of concrete

$w_{CLL}$ = construction live load

$w_g$ = width of the bridge unit measured between the centerline of the exterior girders

$w_{of}$ = combined weight of overhang formwork and bracket

$w_{SIP}$ = weight of SIP form

$w_{wa}$ = weight of walkway

$y_h$ = vertical projection of hanger rod

$z$ = factor

$\alpha$ = factor

$\alpha_c$ = scale factor

$\Delta_{deck}$ = variation in deck profile due to girder deformations

$\Delta_{dif}$ = differential deflection between girders (in.)

$\Delta'_{ex}$ = exterior girder deflection (in.)

$\Delta_{in}$ = interior girder deflection calculated using 1D line-girder analysis (in.)

$\Delta'_{in}$ = interior girder deflection (in.)

$\Delta_{in,m}$ = interior girder midspan deflection using 1D line-girder analysis (in.)

$\eta_L$ = exterior-to-interior girder load ratio (%)

$\theta$ = skew angle (deg), the angle between the axis of support and a line normal to the longitudinal axis of the bridge

$\theta_d$ = rotational deformation of exterior girder due to differential deflection

$\theta_f$ = rotational deformation of exterior girder due to warping when both ends are fixed

$\theta_p$ = rotational deformation of exterior girder due to warping when both ends are pinned
\[ \theta_t = \text{total rotation of overhang bracket} \]
\[ \theta_w = \text{rotational deformation of exterior girder due to warping} \]
\[ \theta_{we} = \text{rotational deformation of exterior girder top flange due to web out-of-plane deformation} \]
\[ \mu_e = \text{normalized mean error (\%)} \]
\[ \nu = \text{Poisson’s ratio} \]
\[ = 0.33 \text{ for steel} \]
1 INTRODUCTION

1.1 Background

Majority of highway bridges are composed of three main components, deck, superstructure, and substructure (FHWA 2012a). The bridge deck is directly exposed to vehicular loads, weather conditions, and detrimental chemicals such as deicing salts. The superstructure is responsible for transmitting the loads from the deck as well as the loads that act on superstructure directly such as wind loads to the substructure through bearings. Bridges are classified with respect to material and superstructure type. Among various bridge types, steel I-girder bridge is the most common bridge type carrying the vehicular traffic in the U.S. Based on the data from National Bridge Inventory (NBI), 17% percent of the U.S. bridge inventory is composed of steel I-girder bridges (Wu et al. 2010). A large majority of these bridges have the span length up to 200 feet (FHWA 2012b).

Conventional steel I-girder highway bridges utilize cast-in-place concrete decks. Figure 1-1 shows the main components of a conventional steel I-girder bridge.

As per the memorandum issued in 2000 by Federal Highway Administration (hereinafter referred to as “FHWA”), bridges in the U.S. should be designed as per the American Association of State Highway and Transportation Officials (hereinafter referred to as “AASHTO”) LRFD Specifications. AASHTO LRFD bridge design philosophy requires the bridges to be designed for
specified limit states to satisfy the requirements of constructability, safety, and serviceability, as well as inspectability, economy, and aesthetics. Limit states are primarily classified into four groups: (1) service limit state, (2) fatigue and fracture limit state, (3) strength limit state, and (4) extreme event limit state. As per the AASHTO (2017a) Article 1.3.2, Eq. (1-1) is the basis of LRFD methodology, and each component and connection shall satisfy this equation for each limit state, regardless of the analysis type used.

\[ \sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r \]  

(1-1)

In Eq. (1-1), \( \eta_i \) is a load modifier, \( \gamma_i \) is a load factor, \( Q_i \) is the force effect, \( \phi \) is the resistance factor, \( R_n \) is the nominal resistance, and \( R_r \) is the factored resistance.

During its entire lifetime, operational responsibilities of a bridge are undertaken by various parties. Owner’s design engineer is responsible for designing the bridge as per the AASHTO LRFD specifications and the state or agency specific policies documented in their design manual and guides. Once the design is completed, it is contractor’s responsibility to construct the bridge as per project specifications. Contractors are expected to have sufficient expertise and experience in the construction of the bridge under consideration. Owner agencies require the contractor to submit erection plans and procedures, deck placement sequence and necessary calculations before the construction starts. These calculations include, but not limited to, girder stability checks during lifting and erection, analysis of partially erected structures, analysis of the bridge frame under non-composite dead loads and construction loads, and the design of falsework and formwork systems to be used during each stage of construction. During the service life of the bridge, the owner is responsible for inspection and maintenance.

Although AASHTO LRFD specifications require bridges to be designed satisfying the requirements of constructability, typical highway bridge design considers the stresses in structural elements at various limit states, which assumes that the elements are fully constructed as per project specifications. Analysis of construction stages is the responsibility of the contractor since the means and methods for construction are defined by the contractor. Construction of conventional steel I-girder bridges starts with the lifting of girder or girder segments. In case of continuous I-girder bridges, girder segments are spliced either on the ground or in the air to develop continuous girders. After the girders are erected, cross-frames are connected. Subsequent to the
erection of the steel frame and formwork, pouring of cast-in-place deck starts as per a predefined pouring sequence. During these construction stages, structural elements and systems are subjected to loading, deformation and boundary conditions that might not be present in the finished, in-service structure. These conditions manifest themselves in various ways during cast-in-place deck placement as well and may create complications in retaining the intended deck profile.

A successful deck finish provides a safe and smooth riding surface for the users. Deck finish tolerances are critical for providing the ride quality. Cast-in-place deck construction possess many challenges. In fact, various state Department of Transportations (hereinafter referred to as “DOTs”) experienced issues during deck placement. As per the NCHRP Synthesis 345, eight DOTs, namely Florida, Kansas, Montana, North Dakota, Oklahoma, Pennsylvania, Tennessee, and Washington, experienced deck profile issues related to deck overhang construction and required very expensive fixes. Some of these fixes have failed to improve deck profile.

As shown in Figure 1-2, during cast-in-place concrete deck construction, fresh concrete between adjacent girders is often supported with stay-in-place (SIP) forms. The deck overhang concrete is supported with temporary formworks. The formwork is supported using overhang brackets. Because of the connection details between the formwork and the girders, girder deformations during deck placement will reflect on formworks and affect deck thickness and profile. Thus, to maintain the intended profile of the finished deck, controlling deformations of the fully erected bridge frame during deck placement is essential.
Potential girder deformations that may create complications in retaining the intended deck profile during deck placement of steel I-girder bridges can be specified as: (1) differential deflection between the girders, (2) girder web out-of-plane deformation, and (3) exterior girder warping.

In conventional steel I-girder bridge construction, it is a common practice to fully fasten cross-frames before deck placement starts so that connectivity between the girders and cross-frames is achieved. Such connections restrict independent deflection of girders and induce rotation in addition to vertical deflection. During deck overhang construction, component and construction loads are carried by overhang brackets that are supported by the exterior girders. The overhang bracket exerts lateral load to exterior girder web at the bearing point that may result in girder web out-of-plane deformations. In addition to these, exterior girders are subjected to non-uniform torsion due to eccentric overhang bracket loading, resulting in warping of the exterior girders.

These deformations challenge maintaining deck profile and need to be considered as a part of constructability evaluation. Determining variations in the deck profile due to girder deformations is necessary so that when the deck finish tolerances cannot be satisfied, remedial actions can be implemented. Often, refined analyses are performed to investigate such cases for complex structures, i.e., structures in which the skew and/or curvature or the irregular geometry significantly affect the behavior. However, for non-complex highway bridges, it is not cost- and time-effective to perform detailed analyses. Therefore, simplified procedures are needed for the evaluation of deck profile under girder deformations during construction.
1.2 Objective and Tasks

The objective of this thesis is developing simplified procedures for determining the deck profile and providing means and methods for maintaining the deck profile in steel I-girder bridges during construction. The objective will be accomplished by completing the following tasks:

(1) reviewing state-of-the-art literature for identifying practices and analyses regarding the topic,

(2) establishing overhang bracket analysis considering the relations with deck construction practices and construction loads and load transfer mechanisms,

(3) developing and adopting simplified analysis tools describing the girder deformations during the deck placement, including differential girder deflection, web out-of-plane deformations and exterior girder warping,

(4) discussing design and construction practice alternatives for maintaining the intended deck profile, and

(5) developing and providing a detailed example depicting the calculation procedure.

1.3 Scope of the Thesis

This thesis aims to develop computational procedures for determining and maintaining deck profile during deck placement of composite steel I-girder bridges. Deck issues unique to phased construction scenarios, i.e., closure pour complications, are not included in the thesis scope. Since the objective of the thesis is developing simplified computational tools for utilization by the design team and contractors, deliverables for girder differential deflection and exterior girder warping are not applicable to the complex structures where the skew and/or curvature or the irregular geometry significantly affect the structural behavior. The complexity of a structure and the required level of analysis for certain types of structural responses in steel I-girder bridges can be identified by the connectivity and skew index. As it will be discussed later, these indices are affected not only by curvature or skew but the number of cross-frames, span type, the bridge width and span length. Simplified analysis tools cannot capture the true behavior of these types of structures, thus, more advanced analysis efforts are required for analyzing these structures. The procedure developed for web out-of-plane deformation applies to curved and skewed bridges.
1.4 Methodology

The methodology followed in this thesis includes:

- Step 1: Identify the problem and the limitations of the available analysis tools for girder differential deflection and girder warping
- Step 2: Develop an analysis procedure for web out-of-plane deformation using fundamentals of thin plate theory
- Step 3: Perform finite element analyses to validate the applicability of the developed procedure
- Step 4: Calibrate the provided procedure based on the findings of finite element analyses
2 STATE-OF-THE-ART LITERATURE REVIEW

2.1 Overview

This chapter presents findings of the literature review on cast-in-place deck construction practices, construction loads and load combinations, and steel I-girder behavior during deck placement. Also, a brief summary of key findings is provided at the end of the chapter.

2.2 Cast-in-Place Deck Construction Practices

Conventional steel I-girder highway bridges utilize cast-in-place concrete deck. The cantilevered portion of the concrete deck, measured from the centerline of the exterior girder to the edge of the deck, is called a deck overhang (Figure 2-1). Deck overhangs enable the effective use of deck width with the least number of girders (Clifton et al. 2008, Fasl 2008).

A complete cast-in-place deck construction can be summarized as follows:

1. Install SIP forms between adjacent girders.
2. Attach overhang brackets to exterior girders and install the overhang formwork and walkway on the overhang brackets.
3. Place deck steel reinforcements.
4. Install screed rails and place the screed machine.
5. Pour and finish deck concrete.
6. Remove walkway, overhang formworks, and overhang brackets.
Figure 2-2 shows formwork, falsework, and their components used during various stages of cast-in-place deck placement in steel I-girder bridges, and each of them needs to be discussed in detail.

During deck placement, freshly poured concrete is supported and shaped by formwork systems. In general, formwork systems used between adjacent girders, and in the overhang portion of a deck are different.

The use of stay-in-place (SIP) forms between adjacent girders is a common practice in deck construction due to several advantages over the conventional plywood formwork. As the name implies, SIP forms are permanent components of a bridge that are not removed after the deck construction is complete. This implementation reduces the construction duration, cost of equipment and labor for formwork removal, and mobility impact time on feature intersect (Merrill 2002, Nims et al. 2006). Secondly, the use of SIP forms allows managing construction activities within the site constraints (Nims et al. 2006). Because of these advantages, most DOTs allow contractors to use SIP forms during deck placement. This is evident from a survey conducted by Grace et al. (2004), twenty-six of thirty-nine respondents allow SIP forms (Figure 2-3).
The fresh concrete of deck overhangs, on the contrary, is supported by temporary formworks. Figure 2-4 shows a typical deck overhang formwork system consisting of timber joists and plywood sheathing. Deck overhang formwork also provides an additional space to accommodate a walkway for the construction crew to access bridge deck.

Deck overhang formwork is supported by falsework members so called overhang brackets. A typical overhang bracket is shown in Figure 2-5. Usually, vertical and diagonal legs of overhang brackets are made of circular hollow sections. When needed, length of these components is
adjusted with respect to depth of the exterior girder and the width of the deck overhang. Bracket beam usually consists of two C-sections connected back to back.

![Bracket beam](image)

**Figure 2-5** A typical overhang bracket and its components (Source: https://www.deslinc.com/c49-bridge-overhang-bracket)

Overhang brackets are connected to steel I-girders by using hanger rods. Hanger rods are either welded or clamped to the exterior girder top flange. Figure 2-6 illustrates an example of a welded hanger rod.

![Hanger rod](image)

**Figure 2-6** A hanger rod welded to the top flange (Source: NCDOT 2011)

After formworks are mounted and the deck reinforcement is placed, a screed machine is installed. A screed machine spreads fresh concrete homogeneously throughout the deck to achieve the required profile. A typical screed machine and its components are illustrated in Figure 2-7. Screed machines are supported on screed rails using wheels (bogies) and move longitudinally along the
bridge span. Often, screed rails are installed on the edge of overhang formwork so that the maximum amount of concrete can be finished without implementing additional procedures (Clifton et al. 2008). Typical screed machines have either 4 or 8 wheels in total. As it will be discussed later, this is an important detail to determine the maximum screed load on a bracket. As the screed machine moves longitudinally, finisher operates in the transverse direction to finish the surface of placed concrete.

![Screed machine and its components](http://www.gomaco.com/downloads/finishers_brochure.pdf)

Once the deck placement is complete and the concrete has gained sufficient strength, temporary fixtures and formwork are removed from the bridge.
2.3 Construction Loads and Load Combinations

2.3.1 Construction Loads

During cast-in-place deck construction, girders are subjected to construction loads in addition to component loads (DC) that consist of the weight of fresh deck concrete and SIP forms. As per the AASHTO (2017a) Article C3.4.2.1, construction loads include the weights of material, removable formworks, personnel and construction equipment such as screed machines, and any loads applied to the structure through falsework or temporary supports. Thus, construction loads can be divided into two categories, construction dead loads (CDL) and construction live loads (CLL).

Various guidelines and recommendations on the application and magnitudes of these loads are presented in literature (AASHTO 2017b, Consolazio et al. 2014, FDOT 2018a, KDOT 2016, INDOT 2013, McPheron et al. 2012, MDOT 2018). Although there is an overall agreement on the component loads, there is no consensus among highway agencies on the magnitudes and application procedures of the construction loads. This is primarily due to the fact that construction procedures are developed by contractors based on their experience and the availability of equipment.

