Investigation of the Use of Steel Intermediate Diaphragms and Temporary Bracing Alternatives for Prestressed Concrete Girder Bridges

by

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Abstract

Common in bridge construction today, diaphragms are placed transverse to the girders, connecting adjacent girders to provide stability and transmit loads. Diaphragms are defined as either end diaphragms – used at the ends of girders in simply supported spans and over the supports for continuous spans – or intermediate diaphragms – used at any number of points with the span. The focus of this research is on the use of intermediate diaphragms in simple-span prestressed concrete girder bridges, specifically those with I-beams and Bulb-tees. Intermediate diaphragms are used in precast concrete girder bridges for three primary reasons: 1) to prevent torsional girder rotations during girder erection and deck placement operations, 2) to increase the vertical load distribution between girders, and 3) to transfer and spread an impact load from an overheight vehicle to adjacent girders. Typical practice includes the design of end diaphragms, but there is significant variation in the practice of specifying or requiring intermediate diaphragms between state transportation agencies. Wide variations exist in the acceptance of steel alternates to traditional cast-in-place concrete. In addition to the lack of cohesion in material choice, there is also significant variation in the type and geometry of steel intermediate diaphragms, spacing within the span, and alignment relative to the girder. The importance of intermediate bracing in a span during construction is widely accepted as essential; however, its contribution to a

bridge in service, after the bridge deck has gained strength