As per the AASHTO (2017b) Article 2.3.3.1, the combined weight of concrete, reinforcing and prestressing steel, and formwork shall not be taken less than 160 pcf for normal concrete. Unlike AASHTO, various DOTs specify the weight of concrete and formwork separately. FDOT (2018a) and INDOT (2013) specify the weight of concrete as 150 pcf. INDOT (2013) and MDOT (2018) indicate the weight of SIP form as 15 psf. Additionally, FDOT (2018a) specifies the combined weight of overhang formwork and overhang bracket as 15 psf. Summation of these loads results in a conservative estimation compared to the combined weight of concrete and formwork provided in AASHTO.

AASHTO (2017b) defines different construction live loads for the design of falsework and formwork. As per the Article 2.3.3.2.1, construction live load for falsework design includes the weight of any equipment to be supported, a uniform load of 20 psf applied over the area supported, and a 75 plf load applied at the outside edge of deck overhangs. On the other hand, for the design of formwork, the Article 3.2.1 states that construction live load shall not be taken less than 50 psf. Construction live loads are transferred to superstructure when falseworks or formworks are
supported by the superstructure. Therefore, DOTs use construction live loads for the design of falsework or formwork specified in AASHTO (2017b) in constructability analyses of the superstructure. As a component of falsework, overhang brackets are supported by girders and subjected to direct and non-redundant load distribution (KDOT 2016). There is no concept such as conservative estimation of displacements, however, deck finish tolerances are provided for worst-case scenarios. Therefore, in the absence of comprehensive and accurate state-specific construction load database, an application of a conservative construction live load for overhang bracket analysis is more appropriate.

The total weight of screed machine depends on the machine size and components. The accurate weight of the machine can be provided by the manufacturer. Various DOTs specify the weight of screed machine in their specifications in the absence of more precise information. INDOT (2013) specifies the screed machine weight as 4500 lb, whereas FDOT (2018a) standardizes the load magnitude by being specific about the bridge width. Since the total weight of the machine differs based on the machine size, it is more convenient to follow the Florida DOT’s guidance.

Table 2-1 provides a summary of component loads, construction dead loads, and construction live loads documented in AASHTO (2017b), FDOT (2018a), INDOT (2013), KDOT (2016), and MDOT (2018). The loads listed in Table 2-1 are used for the analysis presented in this thesis. Figure 2-8 illustrates the application of these loads. Weights of the concrete, SIP forms, combined overhang formwork and bracket, and walkway are applied on locations they occupy. Construction live load is applied on bridge width and walkway. Lastly, screed machine load is applied on the midpoint of the screed rail platform.
Table 2-1 Loads Acting on Girders during Deck Placement

<table>
<thead>
<tr>
<th>Load</th>
<th>Magnitude</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Component Loads (DC)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>150 pcf</td>
<td>FDOT (2018a) &amp; INDOT (2013)</td>
</tr>
<tr>
<td><strong>Construction Dead Loads (CDL)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overhang formwork + Overhang bracket</td>
<td>15 psf</td>
<td>FDOT (2018a)</td>
</tr>
<tr>
<td>Walkway</td>
<td>15 psf</td>
<td>INDOT (2013)</td>
</tr>
<tr>
<td><strong>Construction Live Loads (CLL)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Construction Live Load</td>
<td>50 psf</td>
<td>AASHTO (2017b) &amp; KDOT (2016)</td>
</tr>
<tr>
<td>Screed Machine</td>
<td>7 kips, if $26'\leq W \leq 32'$</td>
<td>FDOT (2018a)</td>
</tr>
<tr>
<td></td>
<td>11 kips, if $32'&lt; W \leq 56'$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>13 kips, if $56'&lt; W \leq 80'$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>16 kips, if $80'&lt; W \leq 120'$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$W = \text{bridge width (ft)}$</td>
<td></td>
</tr>
</tbody>
</table>

![Figure 2-8 Application of loads during deck placement](image)

### 2.3.2 Construction Load Combinations

The AASHTO (2017a) Article 3.4.2 presents the load factors for construction loads. The Article 3.4.2.1 states that load factors for dead load of structural components and appurtenances, $DC$ and $DW$, shall not be taken less than 1.25 while investigating the construction stages using Strength I and Strength III limit states. Additionally, construction loads, including the dynamic effects, shall be factored with a minimum of 1.5 for Strength I limit state. For Strength III limit state, however, construction loads and the wind load during construction shall be factored with a minimum of 1.25. Further, AASHTO (2017a) considers an additional load combination to magnify the effects of component and construction loads in the absence of service loads. In this additional load...
combination, a minimum factor of 1.4 shall be applied to dead load of structural components and construction loads including the dynamic effects.

In addition to these, the Article 3.4.2.2 states that deflections during construction shall be evaluated using Service I limit state. Construction loads shall be added to Service I limit state with a factor of 1.00.

Table 2-2 includes the load combinations for construction stages based on what has been discussed above.

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Load Combination</th>
<th>AASHTO Article</th>
</tr>
</thead>
<tbody>
<tr>
<td>Service I</td>
<td>1.00(DC) + 1.00(CDL+CLL)</td>
<td>Article 3.4.2.2</td>
</tr>
<tr>
<td>Strength I</td>
<td>1.25(DC) + 1.50(CDL+CLL)</td>
<td>Article 3.4.2.1</td>
</tr>
<tr>
<td>Strength III</td>
<td>1.25(DC) + 1.25(CDL+CLL) + 1.25(WS)</td>
<td>Article 3.4.2.1</td>
</tr>
<tr>
<td>Additional load combination</td>
<td>1.40(DC) + 1.40(CDL+CLL)</td>
<td>Article 3.4.2.1</td>
</tr>
</tbody>
</table>

where:
- **CDL** = construction dead loads
- **CLL** = construction live loads
- **DC** = dead load of structural components
- **WS** = wind load on structure

**Note:** During the deck placement, dead load of wearing surfaces and utilities, **DW**, does not exist, thus, it is excluded from the load combinations above.
2.4 Girder Behavior During Deck Placement

2.4.1 Overview

Achieving the cast-in-place deck profile as per project specifications requires controlling deformations of girders and formwork systems during deck placement. Thus, understanding girder behavior during deck placement is essential to predict potential issues with the deck profile and to take necessary actions for overcoming these issues. During deck placement, potential girder deformations that may create complications in retaining the intended deck profile can be due to girder differential deflection, exterior girder web out-of-plane deformation, exterior girder warping, or the combination thereof (ODOT 2007a). Hence, these deformations need to be considered as a part of constructability evaluation.

2.4.2 Differential Deflection between Girders

In general, girder deflection is a function of loads, boundary conditions, girder geometry and stiffness characteristics. Differential deflection of girders is affected by the connection detail between cross-frames and girders that defines the load transfer mechanism and boundary conditions.

In steel I-girder bridges, cross-frames are connected before pouring the concrete deck. As shown in Figure 2-9, there are four types of holes used in cross-frame connections: standard (STD), oversize (OVS), short slot (SSL), and long slot (LSL) (MDOT 2014). When standard holes are used, cross-frames can be fully fastened to control differential deflection. However, fit-up issues with the standard holes have encouraged contractors to use oversized- or slotted-hole cross-frames that allow girders to deflect independently. Although oversized- or slotted-hole cross-frames may present a solution for member fit-up issues, they create stability and deformation related problems.
Using oversized- or slotted-hole cross-frames significantly reduces the stability bracing provided by cross-frames (NCHRP Report 725 2012). Additionally, utilizing these cross-frames can result in difficulties in controlling the deformations during construction since the interconnectivity between the members cannot be achieved. Because of these complications, the NCHRP Report 725 does not recommend using oversized- or slotted-hole cross-frames. NSBA (2016a) underlines similar concerns and does not recommend the use of vertical slotted holes during deck placement. In fact, prohibiting the use of oversized- or slotted-hole cross-frames is a common practice among owner agencies. As per NCHRP Synthesis 345, ten DOTs do not allow the use of oversized holes. Additionally, sixteen DOTs reported that using oversized- or slotted-hole cross-frames are allowed only under certain conditions, where fit-up issues are a concern.

When cross-frames are fully fastened and the girders are subjected to differential deflection, girder cross section rotations occur. Since screed rails are supported by overhang brackets attached to the exterior girders, rotation of the exterior girders may challenge retaining the intended deck profile, as illustrated in Figure 2-10.

According to ODOT (2007a), girder differential deflection is a function of the difference in load magnitudes transmitted to individual girders. Figure 2-11a and Figure 2-11b illustrates the differential deflection when exterior girders and interior girders attract higher load, respectively.
ODOT (2007a) Section 302.2 states that for a new superstructure, girder twist due to differential deflection can be neglected when the tributary deck load carried by exterior girders does not exceed 110% of the average deck tributary load carried by the interior girders. The limit is specified as 115% for existing bridges. These regulations primarily discuss the effect of deck overhang widths with respect to given girder spacing and oversimplify the behavior. There are other parameters affecting differential deflection between girders, such as boundary conditions, structure geometry and stiffness characteristics of individual girders.

![Differential deflection between girders](image)

**Figure 2-11** Differential deflection between girders (a) when exterior girders are loaded heavily, and (b) when interior girders are loaded heavily

In straight steel I-girder bridges, girder deflections due to non-composite loads are often calculated using 1D line-girder analysis. In line-girder analysis, a girder is isolated from the rest of the structure and analyzed individually. Inherently, this analysis method cannot incorporate the effects of cross-frames on the structural behavior. However, the study by Fisher (2006) demonstrated that even in straight girder bridges, transverse load distribution through cross-frames has influence on differential girder deflections under non-composite loads. Fisher (2006) measured girder deflections during deck placement of seven simple-span and three continuous-span steel I-girder bridges with skew angles varying from 0 to 62 degrees. Then, 3D finite element models were developed, and field measurements were used to calibrate these models. Finally, simplified procedures were developed for predicting the girder deflections under non-composite loads by
conducting a parametric study with calibrated 3D finite element models. These procedures were adopted by North Carolina DOT for estimating the girder deflections during deck placement.

2.4.3 Web Out-of-Plane Deformation

Figure 2-12 shows the impact of girder web out-of-plane deformation on deck overhang geometry. As shown in the figure, the horizontal component of the bracket axial force acts on the exterior girder web as a lateral load at the bearing point. Depending on the force magnitude and the web slenderness, web out-of-plane deformation occurs. The corresponding overhang bracket rotation results in an uneven deck thickness. Therefore, the exterior girder web behavior needs to be evaluated during deck placement.

![Figure 2-12 Web out-of-plane deformation due to bearing point load](image)

As per the NCHRP Synthesis 345, eight DOTs reported construction issues related to deck overhang. Florida, Oklahoma, and Tennessee stated that they experienced deflection issues, and consequently poor deck profile as a result of not placing the overhang bracket bearing point at the web-bottom flange intersection. The bearing point of the bracket indirectly refers to the length of the bracket’s vertical leg. For a given bracket width, as the vertical leg of the bracket gets shorter, the angle between the diagonal and the vertical leg of the bracket increases. Therefore, under the same bracket load magnitude, the horizontal load acting on the girder web increases.

Various agencies provide guidance for controlling web out-of-plane deformation. The most common specifications are related to the position of the bearing point along the web. As shown in Figure 2-13, the position of the bearing point is described as the distance between the bearing point and top of the bottom flange. Table 2-3 summarizes the limits specified by various agencies for $H_{br}$, the distance between the bearing point and the bottom flange.
In addition to the limits provided in Table 2-3, Ohio DOT and Pennsylvania DOT provide additional discussions. The ODOT (2007a) Section 302.2.7.2 states that if the requirement in Table 2-3 is satisfied and the web depth under consideration is less than 84 inches, web out-of-plane deformation can be neglected. For the web depths greater than 84 inches, Ohio DOT requires calculating web out-of-plane deformation, however, an explicit method for such an analysis is not provided. As shown in Figure 2-14, the PennDOT (2015) Article 6.10.3.2.5.2P provides the maximum permissible jack (overhang bracket) spacing and the horizontal loads with respect to overhang bracket depths. If the loads are less than the permissible values, buckling of the web due to out-of-plane deformation is not considered. These horizontal loads include the weight of concrete, forms and incidental loads, and screed machine. The table in Figure 2-14 can be used if the following requirements are satisfied: (1) girder web depth is less than 8 ft – 0 in., (2) overhang width is less than 4 ft – 9 in., (3) deck thickness is equal or less than 10 in., (4) transverse stiffener spacing does not exceed the depth of the girder, and (5) $\gamma_w$ (see PennDOT (2015) Article...
$D6.10.1.9.3P$) is less 2.5 in the region under interest and the dead load shear factored with 4.0 is less than the buckling shear calculated as per the AASHTO (2017a) Article 6.10.9.3. These maximum permissible horizontal loads were determined based on the field measurement and FEA studies. When the above requirements are not satisfied, the limit for $H_{br}$ in Table 2-3 should be used.

<table>
<thead>
<tr>
<th>Nominal Depth, $y$ (in.)</th>
<th>Maximum Permissible Horizontal Load, $h$ (kip/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>36</td>
<td>1.250</td>
</tr>
<tr>
<td>56</td>
<td>0.750</td>
</tr>
<tr>
<td>76</td>
<td>0.540</td>
</tr>
</tbody>
</table>

Figure 2-14 Typical overhang bracket details of PennDOT (Source: PennDOT 2015)

When the bearing point is not located close to bottom flange-web intersection, i.e., if the limits in Table 2-3 cannot be satisfied, web out-of-plane deformation and the corresponding formwork rotation need to be determined. With similar concerns, some efforts have been reported in the literature for evaluating the web out-of-plane deformations due to lateral loads resolving from the overhang brackets. Yang et al. (2010) performed a parametric study to investigate the steel girder web out-of-plane deformations under the deck overhang loads. The main parameters considered in their study are the effects of the girder web slenderness, position of the bracket bearing point on the girder web, effects of the transverse web stiffeners, and the effects of the deck overhang width. Additionally, they included the effects of the top flange width, P-delta effects, and the effects of initial web imperfections. They concluded that web slenderness, the position of bracket bearing point on the web, transverse stiffener spacing, and the overhang width are the dominating parameters that have a greater influence on web out-of-plane deformations. Besides these, it was stated that P-delta effects and initial web imperfections do not have profound effects on web out-of-plane deformations. Even though Yang et al. (2010) identified the major parameters affecting steel I-girder web behavior under the deck overhang loads, computational tools are not provided
for calculating the out-of-plane deformation and the corresponding rotational deformation of an exterior girder.

### 2.4.4 Exterior Girder Warping

A girder is subjected to pure bending if loads are applied on the girder through its shear center. If the applied loads do not coincide with the shear center, a girder is subjected to the combined effect of bending and torsion. When a girder is under torsional loading, in addition to twisting, it may warp. Warping means that girder’s cross-section does not remain in plane after twist. Depending on how the member resists the applied torque, torsion concept can be divided into two categories, uniform and non-uniform torsion. In uniform torsion, the member under consideration is allowed to warp freely, and the torsional moment is resisted by St. Venant resisting moments. If the warping of the member is restrained, then the applied torsional moment is resisted by summation of St. Venant and warping resisting moments. This phenomenon is called the non-uniform torsion.

During the deck placement, component and construction loads act eccentrically to the exterior girder. As illustrated in Figure 2-15, if the cross-section warping is not allowed at cross-frame locations, the exterior girder is subjected to a non-uniform torsion between consecutive cross-frames. Overhang bracket also rotates with the exterior girder and results in unintended deck profile. Assuming a full restraint against warping at the cross-frame location is an idealization. The actual warping fixity provided by cross-frames is somewhere between fixed and pinned boundary conditions (KDOT 2016).

![Figure 2-15 Exterior girder behavior under non-uniform torsion (Source: ODOT 2007b)](image)

In literature, approximate solutions and computer-based analysis tools are available for calculating the rotational response of an exterior girder under torsional loading. As described in the Article
6.10.3.4 of Idaho DOT Bridge Manual (2017), applied torsion on an exterior girder can be resolved as a force couple acting on top and bottom flanges (Figure 2-16). Then the flanges are assumed as separate continuous beams and analyzed under these lateral loads. Finally, rotation of the exterior girder can be calculated by dividing the sum of two resulting deflections by the girder depth.

![Figure 2-16 Force couple on an exterior girder (Source: Idaho DOT 2017)](image)

KDOT (2016) utilizes the solution of the governing differential equation for torsion to calculate the rotation of exterior girders. When warping is restrained, the total torsional moment \( T \) resisted by the cross-section is the summation of St. Venant \( T_t \) and warping resisting \( T_w \) moments:

\[
T = T_t + T_w = GJ\theta' - EC_w\theta'''
\]

(2-1)

where \( G \) is the shear modulus of elasticity, \( J \) is the torsional constant of the cross-section, \( \theta \) is torsional rotation about the longitudinal axis, \( E \) is the modulus of elasticity, and \( C_w \) is warping constant of the cross-section.

In addition, the University of Kansas developed a software for Kansas DOT called “Torsional Analysis of Exterior Girders (TAEG)” to provide a tool for estimation of torsional rotation of exterior girders. It is a public domain software and it can be downloaded from KDOT website. Several agencies suggest using TAEG for estimation of torsional rotation in exterior girders (ODOT 2007a, FHWA 2015).

TAEG provides an approximate solution with limitations and assumptions. Ashiquzzaman et al. (2017) investigated the accuracy of the software comparing TAEG output with field measurements and finite element analyses. They concluded that: (1) the software assumes an ideal connection between temporary bracings, which may not be the case in the field, (2) skew effects cannot be addressed adequately, (3) the software has limited cross-frame/diaphragm alternatives, and (4) the software is not able to consider non-uniform spacing of cross-frames/diaphragms. Additionally,
TAEG does not consider curved girders, thus, torsional effects induced by curvature are not addressed (Roddis et al. 2002).

2.4.5 Considerations for Skewed and Curved Bridges

All bridges are subjected to gravity load effects, i.e., shear and bending moment effects, vertical deformations, and major axis bending rotations. In the presence of curvature and/or skew, bridges experience torsional St. Venant shear and warping normal stresses, flange lateral bending, load shifting, and twisting deformations (NSBA 2014).

Structural analyses require models for geometry, boundary conditions and loads acting on the system. Accuracy of the analysis depends on the ability of the models to represent the structural system under consideration. Approximate solutions, i.e., 1D line-girder and 2D grid methods, may not provide sufficiently accurate solutions for certain type of structural responses for complex structures with skew and/or curvature or irregular geometry.

In NCHRP Report 725, an extensive number of analyses was performed to identify the required level of analysis for capturing the accurate behavior of steel I-girder bridges under non-composite dead loads. Authors compared the analyses results of approximate methods, i.e., 1D line-girder and 2D grid methods, for various structural responses with the results obtained from refined 3D finite element models. Based on findings, two different indices were developed to characterize the behavior of curved and skewed bridges: connectivity index ($I_C$) and skew index ($I_S$).

The connectivity index is an ad hoc index used for curved bridges. Cross-frame spacing and radius of curvature are the key parameters affecting the accuracy of simplified analysis of curved steel I-girder bridges (NCHRP Report 12-79 2012). The index includes these parameters, and is expressed as:

$$I_C = \frac{15000}{R(n_{cf} + 1)m}$$  \hspace{1cm} (2-2)

where $R$ is the radius of curvature of bridge centerline (ft), $n_{cf}$ is the number of intermediate cross-frames, and $m$ is a constant, equals to 1 for simple-span bridges and 2 for continuous-span bridges. For continuous bridges, $I_C$ needs to be calculated for each span, and the largest value is taken as
the index of the bridge. Authors identified that the accuracy of simplified analyses decreases when \( I_C \) is greater than 1.

The skew index characterizes bridges based on the significance of skew effects, which are directly related to transverse stiffness and load path of a bridge (NCHRP Report 12-79 2012). The skew index is calculated as:

\[
I_S = \frac{w_g \tan \theta}{L}
\]  

(2-3)

where \( w_g \) is the width of the bridge measured between the centerline of the exterior girders (ft), \( \theta \) is the skew angle, and \( L \) is the span length (ft). It was realized that skew effects tend to be negligible when the skew index is less than 0.30. When the index is between 0.30 and 0.65, flange lateral bending stresses and cross-frame forces due to skew are significant. When the index is greater than 0.65, in addition to flange lateral bending stresses and cross-frame forces, major axis bending stresses and vertical deflections are affected by skew.

The study also considered bridges with both curvature and skew. NCHRP Report 725 states that bridges can be considered as straight-skewed bridges when \( I_C < 0.5 \) and \( I_S > 0.1 \). In the case \( I_C > 0.5 \) and \( I_S \leq 0.1 \), bridges are classified as horizontally curved with no skew.

A matrix was developed for the recommended level of analysis for steel I-girder bridges based on the error measurements obtained from approximate analysis methods. Table 2-4 provides the recommended level of analysis for curved and/or skewed bridges for certain structural responses. Description of the scores shown in the matrix is provided in Table 2-5. Scores were developed with respect to normalized mean errors, which is the deviation of simplified analysis results from 3D finite element analysis results. The report suggests considering the worst-case scores when the bridge under consideration has irregular geometric features such as a poor span balance or unsymmetrical geometry.
<table>
<thead>
<tr>
<th>Response</th>
<th>Geometry</th>
<th>Worst-Case Scores</th>
<th>Mode of Scores</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Traditional</td>
<td>1D-Line Girder</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2D-Grid</td>
<td></td>
</tr>
<tr>
<td><strong>Major-Axis Bending Stresses</strong></td>
<td>C ((I_C \leq 1))</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>C ((I_C &gt; 1))</td>
<td>D</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>S ((I_S &lt; 0.30))</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>S ((0.30 \leq I_S &lt; 0.65))</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>S ((I_S \geq 0.65))</td>
<td>D</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>C &amp; S ((I_C &gt; 0.5 &amp; I_S &gt; 0.1))</td>
<td>D</td>
<td>F</td>
</tr>
<tr>
<td></td>
<td>C ((I_C \leq 1))</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>C ((I_C &gt; 1))</td>
<td>F</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>S ((I_S &lt; 0.30))</td>
<td>B</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td>S ((0.30 \leq I_S &lt; 0.65))</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td>S ((I_S \geq 0.65))</td>
<td>D</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>C &amp; S ((I_C &gt; 0.5 &amp; I_S &gt; 0.1))</td>
<td>F</td>
<td>F</td>
</tr>
<tr>
<td><strong>Vertical Displacements</strong></td>
<td>C ((I_C \leq 1))</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>C ((I_C &gt; 1))</td>
<td>F</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>S ((I_S &lt; 0.30))</td>
<td>NA(^a)</td>
<td>NA(^a)</td>
</tr>
<tr>
<td></td>
<td>S ((0.30 \leq I_S &lt; 0.65))</td>
<td>F(^b)</td>
<td>F(^c)</td>
</tr>
<tr>
<td></td>
<td>S ((I_S \geq 0.65))</td>
<td>F(^b)</td>
<td>F(^c)</td>
</tr>
<tr>
<td></td>
<td>C &amp; S ((I_C &gt; 0.5 &amp; I_S &gt; 0.1))</td>
<td>F(^b)</td>
<td>F(^c)</td>
</tr>
</tbody>
</table>

| a Magnitudes should be negligible for bridges that are properly designed & detailed. The cross-frame design is likely to be controlled by considerations other than gravity-load forces. |
| b Results are highly inaccurate. The improved 2D-grid method discussed in Chapter 6 of NCHRP 12-79 Task 8 report provides an accurate estimate of forces. |
| c Line-girder analysis provides no estimate of cross-frame forces associated with skew. |

where:

- C = curved bridge
- S = skewed bridge
- C&S = curved and skewed bridge
- \(I_C\) = connectivity index
- \(I_S\) = skew index

Table 2-5 Meaning of the Scores in the Recommended Level of Analysis Matrix

<table>
<thead>
<tr>
<th>Score</th>
<th>Normalized Mean Error</th>
<th>Performance</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>(\mu_e &lt; 6%)</td>
<td>Excellent accuracy</td>
</tr>
<tr>
<td>B</td>
<td>(7% &lt; \mu_e &lt; 12%)</td>
<td>Reasonable agreement</td>
</tr>
<tr>
<td>C</td>
<td>(13% &lt; \mu_e &lt; 20%)</td>
<td>Significant deviation</td>
</tr>
<tr>
<td>D</td>
<td>(21% &lt; \mu_e &lt; 30%)</td>
<td>Poor</td>
</tr>
<tr>
<td>F</td>
<td>(\mu_e &gt; 30%)</td>
<td>Unreliable &amp; inadequate</td>
</tr>
</tbody>
</table>

As the matrix in Table 2-4 indicates, 1D and 2D analysis do not provide an accurate estimate of girder behavior under non-composite loads for complex structures due to inability to accurately represent the torsional effects due to curvature and skew as well as the geometry, boundary conditions and loads. Since differential girder deflection and exterior girder warping are directly
related to structural responses noted in Table 2-4, analyses of these deformations require more refined analysis methods for complex structures.

2.5 Summary

Achieving the cast-in-place deck profile as per project specifications requires controlling deformations of girders and formwork systems during deck placement. In a cast-in-place concrete deck construction, fresh concrete between adjacent girders is placed on SIP forms. For the deck overhang portion, temporary formworks are utilized for supporting and shaping the wet concrete. Deck overhang formworks are supported by overhang brackets and the brackets are directly attached to exterior girders. Hence, girder deformations reflect on formworks as well as deck profile.

Component and construction loads acting on superstructure during deck placement are needed to be used in the analysis of overhang bracket and girder deformations. These loads and their applications are identified and adopted from literature.

Potential girder deformations that may challenge retaining the intended deck profile can be specified as differential deflection of girders, exterior girder web out-of-plane deformation, and exterior girder warping. For non-complex highway bridges, simplified procedures are needed for the evaluation of deck profile under these girder deformations.

Often, girder deflections are calculated using 1D line-girder analysis in straight steel I-girder bridges. However, the study by Fisher (2006) demonstrated that even in the straight girder bridges, transverse load distribution through cross-frames has an influence on girder deflections. Procedures developed by Fisher (2006) result in more accurate estimation compared with 1D line-girder analysis. Hence, girder rotational deformations and the corresponding variation in deck profile due to differential girder deflection can be calculated using the simplified procedures developed by Fisher (2006). Details and equations for these simplified procedures are provided in Chapter 4.

When the bearing point of an overhang bracket cannot be located closer to the bottom flange-web intersection, web out-of-plane deformation and the corresponding formwork rotation need to be
determined. However, there is no simplified procedure available for calculating web out-of-plane deformation and its impact on the deck profile.

Various approximate solutions and computer-based analysis tools are available in literature for calculating the rotational response of exterior girders under torsional loading. Hence, these models can be used to calculate girder rotation and evaluate its impact on the deck profile.

Structural behavior of curved and skewed bridges is addressed. When 1D and 2D analysis is not warranted for certain responses, 3D analysis is required. The connectivity and skew index can be used to evaluate the suitability of an analysis method for a given response.
3 OVERHANG BRACKET ANALYSIS

3.1 Overview

During cast-in-place concrete deck construction, the plastic concrete placed on deck overhang is supported by overhang brackets. Brackets also support the loads from walkway and construction equipment such as screed machines. Overhang bracket analysis is required to determine the loads acting on superstructure during deck placement. These loads are used in the analysis of differential girder deflection, web out-of-plane deformation, and exterior girder warping. Rational analysis of an overhang bracket requires modeling geometry, boundary conditions, and loads. This chapter describes modeling and analysis of an overhang bracket.

3.2 Analytical Model of an Overhang Bracket

Figure 3-1 shows the geometry of a typical overhang bracket. A beam and diagonal and vertical legs form the structural configuration of the bracket. Usually, diagonal and vertical legs are fabricated using circular hollow sections, and the bracket beam consists of two C-sections connected back to back. Diagonal and vertical legs are connected to each other and the beam with pins. No loads are acting in the direction transverse to the longitudinal axis of the diagonal and vertical legs. Hence, they are modeled as axially loaded members. On the other hand, the bracket beam is modeled as a 1D beam to support the loads acting on the overhang bracket. Overhang brackets are connected to steel I-girders using hanger rods that are attached to the girder top flange tip. Similar to bracket legs, hanger rods carry axial loads. Often, the angle between the hanger rod and the bracket is 45 degrees. Figure 3-2 shows an analytical model of a typical overhang bracket and the appropriate element types for the accurate representation of structural behavior.
3.3 Boundary Conditions

In a typical overhang bracket configuration, there is no contact between the bracket beam and the exterior girder web (Figure 3-3). Also, loads acting on the bracket beam tend to move the beam away from the exterior girder web. Hence, there is no need for a boundary condition definition at the interface of the girder web and bracket beam.

Hanger rods are either welded or clamped to the top flange of the exterior girder. Considering that the hanger rod is modeled as an axially loaded member, a pin support is defined at the top flange tip. A frictional force develops between the bracket and girder web in the vertical direction (Clifton et al. 2008) and constrains the vertical movement to a certain extent. Hence, the boundary condition at bracket bearing point is defined as pin support to develop a statically stable model.
The model shown in Figure 3-2 represents a statically determinate structure. When a contact is provided between the bracket beam and the girder web, an additional boundary condition is required at this location and the structure becomes statically indeterminate. Additional information such as material properties and geometry of the structural members are needed to analyze an indeterminate structure.

### 3.4 Loads

The bracket loads and the corresponding notations are shown in Figure 3-4 and Figure 3-5. Conservatively, loads acting beyond the centerline of the exterior girder are considered for bracket analysis. Eqs. (3-1) to (3-4) are used to calculate equivalent concentrated loads for bracket analysis.

\[
P_c = w_c t s b_{ov} s_b \tag{3-1}
\]

\[
P_{CLL} = w_{CLL}(b_{ov} + b_{sr} + b_{wa}) s_b \tag{3-2}
\]

\[
P_{of} = w_{of}(b_{ov} + b_{sr} + b_{wa}) s_b \tag{3-3}
\]

\[
P_{wa} = w_{wa} b_{wa} s_b \tag{3-4}
\]

where, \(P_c\) is concrete weight per bracket, \(P_{CLL}\) is construction live load per bracket, \(P_{of}\) is the combined overhang formwork and bracket load per bracket, and \(P_{wa}\) is walkway load per bracket.
Lastly, the screed machine load per bracket is needed for the analysis. Typical screed machines have either 4 or 8 wheels in total. The total screed machine weight ($P_{sm}$) is divided by the number of wheels ($n_w$) to obtain the load per wheel. However, a single bracket may be subjected to a larger load than the load per wheel based on bracket and wheel spacing. In other words, when the bracket spacing ($s_b$) is greater than the wheel spacing ($b_{sm}$), a single bracket carries a higher load than the load per wheel. Hence, Meadow Burke (2017) recommends amplifying the loads by the factors provided in Table 3-1. When more precise loads are needed, moving load or influence line analysis can be performed.
Table 3-1 Factors Recommended for the Maximum Screed Machine Load per Bracket (Source: Meadow Burke 2017)

<table>
<thead>
<tr>
<th>Ratio of bracket spacing to wheel spacing</th>
<th>( \frac{s_b}{b_{sm}} \leq 1 )</th>
<th>( 1 &lt; \frac{s_b}{b_{sm}} \leq 1.5 )</th>
<th>( 1.5 &lt; \frac{s_b}{b_{sm}} \leq 2.5 )</th>
<th>( s_b / b_{sm} &gt; 2.5 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor, ( f_{sm} )</td>
<td>1</td>
<td>1.25</td>
<td>1.5</td>
<td>1.75</td>
</tr>
</tbody>
</table>

Screed machine load per bracket is expressed as:

\[
P_{sm} = \frac{P_{tsm}}{n_w} f_{sm}
\]

where, \( P_{sm} \) is the screed machine load per bracket.

3.5 Bracket Analysis

Figure 3-6 shows analysis model geometry, element types, boundary conditions, and loads. Figure 3-7 shows the support reactions at point A and B of this statically determinate structure.

---

Figure 3-6 Bracket analysis model showing geometry, element types, loads, and boundary conditions.
Recalling that the angle between the hanger rod and bracket is 45 degrees, vertical and horizontal reactions at the top flange tip (point A) can be calculated considering the moment equilibrium at the bearing point (point B).

\[
R_{A,x} = R_{A,y} = \frac{\frac{P_c b_{ov}}{2} + \left(\frac{P_{cil} + P_{of}}{2}\right)(b_{ov} + b_{sr} + b_{wa}) + P_{sm} \left(\frac{b_{tw}}{2} + b_{ov}\right) + P_{wa} \left(\frac{b_{wa}}{2} + b_{sr} + b_{ov}\right)}{b_{fc}^2 + D + t_{fc} - H_{br}}
\]

Considering horizontal force equilibrium, the horizontal force acting on the girder web can be expressed as:

\[
R_{B,x} = R_{A,x}
\]

Overhang bracket analysis is needed to identify the loads acting on the exterior girder during deck placement. Since the interest is controlling girder deformations during deck placement, as per the AASHTO (2017a) Article 3.4.2.2, Service I limit state is considered and the load factor of 1.0 is used for component and construction loads.
4 DETERMINING DECK PROFILE UNDER GIRDER DEFORMATIONS

4.1 Overview

Maintaining intended deck profile as per project specifications requires controlling deformation of girders and formwork. Differential deflection between girders, exterior girder web out-of-plane deformation, exterior girder warping, or a combination thereof could impact the deck profile.

Figure 4-1 shows the most commonly observed deformation profile of a steel I-girder bridge superstructure when a screed machine is mounted on screed rails and fresh concrete is placed on the deck. Assuming that the overhang formwork and falsework are attached firmly to the exterior girder, the total rotation of an overhang bracket ($\theta_t$), as given in Eq. (4-1), can be expressed as the summation of the rotational deformation of the exterior girder due to differential deflection ($\theta_d$), rotational deformation of the exterior girder top flange due to web out-of-plane deformation ($\theta_{we}$), and rotational deformation of the exterior girder due to warping ($\theta_w$).

$$\theta_t = \theta_d + \theta_{we} + \theta_w \quad (4-1)$$

Once the total rotation is determined, Eq. (4-2) can be used for calculating the variation in the deck profile due to girder deformations during deck placement.

$$\Delta_{deck} = \left(\frac{b_{sr}}{2} + b_{ov}\right) \tan^{-1}(\theta_t) \quad (4-2)$$

In general, deck profile tolerances are specified considering the worst-case scenarios. Therefore, efforts provided in this chapter are focused on estimating the maximum rotational deformations. It should be realized that the maximum values of $\theta_d$, $\theta_{we}$, and $\theta_w$ are often at the same point along the span length, which is the midspan. If this is not the case, $\theta_d$, $\theta_{we}$, and $\theta_w$ should be calculated and superposed for each point along the span (typically at 1/10th location of the span) to estimate the controlling values of the total rotation.
4.2 Differential Deflection Between Girders

With fully fastened cross-frames and non-uniform girder deflection, girders are subjected to rotation in addition to vertical deflection. Overhang brackets also rotate with the exterior girders and may require implementing remedial measures to maintain the intended deck profile.

Fisher (2006) developed three different procedures for accurate estimation of girder deflections under non-composite loads: simplified procedure (SP), alternative simplified procedure (ASP), and single girder line straight line (SGLSL) procedure. Depending on span type and the difference between exterior-to-interior load ratios, the appropriate procedure needs to be employed to estimate girder deflections. Exterior-to-interior girder load ratio is the ratio of the loads acting on the exterior and interior girders. For a bridge under consideration, two separate ratios are calculated for each of the exterior girders. Fisher (2006) named exterior-to-interior girder load ratios as “equal” if the difference between them is less than 10%. When the difference exceeds this limit, the ratios are considered as unequal. SP procedure is used for calculating deflections in simple-span bridges when exterior-to-interior girder load ratios are equal. For simple-span bridges with unequal exterior-to-interior load ratios, ASP procedure is employed. SGLSL procedure was proposed for calculating deflections in continuous steel I-girder bridges when exterior-to-interior load ratios are equal. For continuous steel I-girder bridges with unequal load ratios, Fisher (2006) does not provide any procedure.

Unequal load ratios are common during phased construction, where phases are constructed separately and a closure pour is performed to achieve the structural integrity. Often, phases have unsymmetrical deck overhangs during the construction. To perform a smooth closure pour, deformation of phases needs to be controlled. Elevation differences between the phases may occur due to differential deflection of phases, as well as twisting of phases due to unsymmetrical overhangs. These closure pour issues are unique to phased construction scenarios, and they are not included within the scope of this thesis. Hence, a further discussion of the ASP procedure is not included.

The simplified procedures provided in Fisher (2006) are applicable under the following circumstances:

- Span length is less than 250 ft.
• Girder spacing is less than 11.5 ft.
• Number of girders is less than 10 (this limitation is only valid for ASP method).
• Girder spacing to span ratio is less than 0.08.

Since these limits encompass the majority of steel I-girder bridges in the U.S., procedures developed by Fisher (2006) have a broad range of applicability.

For a simple-span I-girder bridge with equal exterior-to-interior load ratios, Eqs. (4-3) and (4-4) can be used to determine the deflection profile of the bridge at any location along the span. AASHTO LRFD notations are used in these equations instead of the notations given in Fisher (2006).

\[
\Delta'_{ex} = [\Delta_{in} - (0.03 - a\theta)(100 - \eta_L)][1 - 0.1\tan(1.2\theta)] 
\]  \hspace{1cm} (4-3)

\[
\Delta_{dif} = \frac{\Delta_{in}}{\Delta_{in,m}} \left[ (3 - b\theta) \left(\frac{S}{L} - 0.04\right)(1 + z) - 0.1\tan(1.2\theta) \right] 
\]  \hspace{1cm} (4-4)

where

- \(b\) = -0.08, if \(\frac{S}{L} \leq 0.05\)
  = -0.08 + 8(\frac{S}{L} - 0.05), if \(0.05 < \frac{S}{L} \leq 8.2\)
- \(L\) = span length (ft)
- \(S\) = girder spacing (ft)
- \(z\) = \([10(\frac{S}{L} - 0.04) + 0.02](2 - \frac{\eta_L}{50})\)
- \(\alpha\) = 0.0002, if \(S \leq 8.2\ ft\)
  = 0.0002 + 0.000305(S-8.2), if \(8.2\ ft < S \leq 11.5\ ft\)
- \(\Delta_{dif}\) = differential deflection between girders (in.)
- \(\Delta'_{ex}\) = exterior girder deflection (in.)
- \(\Delta_{in}\) = interior girder deflection calculated using 1D line-girder analysis (in.)
- \(\Delta_{in,m}\) = interior girder midspan deflection calculated using 1D line-girder analysis (in.)
- \(\eta_L\) = exterior-to-interior girder load ratio (%)
- \(\theta\) = skew angle (deg)
  = the angle between the axis of support and a line normal to the longitudinal axis of the bridge

The negative values of \(\Delta_{dif}\) indicate that the exterior girders deflect more than interior girders. Maintaining an appropriate sign convention is important since the deflections and rotations are superposed to estimate the total rotation of the overhang bracket \((\theta_t)\).
Exterior-to-interior girder load ratio is needed for implementing Eqs. (4-3) and (4-4). Figure 4-2 shows the loads acting on girders during deck placement. Loads carried by individual girders can be calculated using the tributary area approach. For calculation simplicity, the weight of SIP form is assumed to be applied on the girder top flange. Assuming a uniform spacing between girders, the total load acting on an interior girder is calculated using Eq. (4-5).

\[
(w_c t_s + w_{CLL} + w_{SIP})SL
\]  

Likewise, the total load acting on an exterior girder is calculated as:

\[
\left[w_c t_s \left(b_{ov} + \frac{S}{2}\right) + w_{CLL} \left(b_{ov} + b_{sr} + b_{wa} + \frac{S}{2}\right) + w_{of} (b_{ov} + b_{sr} + b_{wa}) + w_{wa} b_{wa} + w_{SIP} \frac{S}{2}\right] L + \frac{P_{tsm}}{2} \]  

Thus, the exterior-to-interior girder load ratio, \( \eta_L \), is expressed as:

\[
\left[w_c t_s \left(b_{ov} + \frac{S}{2}\right) + w_{CLL} \left(b_{ov} + b_{sr} + b_{wa} + \frac{S}{2}\right) + w_{of} (b_{ov} + b_{sr} + b_{wa}) + w_{wa} b_{wa} + w_{SIP} \frac{S}{2}\right] L + \frac{P_{tsm}}{2} \times 100
\]  

Since the maximum differential deflection occurs at midspan of simple-span bridges, the maximum variation in the deck profile due to differential girder deflection also occurs at the midspan. Midspan deflection of an interior girder (\( \Delta_{in,m} \)) can be expressed as:

\[
\Delta_{in,m} = \frac{5(w_c t_s + w_{CLL} + w_{SIP})SL^4}{384EI_x}
\]  

Additionally, for straight I-girder bridges, terms with the skew angle are eliminated. Implementing these conditions, Eqs. (4-3) and (4-4) are rewritten and shown in Eqs. (4-9) and (4-10).

\[
\Delta'_{ex} = \left[\frac{5(w_c t_s + w_{CLL} + w_{SIP})SL^4}{384EI} - 0.03(100 - \eta_L)\right]
\]  

![Figure 4-2 Loads acting on girders during deck placement](image)
\[ \Delta_{dif} = \left[ 3 \left( \frac{S}{L} - 0.04 \right) (1 + z) \right] \]  

(4-10)

Finally, rotation of an exterior girder and the corresponding bracket rotation due to girder differential deflection in a simple-span steel I-girder bridge is calculated as:

\[ \theta_d = \tan^{-1}\left( \frac{-\Delta_{dif}}{S} \right) \]  

(4-11)

The SGLSL procedure was developed for the estimation of girder deflections in continuous steel I-girder bridges with equal exterior-to-interior load ratios. In this procedure, exterior girder deflections are calculated using 1D line-girder analysis for the locations under interest. Then, calculated exterior girder deflections are taken as interior girder deflections. In other words, the SGLSL procedure results in a straight-line deflection profile throughout the bridge cross-section, implying that there is no differential deflection between the girders. Although the deflected shape of continuous span bridges tends to be flat in general, the SGLSL procedure over-simplifies the behavior. It indirectly states that the when difference between the exterior-to-interior girder load ratios is up to 10\%, in other words, exterior girders are subjected to the same amount of loads, there is no differential deflection. The procedure may be used as an approximate tool; however, a refined analysis is required for estimating rotation of exterior girders in continuous bridges due to differential girder deflection.

4.3 Web Out-of-Plane Deformation

4.3.1 Overview

As illustrated in Figure 4-3, the horizontal component of the bracket axial force acts on the exterior girder web as a lateral load at the bearing point. The procedure for calculating this lateral load is discussed in Section 3.5. The bearing point cannot be located close to the bottom flange-web intersection due to deep exterior girders, inappropriate bracket sizes, or combination thereof. Under these circumstances, web out-of-plane deformation and the corresponding bracket rotation need to be determined. However, other than using 3D finite element method, an analysis procedure is not presented in literature for calculating web out-of-plane deformation. Hence, a procedure is developed using the theory of thin plates with small deformations and presented in this thesis.
4.3.2 Analysis of an Exterior Girder Web Deformation using Theory of Thin Plates

Plates are often considered in three main groups: thin plates with small deflections, thin plates with large deflections, and thick plates. A plate is classified as a thin plate if its smallest dimension is less than 1/20 of other two dimensions. Small deformation is a simplifying yet an accurate assumption for analyses of plates commonly used in structures (Ugural 2010). Fundamental assumptions of small deformation theory, also known as Kirchhoff hypotheses, are summarized as follows:

- Comparing with the plate thickness, the deflection of the midsurface is small.
- No strain develops in midplane due to the bending.
- Plane sections remain plane after the bending.
- Comparing with other stress components, the stress normal to the midplane is small.

The governing differential equation for deflection of plates was first derived by Lagrange as follows:

\[
\frac{\partial^4w}{\partial x^4} + 2 \frac{\partial^4w}{\partial x^2 \partial y^2} + \frac{\partial^4w}{\partial y^4} = \frac{p}{D_f}
\]  

(4-12)

where, \(w\) is the deflection of the plate, \(p\) is uniformly distributed load per unit area, and \(D_f\) is the flexural rigidity of the plate, which is calculated as:

\[
D_f = \frac{Et^3}{12(1 - \nu^2)}
\]  

(4-13)

where, \(E\) is the modulus of elasticity and \(\nu\) is Poisson’s ratio.
For a plate simply supported on all edges and having sides of \(a\) and \(b\), Eq. (4-12) should satisfy the following boundary conditions, implying that deflection and bending moment should be zero along simply supported edges:

\[
w = 0, \quad \frac{\partial^2 w}{\partial x^2} = 0 \quad (x = 0, \ x = a)
\]

\[
w = 0, \quad \frac{\partial^2 w}{\partial y^2} = 0 \quad (y = 0, \ y = b)
\]

In 1820, Navier proposed a solution for Eq. (4-12) using Fourier series expansions. Figure 4-4 shows a rectangular plate with a thickness, \(t\), and side lengths of \(a\) and \(b\), under a concentrated load, \(P\), acting on point \((x_1, y_1)\). Using Navier’s approach to the problem, deflection of any point \((x, y)\) on the plate surface calculated as:

\[
w(x, y) = \frac{4P}{\pi^4 D_f ab} \sum_{m} \sum_{n} \frac{\sin \left(\frac{m\pi x_1}{a}\right) \sin \left(\frac{n\pi y_1}{b}\right)}{\left(\left(\frac{m}{a}\right)^2 + \left(\frac{n}{b}\right)^2\right)^2} \sin \left(\frac{m\pi x}{a}\right) \sin \left(\frac{n\pi y}{b}\right)
\]

Assuming that the exterior girder web is encased and supported by girder flanges and transverse stiffeners, an analogy can be developed between the deflection of thin plates under concentrated loads and an exterior girder web out-of-plane deformation due to lateral loads. Thus, Eq. (4-16)
can be manipulated to reflect this analogy. An overhang bracket occupies constant vertical location along the exterior girder length. The maximum lateral deformation of the exterior girder web and the corresponding maximum rotation of bracket occurs when the bracket is located at the middle of the consecutive transverse stiffeners. Implementing these into Eq. (4-16), the lateral deformation profile of the exterior girder web along the web depth can be expressed as:

\[
w(x) = \frac{4R_{B,x}}{\pi^4D_fDd_o} \sum_{m} \sum_{n} \sin\left(\frac{m\pi(D - H_{br})}{D}\right) \sin\left(2\frac{n\pi}{2}\right) \sin\left(\frac{m\pi x}{D}\right) \sin\left(n\frac{\pi}{2}\right)\]

(4-17)

where, \(R_{B,x}\) is the lateral load acting on the exterior girder web, \(D\) is the web depth, \(d_o\) is the spacing between two consecutive transverse stiffeners, \(H_{br}\) is the overhang bearing point measured from the top of the bottom flange, and \(D_f\) is the flexural rigidity of the plate, which is calculated as:

\[
D_f = \frac{Et_{w}^3}{12(1 - v^2)}
\]

(4-18)

in which \(t_{w}\) is the web thickness. The accuracy of Eq. (4-17) improves as the number of terms used in the series increases. After several iterations, it was decided to set \(m\) and \(n\) values to 10.

To determine the effect of web out-of-plane deformation on deck profile, rotation of overhang bracket needs to be determined. Brackets are attached to the exterior girder at top flange. Assuming a rigid connection between the bracket and the exterior girder, rotation of the bracket is equal to rotation of the top flange. Rotation of any point under interest along the web depth can be obtained by calculating the slope of the deformation profile at this point. Hence, bracket rotation can be calculated by differentiating Eq. (4-17) at \(x = 0\):

\[
\theta_{we} = \frac{\partial w(0)}{\partial x}
\]

(4-19)

With multiple brackets located between consecutive transverse stiffeners, associated top flange rotation can be calculated at each bracket location using Eqs. (4-16) and (4-19). Then, the values can be superposed to obtain the maximum rotation.

Eq. (4-17) is valid when the web is simply supported along all the edges. However, the actual boundary conditions provided by flanges and transverse stiffeners are different from this
idealization. Therefore, the flange rotation calculated using the plate theory needs to be calibrated. Section 4.3.3 presents the calibration process.

4.3.3 Finite Element Analysis of an Exterior Girder with Transverse Stiffeners

4.3.3.1 Model Description

To verify the procedure presented in Section 4.3.2 and calibrate the solution considering the actual boundary conditions provided by flanges and stiffeners, linear elastic finite element analyses are performed assuming stresses developed in girders during the construction are well below the yielding limits. Analyses are performed using ABAQUS finite element software suite (ABAQUS 2014).

In analyses, eight different simply-supported three-dimensional doubly-symmetric girders are modeled. Cross-section of girders and transverse stiffeners are proportioned by following the preliminary design guidelines presented in AASHTO (2017a) and NSBA (2016b). This is a required approach for developing the calibration factors in accordance with the practical design range of girders. In analyses, four different span lengths are considered: 50, 100, 150, and 200 feet. First, for each of span lengths, four different girder models are developed considering the following parameters:

- **Web Depth (D)**
  
  As per the AASHTO (2017a) Table 2.5.2.6.3-1, the minimum web depth shall be $0.033L$ for simply-supported composite I-girder bridges, where $L$ is the span length.

- **Web Thickness ($t_w$)**
  
  As per the AASHTO (2017a) Eq. 6.10.2.1.1-1, for webs without longitudinal stiffeners, the minimum web thickness shall satisfy:

  \[
  \frac{D}{t_w} \leq 150
  \]  

  (4-20)

  In addition to that, NSBA (2016b) recommends a minimum web thickness of 0.5 inches. For each girder, the governing condition is used to determine the web thickness.

- **Flange Width ($b_f$)**
  
  As per the AASHTO (2017a) Eq. 6.10.2.2-2, the minimum flange width, $b_f$, shall satisfy:
The AASHTO (2017a) Article C6.10.3.4 provides additional guidance regarding the minimum top flange width. This additional guidance assures more stable and convenient handling of girder segments and limiting the out-of-plane distortions of the compression flange and web during deck placement. As per the Eq. C6.10.3.4-1, the minimum top flange width shall satisfy:

\[ b_f \geq \frac{D}{6} \]  \hspace{1cm} (4-21)

where, \( L \) is the length of girder shipping piece. In this thesis, length of girder shipping piece is taken as span length.

- **Flange Thickness \((t_f)\)**

As per the AASHTO (2017a) Eq. 6.10.2.2-2, the minimum flange thickness, \( t_f \), shall satisfy:

\[ t_f \geq 1.1t_w \]  \hspace{1cm} (4-23)

In addition to that, NSBA (2016b) recommends a minimum web thickness of 0.75 inches. For each girder, the governing condition is used to determine the flange thickness.

- **Transverse Stiffener Spacing \((d_o)\)**

The AASHTO (2017a) Article C6.10.2.1.1 states that by applying Eq. (4-20), the transverse stiffener spacing up to \( 3D \) is permitted. Additionally, the Article C6.10.9.3.3 states that spacing from the support to the first stiffener shall not exceed \( 1.5D \). A uniform stiffener spacing is selected based on the maximum of \( 1.5D \) limitation to reduce modeling efforts.

- **Transverse Stiffener Thickness \((t_p)\)**

NSBA (2016b) recommends a minimum transverse stiffener thickness of 0.5 inches.

- **Transverse Stiffener Depth**

For all girder models, full depth transverse stiffeners are used.

- **Transverse Stiffener Width \((b_t)\)**
As per the AASHTO (2017a) Article 6.10.11.1.2, transverse stiffener width shall satisfy the following equations simultaneously:

\[ b_t \geq 2 + \frac{D}{30} \]  \hspace{1cm} (4-24)

\[ 16t_p \geq b_t \geq \frac{b_f}{4} \]  \hspace{1cm} (4-25)

Dimensions calculated by following the above stated guidelines are rounded up to the nearest quarter of an inch. Table 4-1 summarizes the geometric properties of girder models G1 to G4.

<table>
<thead>
<tr>
<th>Girder</th>
<th>L (ft)</th>
<th>D (in.)</th>
<th>t_w (in.)</th>
<th>b_f (in.)</th>
<th>t_f (in.)</th>
<th>d_o (in.)</th>
<th>t_p (in.)</th>
<th>b_t (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>50</td>
<td>20.00</td>
<td>0.50</td>
<td>7.25</td>
<td>0.75</td>
<td>30</td>
<td>0.5</td>
<td>2.75</td>
</tr>
<tr>
<td>G2</td>
<td>100</td>
<td>39.75</td>
<td>0.50</td>
<td>14.25</td>
<td>0.75</td>
<td>50</td>
<td>0.5</td>
<td>3.75</td>
</tr>
<tr>
<td>G3</td>
<td>150</td>
<td>59.50</td>
<td>0.50</td>
<td>21.25</td>
<td>0.75</td>
<td>75</td>
<td>0.5</td>
<td>5.50</td>
</tr>
<tr>
<td>G4</td>
<td>200</td>
<td>79.25</td>
<td>0.75</td>
<td>28.25</td>
<td>1.00</td>
<td>100</td>
<td>0.5</td>
<td>7.25</td>
</tr>
</tbody>
</table>

Inherently, Eq. (4-17) accounts for the effects of web slenderness and stiffener spacing. To investigate the effects of flange width, flange thickness, stiffener width, and stiffener thickness, four additional girder models are developed by modifying the girder G4. Each of the parameters, i.e., flange width, flange thickness, stiffener width, and stiffener thickness, are approximately increased by 25%. Table 4-2 summarizes the geometric properties of girder models G5 to G8.

<table>
<thead>
<tr>
<th>Girder</th>
<th>L (ft)</th>
<th>D (in.)</th>
<th>t_w (in.)</th>
<th>b_f (in.)</th>
<th>t_f (in.)</th>
<th>d_o (in.)</th>
<th>t_p (in.)</th>
<th>b_t (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>G5</td>
<td>200</td>
<td>79.25</td>
<td>0.75</td>
<td>35.31</td>
<td>1.00</td>
<td>100</td>
<td>0.5</td>
<td>7.25</td>
</tr>
<tr>
<td>G6</td>
<td>200</td>
<td>79.25</td>
<td>0.75</td>
<td>28.25</td>
<td>1.25</td>
<td>100</td>
<td>0.5</td>
<td>7.25</td>
</tr>
<tr>
<td>G7</td>
<td>200</td>
<td>79.25</td>
<td>0.75</td>
<td>28.25</td>
<td>1.00</td>
<td>100</td>
<td>0.63</td>
<td>7.25</td>
</tr>
<tr>
<td>G8</td>
<td>200</td>
<td>79.25</td>
<td>0.75</td>
<td>28.25</td>
<td>1.00</td>
<td>100</td>
<td>0.5</td>
<td>9.06</td>
</tr>
</tbody>
</table>

I-sections and transverse stiffeners are modeled using 3D 4-node shell elements. For computational efficiency, reduced integration elements are used. In ABAQUS element library, such elements are named as S4R, and they are general purpose elements appropriate for modeling of thin plates (ABAQUS 2014). S4R elements have 6 degrees of freedom at each node: three translational and three rotational degrees of freedoms. Transverse stiffeners are rigidly connected to I-sections using tie constraints at each contact surface between girders and stiffeners. Mesh refinement is decided by performing a series of mesh convergence trials.
Simply supported boundary conditions are provided by assigning pin and roller supports along the bottom flanges at girder ends. Since finite element analyses are performed to investigate the web out-of-plane deformations, torsional rotation of girders is prevented by providing continuous lateral restraints along the top and bottom flange edges. Further, providing lateral restraints at flange edges eliminates the buckling of girders.

First, to generate the maximum rotational response, brackets are assumed to be located at the middle of consecutive transverse stiffeners. Component and construction loads result in lateral loads acting on both top flange and bearing point of the bracket. Hence, a unit load of 1 kip is applied to top flange and to a predefined point along the girder web. Five different loading cases are defined to apply lateral loads to girders. Under each load case, flanges are loaded at the same point, however, web lateral loads are applied at 0.5D, 0.6D, 0.7D, 0.8D, or 0.9D along the web depth (Figure 4-5). Locating the bracket bearing point above the mid-depth of the web is not likely a construction practice, thus, the above stated load points are considered.

Secondly, to investigate the behavior when brackets are not located at the middle of consecutive stiffeners, the girder G4 is analyzed for various bracket locations. In these cases, brackets are assumed to be located at 15 in., 25 in., and 35 in. away from the left transverse stiffeners (Figure 4-6). For each bracket location, lateral loads are applied at the various point along the web depth as described earlier.
4.3.3.2 Discussion of Results

The ability of the plate theory for capturing the behavior of exterior girder web under lateral loads needs to be verified. This can be accomplished by plotting the out-of-plane deformation profile of the web obtained from finite element analyses and plate theory solution. Figure 4-7 includes deformation curves along the web depth of the girder G4 loaded at 0.5D, 0.7D and 0.9D obtained from finite element analyses and plate solution. Deformation profiles obtained from the plate theory and finite element analyses follow identical trends. Hence, the theory of thin plates with small deformations can accurately address the behavior of the exterior girder web under lateral loads. However, solutions need to be calibrated considering actual boundary conditions provided by flanges and stiffeners.
The main objective is to calculate the rotation of overhang brackets, which is equal to the rotation of the top flange when a rigid connection is assumed between the bracket and exterior girder. Thus, as shown in Figure 4-8, the maximum flange rotation about the longitudinal axis of the girder is requested for each finite element analysis. Then, using Eqs. (4-17) and (4-18), the same response is calculated for each girder and loading case. Table 4-3, Table 4-4, and Table 4-5 summarize the rotational deformation of the top flange obtained from the plate theory solution and finite element analysis for each loading case, as well as the ratio of FEA results to plate theory solutions. In these tables, rotational deformations are reported in terms of radians.
Table 4-3 Top Flange Rotations about Girder Longitudinal Axis, Girders G1 to G4

<table>
<thead>
<tr>
<th>Load Case</th>
<th>FE (rad)</th>
<th>Plate Theory (rad)</th>
<th>Ratio</th>
<th>FE (rad)</th>
<th>Plate Theory (rad)</th>
<th>Ratio</th>
<th>FE (rad)</th>
<th>Plate Theory (rad)</th>
<th>Ratio</th>
<th>FE (rad)</th>
<th>Plate Theory (rad)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5D</td>
<td>0.00056</td>
<td>0.00245</td>
<td>0.229</td>
<td>0.00086</td>
<td>0.00437</td>
<td>0.197</td>
<td>0.00124</td>
<td>0.00655</td>
<td>0.189</td>
<td>0.00068</td>
<td>0.00259</td>
<td>0.263</td>
</tr>
<tr>
<td>0.6D</td>
<td>0.00042</td>
<td>0.00205</td>
<td>0.205</td>
<td>0.00064</td>
<td>0.00361</td>
<td>0.177</td>
<td>0.00090</td>
<td>0.00541</td>
<td>0.166</td>
<td>0.00051</td>
<td>0.00214</td>
<td>0.238</td>
</tr>
<tr>
<td>0.7D</td>
<td>0.00027</td>
<td>0.00161</td>
<td>0.167</td>
<td>0.00042</td>
<td>0.00280</td>
<td>0.150</td>
<td>0.00060</td>
<td>0.00421</td>
<td>0.143</td>
<td>0.00035</td>
<td>0.00166</td>
<td>0.211</td>
</tr>
<tr>
<td>0.8D</td>
<td>0.00012</td>
<td>0.00109</td>
<td>0.110</td>
<td>0.00021</td>
<td>0.00189</td>
<td>0.111</td>
<td>0.00031</td>
<td>0.00283</td>
<td>0.110</td>
<td>0.00019</td>
<td>0.00121</td>
<td>0.170</td>
</tr>
<tr>
<td>0.9D</td>
<td>≈ 0</td>
<td>0.00056</td>
<td>≈ 0</td>
<td>0.00003</td>
<td>0.00096</td>
<td>0.031</td>
<td>0.00008</td>
<td>0.00144</td>
<td>0.056</td>
<td>0.00006</td>
<td>0.00057</td>
<td>0.105</td>
</tr>
</tbody>
</table>

Table 4-4 Top Flange Rotations about Girder Longitudinal Axis, Girders G5 to G8

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Plate Theory (rad)</th>
<th>FE (rad)</th>
<th>Ratio</th>
<th>Plate Theory (rad)</th>
<th>FE (rad)</th>
<th>Ratio</th>
<th>Plate Theory (rad)</th>
<th>FE (rad)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5D</td>
<td>0.00259</td>
<td>0.00061</td>
<td>0.236</td>
<td>0.00047</td>
<td>0.181</td>
<td>0.251</td>
<td>0.00065</td>
<td>0.251</td>
<td>0.232</td>
</tr>
<tr>
<td>0.6D</td>
<td>0.00214</td>
<td>0.00046</td>
<td>0.215</td>
<td>0.00036</td>
<td>0.168</td>
<td>0.229</td>
<td>0.00049</td>
<td>0.199</td>
<td>0.206</td>
</tr>
<tr>
<td>0.7D</td>
<td>0.00166</td>
<td>0.00031</td>
<td>0.187</td>
<td>0.00024</td>
<td>0.145</td>
<td>0.199</td>
<td>0.00033</td>
<td>0.161</td>
<td>0.175</td>
</tr>
<tr>
<td>0.8D</td>
<td>0.00112</td>
<td>0.00017</td>
<td>0.152</td>
<td>0.00013</td>
<td>0.116</td>
<td>0.161</td>
<td>0.00018</td>
<td>0.161</td>
<td>0.134</td>
</tr>
<tr>
<td>0.9D</td>
<td>0.00057</td>
<td>0.00004</td>
<td>0.070</td>
<td>0.00004</td>
<td>0.070</td>
<td>0.088</td>
<td>0.00005</td>
<td>0.088</td>
<td>0.070</td>
</tr>
</tbody>
</table>

Table 4-5 Top Flange Rotations about Girder Longitudinal Axis for Various Bracket Locations, Girder G4

<table>
<thead>
<tr>
<th>Position of the bracket from the stiffener</th>
<th>15 in.</th>
<th>25 in.</th>
<th>35 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FEA (rad)</td>
<td>Plate Theory (rad)</td>
<td>Ratio</td>
</tr>
<tr>
<td>0.5D</td>
<td>0.00023</td>
<td>0.00092</td>
<td>0.250</td>
</tr>
<tr>
<td>0.6D</td>
<td>0.00018</td>
<td>0.00069</td>
<td>0.261</td>
</tr>
<tr>
<td>0.7D</td>
<td>0.00012</td>
<td>0.00051</td>
<td>0.235</td>
</tr>
<tr>
<td>0.8D</td>
<td>0.00006</td>
<td>0.00032</td>
<td>0.188</td>
</tr>
<tr>
<td>0.9D</td>
<td>0.00002</td>
<td>0.00016</td>
<td>0.125</td>
</tr>
</tbody>
</table>
Referring to Table 4-3, Table 4-4, and Table 4-5, ratios of FEA results and plate theory solutions are affected by the load application point along the web depth. Ratios are maximum when the lateral load is applied at 0.5D. As the point of application moves towards the bottom flange, ratios significantly decrease. Therefore, plate theory solutions need to be calibrated considering the load application point. In other words, based on the bracket bearing point, different calibration factors need to be used to calculate the top flange rotation.

The plate theory solution does not account for the effects of flange width, flange thickness, stiffener width, and stiffener thickness on rotational fixity provided by flanges and stiffeners. Effects of these parameters can be investigated comparing the FEA results of the girder G4 and the girders G5 to G8. Overall, any dimensional increase in these parameters increased the rotational fixity provided to the web. Among the parameters, the flange thickness has the most dominant effect. Increasing the flange thickness by 25% resulted in approximately 30% decrease in the top flange rotation. Flange width has also a significant effect on rotational fixity. Increasing the flange width by 25% resulted in approximately 10% decrease in the top flange rotation. Effect of the stiffener width could not be quantified with a constant value, however, FEA results clearly indicated that increasing the stiffener width significantly affect the rotational behavior of the top flange. Stiffener thickness has the least dominant effect, increasing the stiffener thickness by 25% resulted in approximately 5% decrease in the top flange rotation.
Based on the finite element analyses results, the factors given in Table 4-6 are proposed as scale factors to multiply the plate theory results for calculating the overhang bracket rotation due to web out-of-plane deformation. Although the maximum top flange rotation occurs when brackets are located at the middle of two consecutive transverse stiffeners, suggested scale factors can also be used when the bracket is at other locations. Girder models for finite element analyses were created following the requirements of AASHTO (2017a) and NSBA (2016b). While satisfying those requirements, the minimum dimensions were selected for girder cross-section and stiffeners. Comparison of FEA results of the girders G4 and G5 to G8 indicates that any dimensional increase in girder cross-section or stiffeners results in smaller rotational deformation of the top flange and the smaller ratio of FEA results to plate theory solutions. Thus, suggested scale factors provide a conservative estimate for the girders that are designed as per the AASHTO and NSBA requirements. The scale factors can also be used for curved and skewed girders. The spacing between two consecutive transverse stiffeners is small compared to girder length. Hence, a portion of the exterior girder web that is encased by flanges and stiffeners can be assumed as straight and not affected by curvature. The interface between the overhang bracket and exterior girder is not affected by skew, i.e., the bracket occupies a line parallel to web depth.

<table>
<thead>
<tr>
<th>Load Application Point</th>
<th>Factor, $\alpha_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.5D \leq D - H_{br} &lt; 0.6D$</td>
<td>0.300</td>
</tr>
<tr>
<td>$0.6D \leq D - H_{br} &lt; 0.7D$</td>
<td>0.275</td>
</tr>
<tr>
<td>$0.7D \leq D - H_{br} &lt; 0.8D$</td>
<td>0.250</td>
</tr>
<tr>
<td>$0.8D \leq D - H_{br} &lt; 0.9D$</td>
<td>0.200</td>
</tr>
<tr>
<td>$0.9D \leq D - H_{br} &lt; D$</td>
<td>0.125</td>
</tr>
</tbody>
</table>

Another conclusion can be made regarding the guidance provided by various agencies for controlling web out-of-plane deformations. Finite element analyses revealed that when the bracket bearing point is located at 0.9D, rotation of the top flange is approximately zero for all girder models. Thus, limits provided by agencies are verified to be accurate.

### 4.3.4 Summary

During deck placement, overhang brackets exert lateral load to the exterior girder web at the bearing point that may result in web out-of-plane deformations and consequently a poor deck profile. Several highway agencies provide guidance in their specifications for controlling web out-of-plane deformation. Commonly, the guidance suggests locating the bracket bearing point close
to the bottom flange. For cases this cannot be achieved, overhang bracket rotation due to web out-of-plane deformation needs to be evaluated. However, other than using 3D finite element method, an analysis procedure is not presented in literature for calculating web out-of-plane deformation. In developing a simplified solution, the problem was approached using the theory of thin plates with small deformations. Theory fundamentals and calculation procedures were provided for thin plates with simply supported edges. Then, finite element analyses were performed to demonstrate that plate theory solution can accurately represent the behavior of exterior girder web under lateral loads. Finally, scale factors were developed to account for the impact of restraints provided by flanges and transverse stiffeners. Using Eqs. (4-17), (4-18), and (4-19) with an appropriate scale factor \((\alpha_c)\), rotation of the overhang bracket due to web out-of-plane deformation \((\theta_{we})\) can be calculated.

4.4 Exterior Girder Warping

A girder is under pure bending if loads are applied through its shear center. If the applied loads do not coincide with the shear center, in addition to bending moment, torsion develops. When a girder is under torsional loading, in addition to twist, it may warp. As illustrated in Figure 4-9, girder cross-section does not remain in the plane under warping. Depending on how the girder resists to the applied torque, torsion concept can be divided into two categories, uniform and non-uniform torsion. In uniform torsion, the member is allowed to warp freely, and the applied torque is resisted by St. Venant resisting moments. If the warping is restrained, then the applied torsional moment is resisted by summation of St. Venant and warping resisting moments. This phenomenon is called as non-uniform torsion.

![Figure 4-9 I-girder warping](image)

Figure 4-9 I-girder warping
During deck placement, component and construction loads act eccentrically to the exterior girder. If the cross-section warping is not allowed at cross-frame locations, the exterior girder is subjected to a non-uniform torsion between consecutive cross-frames. Overhang bracket rotation combined with exterior girder warping and results in unintended deck profile. Assuming a full restraint against warping at the cross-frame location is an idealization. The actual warping fixity provided by cross-frames is somewhere between fixed and pinned boundary conditions (KDOT 2016).

In literature, resources are available regarding the torsional analysis of girders. AISC (2003) provides detailed discussions about the fundamentals of the theory, as well as the analyses implementations with respect to various loading scenarios and boundary conditions. While adopting these, modifications are incorporated to solutions and notations to reflect the bridge terminology and the task at hand.

When the warping of a girder is restrained, the total torsional moment resisted by the cross-section \((T)\) is expressed as the summation of St. Venant and warping resisting moments:

\[
T = T_t + T_w = GJ\theta' - EC_w\theta'''
\]

where, \(T_t\) is St. Venant resisting moment, \(T_w\) is warping resisting moment, \(G\) is the shear modulus of elasticity, \(E\) is the modulus of elasticity, and \(\theta'\) and \(\theta'''\) are the first and third derivate of the total torsional rotation about the longitudinal axis along the girder length. Torsional and warping constant of the cross-section are \(J\) and \(C_w\), respectively, and for singly and doubly symmetric I-sections, they are calculated as follows:

\[
J = \frac{b_{fc}t_{fc}^3 + b_{ft}t_{ft}^3 + Dt_w^3}{3}
\]

\[
C_w = \frac{\left(\frac{t_{fc} + t_{ft}}{2}\right) \left(D + \frac{t_{fc}}{2} + \frac{t_{ft}}{2}\right)^2}{12} \left(\frac{b_{fc}^3b_{ft}^3}{b_{fc}^3 + b_{ft}^3}\right)
\]

where, \(b_{fc}\) and \(b_{ft}\) are the top and bottom flange width and \(t_{fc}\) and \(t_{ft}\) are the top and bottom flange thickness. Eq. (4-26) can be simplified as:

\[
\frac{T}{EC_w} = \frac{\theta'}{a^2} - \theta'''
\]

where
\[ a^2 = \frac{EC_w}{6J} \]  

(4-30)

During deck placement, the exterior girder is under torsional moment due to eccentric component and construction loads applied at each overhang bracket location. This loading configuration can be idealized as a uniformly distributed torsional moment along the girder length. Thus, Eq. (4-29) needs to be rearranged to account for the uniformly distributed torsional moment \((T_u)\). Figure 4-10 shows a girder segment under uniform torsional moment with a length of \(dz\). From the equilibrium of torsional moments:

\[ T + dT_u + T_u dz - T = 0 \]  

(4-31)

Further simplification yields:

\[ \frac{dT}{dz} = -T_u \]  

(4-32)

**Figure 4-10** A girder segment under uniform torsional moment

Differentiating Eq. (4-29) and inserting into Eq. (4-32) yields:

\[ \frac{-T_u}{EC_w} = \frac{\theta''}{a^2} - \theta'''' \]  

(4-33)

For girders under uniformly distributed torsional moments, Eq. (4-33) is the governing differential equation for torsion. Since the actual warping fixity provided by cross-frames is somewhere between fixed and pinned boundary conditions, Eq. (4-33) needs to be solved separately considering different boundary conditions. Table 4-7 describes torsional boundary conditions, their physical meanings, and corresponding mathematical expressions.
<table>
<thead>
<tr>
<th>Physical Condition</th>
<th>Torsional End Condition</th>
<th>Mathematical Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>No rotation</td>
<td>Fixed or Pinned</td>
<td>$\theta = 0$</td>
</tr>
<tr>
<td>Warping restrained</td>
<td>Fixed</td>
<td>$\theta' = 0$</td>
</tr>
<tr>
<td>Warping allowed</td>
<td>Pinned of Free</td>
<td>$\theta'' = 0$</td>
</tr>
</tbody>
</table>

In addition to the mathematical conditions, solutions of Eq. (4-33) should satisfy compatibility conditions. For a point under consideration, the torsional rotation ($\theta$) and its first and second derivatives should be equal from the left and right side of the point.

Solutions of Eq. (4-33) for girders with fixed-fixed and pinned-pinned boundary conditions at their ends are given in Eq. (4-34) and (4-35), respectively.

$$\theta_f = \frac{T_u l a}{2GJ} \left[ \left( 1 + \frac{\cosh \frac{l}{a}}{\sinh \frac{l}{a}} \right) \left( \cosh \frac{z}{a} - 1 \right) + \frac{z}{a} \left( 1 - \frac{z}{l} \right) - \sinh \frac{z}{a} \right] \quad (4-34)$$

$$\theta_p = \frac{T_u a^2}{GJ} \left[ \frac{l^2}{2a^2} \left( \frac{z}{l} - \frac{z^2}{l^2} \right) + \cosh \frac{z}{a} - \tanh \left( \frac{l}{2a} \right) \sinh \left( \frac{z}{a} \right) - 1 \right] \quad (4-35)$$

In these equations, $T_u$ is the uniformly distributed torsional moment, $l$ is the span length, and $z$ is the distance from the left support along the girder length. Recalling from Section 3.5, the uniform torsional moment ($T_u$) can be expressed as:

$$T_u = \frac{P_c b_{ov} + (P_{CLL} + P_{of})(b_{ov} + b_{sr} + b_{wa}) + P_{sm}(b_{sr} + b_{ov}) + P_{wa}(b_{wa} + b_{sr} + b_{ov})}{l} \quad (4-36)$$

Assuming cross-frames provide torsional restraints, span length ($l$) becomes unbraced length ($L_b$) of the exterior girder. Further, since the maximum torsional rotation occurs at the middle of supports, $z$ can be expressed as half of the unbraced length. Implementing these into Eqs. (4-34), (4-35), and (4-36) yields:

$$\theta_f = \frac{T_u L_b a}{2GJ} \left[ \left( 1 + \frac{\cosh \frac{L_b}{a}}{\sinh \frac{L_b}{a}} \right) \left( \cosh \frac{L_b}{2a} - 1 \right) + \frac{L_b}{4a} - \sinh \frac{L_b}{2a} \right] \quad (4-37)$$

$$\theta_p = \frac{T_u a^2}{GJ} \left[ \frac{L_b^2}{8a^2} + \cosh \frac{L_b}{2a} - \tanh \left( \frac{L_b}{2a} \right) \sinh \left( \frac{L_b}{2a} \right) - 1 \right] \quad (4-38)$$
\[ T_u = \frac{P_e b_{ov}}{2} + \frac{(P_{CLL} + P_{of})(b_{ov} + b_{sr} + b_{wa})}{2} + \frac{P_{sm}(b_{sr}^2 + b_{ov})}{2} + P_{wa}\left(\frac{b_{wa}}{2} + b_{sr} + b_{ov}\right) \]  \hspace{1cm} (4-39)

Eqs. (4-37) and (4-38) provides upper and lower bounds for the rotation of exterior girders due to warping. It should be realized that these equations do not account for the increased torsional stiffness characteristic of the girder due to transverse stiffeners, as well as the effects of curvature and skew. Considering these, engineering judgment is required while determining the rotational deformation of an exterior girder due to warping.

4.5 Summary

Determining deck profile under girder deformations is required before deck placement so that a check can be performed whether deck finish tolerances will be satisfied. This can be fulfilled by understanding and analyzing the girder behavior under various loads and boundary conditions.

- To calculate exterior girder rotation and the corresponding bracket rotation due to differential girder deflection, simplified procedures developed by Fisher (2006) were employed. The SP procedure provides an accurate estimation of girder differential deflection for simple-span steel I-girder bridges. The procedure accounts for transverse load distribution through cross-frames. The SGLSL procedure oversimplifies the behavior of continuous bridges.

- To calculate web out-of-plane deformation and the corresponding bracket rotation, an analytical procedure was developed using the theory of thin plates. In order to account for the constraints provided by the flanges and stiffeners, a scale factor was provided using refined finite element analysis by considering the effects of cross-sectional dimensions and the load application point. Girder flange rotation, thus the overhang bracket rotation, due to web out-of-plane deformation can be calculated for a given steel I-girder bridge.

- The torsional behavior of exterior girders was investigated. Fundamentals of the theory and the analyses implementations with respect to various loading scenarios and boundary conditions were adopted from the literature. Then, equations were used to obtain the maximum bracket rotation. In the end, upper and lower bounds were provided for the estimation of overhang bracket rotations due to exterior girder warping.
The procedures provided in this chapter provides a conservative estimation for the total overhang bracket rotation and the corresponding variation in the deck profile for straight steel I-girder bridges. Estimated deck profile needs to be checked with specified deck finish tolerances stipulated in highway agency specifications. An example calculation procedure is provided in the Appendix. The next chapter presents the remedial actions that can be implemented when the estimated deck profile exceeds the specified tolerances.
5 MEANS AND METHODS FOR MAINTAINING DECK PROFILE DURING CONSTRUCTION

5.1 Overview

Chapter 4 presents calculation procedures for determining deck profile under girder deformations during deck placement. Since deck finish tolerances are specified for worst-case scenarios, the attention was given to estimating the maximum rotational deformations. Once the deck profile is obtained, a check is performed to determine if the specified deck tolerances are met. Owner agencies have different ways of reporting deck finish tolerances. For instance, KDOT (2016) requires a maximum exterior girder rotation of 1 degrees, whereas ODOT (2007a) directly limits the maximum deck loss as 0.5 inches.

When deck profile violates the specified deck finish tolerance, remedial actions need to be taken before deck placement. Alternatives can be grouped under two categories: design considerations and construction practices. In this chapter, various remedial alternatives and the consequent effects are discussed.

5.2 Design Considerations

When deck finish tolerances cannot be met, various alternatives can be employed by the design team. These alternatives include decreasing the deck overhang width, increasing the number of cross-frames, and increasing the number of transverse stiffeners. Changing the cross-section of exterior girders is not considered as a remedial alternative since it is not practical to redesign the girders considering each limit state and repeat all design steps.

Selecting an appropriate deck overhang width is an essential part of the bridge design since it directly affects the load demand for exterior girders. The preferred practice is to specify the overhang width in a way that the resulting interior and exterior girder dimensions are similar. NSBA (2014) states that specifying overhang widths within the range of $\frac{S}{4}$ to $\frac{S}{3}$, $S$ being girder spacing, fulfills this task. Similarly, FHWA (2012c) suggests selecting overhang width around 30% to 32% of girder spacing (i.e., $\sim\frac{S}{3}$) to have similar sections for interior and exterior girders.

Without violating this rule of thumb, overhang width can be decreased as a remedial action. Often, exterior girders are subjected to greater loads than interior girders during deck placement due to
additional construction dead and live loads. Reduced overhang width lowers the load acting on the exterior girder. As a result, exterior girder rotation due to girder differential deflection is decreased. Additionally, reducing the overhang width results in smaller lateral loads acting on the exterior girder web due to overhang brackets. As a result, web out-of-plane deformations and the corresponding top flange rotations are reduced. Lastly, the eccentricity of component and construction loads acting on overhang brackets is decreased by reducing the overhang width. Consequently, torsional moments acting on exterior girders and the corresponding rotation due to warping decreases.

Earlier AASHTO ASD and LFD Bridge Design Specifications required a maximum limit of 25 feet for cross-frame spacing (FHWA 2012c). This requirement is removed from current AASHTO LRFD (8th Ed.) specifications. Instead, the designer is allowed to perform rational analyses to determine the required cross-frame spacing. For a given girder length, increasing the number of cross-frames, or decreasing the cross-frame spacing, significantly affect the torsional behavior of exterior girders. Rotation of exterior girders due to warping is a function of unbraced length. Considering torsional restraint provided by cross-frames, the unbraced length becomes the spacing between consecutive cross-frames. Therefore, torsional rotation of exterior girders can be controlled by increasing the number of cross-frames. In addition to that, in the absence of hardened deck, cross-frames serve as lateral bracing and provides lateral stability to the partially erected structure.

AASHTO (2017a) provides limits for the spacing of transverse stiffeners. The Article C6.10.2.1.1 states that by applying Eq. (4-20), the transverse stiffener spacing up to $3D$ is permitted. On the other hand, the Article C6.10.9.3.3 states that spacing from the support to the first stiffener shall not exceed $1.5D$. As explained in Section 4.3, along with the flanges, transverse stiffeners enclose the exterior girder web and serve as support conditions. For a given girder, reducing the spacing between consecutive transverse stiffeners significantly decreases the web out-of-plane deformations and the corresponding top flange rotation under the same amount of lateral load. Additionally, transverse stiffeners increase the torsional stiffness of exterior girders, especially the warping resistance. Thus, the torsional rotation of exterior girders is decreased by increasing the number of stiffeners. Likewise, differential deflection of girders may be decreased due to increased major axis stiffness of exterior girders.
To maintain the deck profile within specified tolerance limits, the design team can consider the remedial alternatives above. When needed, several of them can be implemented simultaneously, however, the design team is responsible for evaluating the effects of each remedial action on overall structural behavior.

5.3 Construction Practices

Often, owners require contractors to submit deck pour sequence and procedures, and necessary calculations before deck placement. Contractors should evaluate whether the intended deck profile can be achieved with the proposed deck placement activities. When deck finish tolerances cannot be met with the proposed construction method, remedial actions need to be considered to achieve the intended deck profile. Alternatives include providing temporary bracings, use of appropriate bracket size, and pre-rotating the overhang brackets.

Installing temporary bracings between exterior girders and adjacent interior girders can effectively prevent the torsional rotation of exterior girders. Bracings can be implemented in various configurations. Configurations with a combination of a transverse and a diagonal member are the most efficient bracings for preventing the rotation (Ashiquzzaman et al. 2016). However, temporary bracings are not cost- and time-effective options.

Selecting the appropriate bracket size is essential for deck overhang construction. Locating the bracket bearing point close to bottom flange-web intersection eliminates web out-of-plane deformations and the corresponding top flange and bracket rotations. Further, for a given bracket width, the angle between the diagonal and vertical legs of the bracket decreases as the length of the vertical leg increases. Under the same amount of component and construction loads, the lateral load acting on the exterior girder web through overhang brackets gets smaller as the angle decreases.

Lastly, pre-rotating overhang brackets can be a remedial action. In Chapter 4, procedures are provided for determining the total rotation of overhang brackets during deck placement. Utilizing these, the required angle for pre-rotation can be calculated. However, procedures provided in Chapter 4 provide a conservative estimate. Therefore, the required pre-rotation angle calculated with the given procedures does not equally apply to all the brackets attached to an exterior girder. Calculating the required pre-rotation angle for each bracket and adjusting each bracket is not a
practical implementation. Instead, pre-rotating can be applied to brackets where deck finish tolerances cannot be satisfied along the bridge span.

5.4 Summary

When the deck profile violates the deck finish tolerance due to girder deformations, remedial actions need to be considered to achieve the intended deck profile as per project specifications. Remedial alternatives are grouped into two categories: design considerations and construction practices. Table 5-1 summarizes these alternatives and their effectiveness in controlling different rotational deformations.

<table>
<thead>
<tr>
<th>Remedial Alternatives for Achieving the Intended Deck Profile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Remedial Action</td>
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<tr>
<td>Design Considerations</td>
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<tr>
<td>Reducing deck overhang width</td>
</tr>
<tr>
<td>Increasing number of cross-frames</td>
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<td>Increasing number of transverse stiffeners</td>
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<tr>
<td>Construction Practices</td>
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<tr>
<td>Providing temporary bracings</td>
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<td>Selecting appropriate bracket size</td>
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<td>Pre-rotating overhang brackets</td>
</tr>
</tbody>
</table>

$\theta_d$ = rotational deformation of exterior girder due to differential deflection  
$\theta_{we}$ = rotational deformation of exterior girder top flange due to web out-of-plane deformation  
$\theta_w$ = rotational deformation of exterior girder due to warping
6 SUMMARY, CONCLUSION AND FUTURE RESEARCH

6.1 Summary and Conclusion

This thesis aims to develop analytical procedures for calculating deck profile and to provide guidance on selecting the most suitable means and methods for maintaining the intended deck profile in steel I-girder bridges during construction. Maintaining deck profile as per project specifications might be a challenging task. In literature, several issues related to unintended deck finish due to complications during deck placement are reported. Controlling girder deformations is the key for achieving the intended deck profile since deformations reflect on formworks and affect the final geometry of the deck. For non-complex highway bridges, cost- and time-effective procedures are needed for the evaluation of deck profile under girder deformations during the construction.

Determining deck profile under girder deformations or providing appropriate remedial actions for maintaining the deck profile requires the identification of cast-in-place deck construction practices, load transfer mechanisms, and steel I-girder behavior during deck placement. These details constitute the basis of analysis models employed for numerical evaluations.

Overhang bracket analysis is required for determining the loads acting on superstructure during deck placement. Specifically, bracket analysis results are used for differential girder deflection, girder web out-of-plane deformation, and exterior girder warping calculation. Considering overhang bracket geometry, boundary conditions, and loading, a statically determinate analysis was developed.

Differential girder deflection, exterior girder web out-of-plane deformation, and the exterior girder warping lead to complications in finishing the deck as per project specifications. These deformations cause exterior girder to rotate about the longitudinal axis of the bridge. When overhang brackets are rigidly connected to the exterior girders, the overhang bracket loads could result in unintended deck profile. The following procedures were presented to calculate the exterior girder top flange rotation due to differential girder deflection, exterior girder web out-of-plane deformation, and the exterior girder warping:
1) Exterior girder rotation and the corresponding bracket rotation due to differential girder deflection were analyzed using the procedures developed by Fisher (2006).

2) For evaluating bracket rotation due to web out-of-plane deformation, a procedure was developed using the theory of thin plates. Linear elastic finite element analyses were performed to validate the ability of plate theory on representing the behavior of exterior girder web under lateral loads and to evaluate the impact of constraints provided by flanges and transverse stiffeners on web out-of-plane deformations. Scale factors were derived to multiply the girder flange rotation calculated using the plate theory to account for the constraints provided by the flanges and stiffeners.

3) Bracket rotation due to exterior girder warping was analyzed using fundamental of torsion. Procedures were provided with respect to various boundary conditions. Upper and lower bounds were provided for the estimation of overhang bracket rotations.

4) The analysis procedures described in the above three steps can be used for estimating total overhang bracket rotation and the corresponding variation in deck profile for straight steel I-girder bridges.

Remedial actions presented for the cases where deck finish tolerances cannot be satisfied. These actions were grouped as design considerations and construction practices. Reducing deck overhang width, increasing number of cross-frames and increasing number of transverse stiffeners were specified as the design alternatives. Providing temporary bracings, utilizing appropriate bracket size, and pre-rotating brackets were proposed as the alternatives that can be implemented during construction.

Following conclusions are derived from this research:

- The SP procedure developed by Fisher (2006) provides an accurate estimation of girder differential deflection for simple-span steel I-girder bridges. The procedure accounts for transverse load distribution through cross-frames. The SGLSL procedure oversimplifies the behavior of continuous bridges.
• Solutions of the governing differential equation for fixed-fixed and pinned-pinned boundary conditions give upper and lower limits for rotation of exterior girders due to warping. These solutions do not consider the effects of transverse stiffeners, curvature, and skew. Thus, an engineering judgment is required to use the appropriate solution for the bridge under consideration.

• Theory of thin plates with small deformations can be used for calculating web out-of-plane deformation. The scale factors recommended through this study allows incorporating the impact of constraints provided by flanges and stiffeners on flange rotation due to web out-of-plane deformation.

• The scale factors were derived using the minimum dimensions specified in AASHTO (2017a) and NSBA (2016b) for girder cross-section and stiffeners. Top flange rotation due to web out-of-plane deformation decreases when flange width, flange thickness, stiffener width, or stiffener thickness increases. Hence, the scale factors can be used for the conservative estimate of flange rotation for any girder that is designed as per AASHTO LRFD and NSBA specifications, including curved and skewed bridges.

• Deck finish tolerances can be checked using the analytical procedures presented in this thesis before deck placement to identify the need for any remedial measures.
6.2 Future Research

This study aimed to develop simplified procedures for determining deck profile and providing means and methods for maintaining the deck profile in steel I-girder bridges during construction. The accuracy of simplified procedures provided herein needs to be verified using field measurement data. Additional procedures are needed for bridges having minor skew and/or curvature that structural behavior is not significantly affected and can be captured with simplified analysis tools. Further, similar procedures need to be developed for evaluating the deck finish complications in phased construction scenarios.
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APPENDIX

In the appendix, a complete procedure is provided for estimating the total rotation of overhang brackets and the corresponding variation in the deck profile considering the worst-case scenarios during deck placement. All the required parameters, i.e., bridge and girder geometry, falsework, formwork and equipment properties, were assumed considering the common design and construction practices. Calculations were provided in the form of Mathcad sheet. Some of the bridge characteristics are as follows:

- The bridge is simple-span straight steel I-girder bridge with the span length of 200 feet.
- It is composed of six identical girders. Girder spacing is uniform 10 feet.
- Girders are doubly symmetric and prismatic.
- Deck thickness is 9 inches. Overhang width is 3 ft.
- Overhang brackets are located at the middle of consecutive transverse stiffeners.
- Cross-frame spacing is 25 feet.
Input variables are shaded.

Bridge Geometry

\[ N_b := 6 \quad \text{number of girders} \]
\[ L := 200\text{-ft} \quad \text{span length} \]
\[ S := 10\text{-ft} \quad \text{girder spacing} \]
\[ t_s := 9\text{-in} \quad \text{deck thickness} \]
\[ b_{ov} := 36\text{-in} \quad \text{deck overhang width} \]
\[ L_b := 25\text{-ft} \quad \text{unbraced length} \]
\[ W := \left( N_b - 1 \right) \cdot S + 2 \cdot b_{ov} = 56\text{-ft} \quad \text{bridge with} \]

Girder Geometry

\[ b_{fc} := 28.25\text{-in} \quad \text{top flange width} \]
\[ b_{ft} := 28.25\text{-in} \quad \text{bottom flange width} \]
\[ t_{fc} := 1\text{-in} \quad \text{top flange thickness} \]
\[ t_{ft} := 1\text{-in} \quad \text{bottom flange thickness} \]
\[ D := 79.25\text{-in} \quad \text{web depth} \]
\[ t_w := 0.75\text{-in} \quad \text{web thickness} \]
\[ d_o := 100\text{-in} \quad \text{transverse stiffener spacing} \]
\[ t_p := 0.5\text{-in} \quad \text{transverse stiffener thickness} \]
\[ b_t := 7.25\text{-in} \quad \text{transverse stiffener width} \]
\[ I_x := 122079\text{-in}^4 \quad \text{major axis moment of inertia} \]
Material Properties

\[ E := 29000 \text{ ksi} \] modulus of elasticity

\[ G := 11200 \text{ ksi} \] shear modulus of elasticity

\[ v := 0.33 \] Poisson's ratio

Falsework, Formwork and Equipment Properties

\( b_{sm} := 3 \text{- ft} \) screeed machine wheel spacing

\( b_{sr} := 4 \text{- in} \) width of screeed rail platform

\( b_{wa} := 24 \text{- in} \) walkway width

\( n_w := 8 \) number of screeed machine wheels

\( s_b := 50 \text{- in} \) overhang bracket spacing

\( H_{br} := 15.85 \text{- in} \) bracket bearing point measured from top of the bottom flange

Component and Construction Loads

\( w_c := 150 \text{- pcf} \) weight of concrete (FDOT 2018a & INDOT 2013)

\( w_{SIP} := 15 \text{- psf} \) weight of SIP form (INDOT 2013 & MDOT 2018)

\( w_{of} := 15 \text{- psf} \) combined weight of overhang formwork and bracket (FDOT 2018a)

\( w_{wa} := 15 \text{- psf} \) weight of walkway (INDOT 2013)

\( w_{CLL} := 50 \text{- psf} \) construction live load (AASHTO 2017b & KDOT 2016)

\[ P_{tsm} := \begin{cases} 7 \text{- kip} & \text{if } 26 \text{- ft} \leq W < 32 \text{- ft} \\ 11 \text{- kip} & \text{if } 32 \text{- ft} \leq W < 56 \text{- ft} \\ 13 \text{- kip} & \text{if } 56 \text{- ft} \leq W < 80 \text{- ft} \\ 16 \text{- kip} & \text{if } 80 \text{- ft} \leq W < 120 \text{- ft} \end{cases} \] total weight of screeed machine (FDOT 2018a)

\[ f_{sm} := \begin{cases} 1 & \text{if } \frac{s_b}{b_{sm}} \leq 1 \\ 1.25 & \text{if } 1 < \frac{s_b}{b_{sm}} \leq 1.5 \\ 1.5 & \text{if } 1.5 < \frac{s_b}{b_{sm}} \leq 2.5 \\ 1.75 & \text{otherwise} \end{cases} \] Factors for the maximum screeed machine load per bracket (Meadow Burke 2017)
Overhang Bracket Analysis

\[ P_c := w_c \cdot t_s \cdot b_{ov} \cdot q_b = 1406.25 \text{ lbf} \] concrete load per bracket

\[ P_{CLL} := w_{CLL} \left( b_{ov} + b_{sr} + b_{wa} \right) \cdot q_b = 1111.11 \text{ lbf} \] construction live load per bracket

\[ P_{of} := w_{of} \left( b_{ov} + b_{sr} + b_{wa} \right) \cdot q_b = 333.33 \text{ lbf} \] combined overhang formwork and bracket load per bracket

\[ P_{wa} := w_{wa} \cdot b_{wa} \cdot q_b = 125 \text{ lbf} \] walkway load per bracket

\[ P_{sm} := \frac{P_{sm}}{u_w} = 2031.25 \text{ lbf} \] screed machine load per bracket

\[ R_{A_X} := \frac{\frac{P_c \cdot b_{ov}}{2} + \left( P_{CLL} + P_{of} \right) \left( \frac{b_{ov} + b_{sr} + b_{wa}}{2} \right) + P_{sm} \left( \frac{b_{sr}}{2} + b_{ov} \right) + P_{wa} \left( \frac{b_{wa}}{2} + b_{sr} + b_{ov} \right)}{b_{fc} \cdot D + t_{fc} - H_{br}} \]

\[ R_{B_X} := R_{A_X} = 1976.724 \text{ lbf} \] lateral load acting on the exterior girder web at bracket bearing point

Differential Deflection between Girders (Fisher 2006)

\[ b := \begin{cases} -0.08 & \text{if } \frac{S}{L} \leq 0.05 \\ -0.08 + 8 \left( \frac{S}{L} - 0.05 \right) & \text{if } 0.05 < \frac{S}{L} \leq 8.2 \end{cases} \] factor

\[ \alpha := \begin{cases} 0.0002 & \text{if } S \leq 8.2 \text{ ft} \\ 0.0002 + 0.0003 / 8 (S - 8.2) & \text{if } 8.2 \text{ ft} < S \leq 11.5 \text{ ft} \end{cases} \] factor

\[ \eta_{L} := \frac{w_c \cdot t_s \left( b_{ov} + \frac{S}{2} \right) + w_{CLL} \left( b_{ov} + b_{sr} + b_{wa} + \frac{S}{2} \right) + w_{of} \left( b_{ov} + b_{sr} + b_{wa} \right) + w_{wa} \cdot b_{wa} + w_{SIP} \cdot \frac{S}{2}}{L + \frac{P_{sm}}{2}} \cdot 100 \] exterior-to-interior girder load ratio (%)

\[ z := \left[ 10 \left( \frac{S}{L} - 0.04 \right) + 0.02 \right] \left( 2 - \frac{\eta_{L}}{50} \right) \] factor

\[ \Delta_{\text{dif}} := \left( 3 \left( \frac{S}{L} - 0.04 \right) \right) \left( 1 + z \right) = 0.031 \] inches. Positive indicates that interior girders deflected more than exterior girders.

\[ \theta_d := \arctan \left( \frac{-\Delta_{\text{dif}}}{S} \right) = -0.015 \text{ deg} \] rotational deformation of the exterior girder due to differential deflection rotation of the overhang bracket
Web Out-of-Plane Deformation

$$D_f := \frac{E t_w^3}{12(1 - \nu^2)} = 1144.127 \text{kip in}$$  flexural rigidity of the plate

$$w(x) := \frac{4 - R_B x}{\pi^4 D_f D_d^2} \sum_{m=1}^{10} \sum_{n=1}^{10} \left[ \frac{m^2 \pi (D - H_{br})}{D} \frac{n^2 \pi x}{D} \frac{\sin \left( \frac{m^2 \pi}{2} \right)}{D} \frac{\sin \left( \frac{n^2 \pi}{2} \right)}{D_d} \right]$$  exterior girder web out-of-plane deformation along the web depth

$$\alpha_p := \begin{cases} 0.3 & \text{if } 0.5 D \leq D - H_{br} < 0.6 D \\ 0.275 & \text{if } 0.6 D \leq D - H_{br} < 0.7 D \\ 0.25 & \text{if } 0.7 D \leq D - H_{br} < 0.8 D \\ 0.2 & \text{if } 0.8 D \leq D - H_{br} < 0.9 D \\ 0.125 & \text{if } 0.9 D \leq D - H_{br} < D \end{cases}$$  calibration factor to plate theory solution

$$\theta_{we}(x) := \alpha_p \frac{d}{dx} w(x)$$  rotation of the exterior girder top flange due to web out-of-plane deformation

$$\theta_{we}(0) = 0.025 \text{deg}$$  rotation of the overhang bracket

**Exterior Girder Warping**

$$J := \frac{b_{fc} t_{fc}^3 + b_{ft} t_{ft}^3 + D t_w^3}{3} = 29.978 \text{in}^4$$  torsional constant of the cross-section

$$C_w := \left( \frac{t_{fc} + t_{ft}}{2} \right) \left( \frac{D + b_{fc} + b_{ft}}{2} \right)^2 \left( \frac{b_{fc} \cdot b_{ft}^3}{b_{fc}^3 + b_{ft}^3} \right) = 6049704.988 \text{in}^6$$  warping constant of the cross-section

$$a := \frac{E C_w}{G J} = 722.864 \text{in}$$

$$T_u := \frac{P_c b_{ov} + (P_{CLL} + P_{olf}) (b_{ov} + b_{sr} + b_{wal})}{2} + p_{sm} \left( \frac{b_{sr} + b_{ov}}{2} \right) + p_{wa} \left( \frac{b_{wa} + b_{sr} + b_{ov}}{2} \right)$$  uniformly distributed torsional moment

$$= 517.407 \text{lb f-in}$$
\[
\theta_f := \frac{T_u L_b a}{2GJ} \left( \frac{1 + \cosh \left( \frac{L_b}{a} \right)}{\sinh \left( \frac{L_b}{2a} \right)} - 1 \right) + \frac{L_b}{4a} \sinh \left( \frac{L_b}{2a} \right) = 0.004 \text{ deg} \\
\text{rotational deformation of exterior girder due to warping when both ends are fixed}
\]

\[
\theta_p := \frac{T_u a^2}{GJ} \left( \frac{L_b^2}{8a^2} + \cosh \left( \frac{L_b}{2a} \right) - \tanh \left( \frac{L_b}{2a} \right) \sinh \left( \frac{L_b}{2a} \right) - 1 \right) = 0.018 \text{ deg} \\
\text{rotational deformation of exterior girder due to warping when both ends are pinned}
\]

\[
\theta_w := \theta_p \quad \text{worst-case scenario}
\]

**Tolerance Check**

\[
\theta_t := \theta_d + \theta_{we}(0) + \theta_w = 0.028 \text{ deg} \\
\text{total rotation of overhang bracket}
\]

\[
\Delta_{\text{deck}} = \left( \frac{b_{sr}}{2} + b_{ov} \right) \tan(\theta_t) = 0.019 \text{ -in} \\
\text{variation in deck profile}
\]

\[
\text{if}(\theta_t \leq 1 \text{ -deg}, "\text{Tolerance is satisfied.}", "\text{Remedial actions are needed.}"
\) = "\text{Tolerance is satisfied.}" \\
\text{Tolerance on bracket rotation (KDOT 2016)}
\]

\[
\text{if}(\Delta_{\text{deck}} \leq 0.5 \text{-in}, "\text{Tolerance is satisfied.}", "\text{Remedial actions are needed.}"
\) = "\text{Tolerance is satisfied.}" \\
\text{Tolerance on variation in deck profile (ODOT 2007a)}
\]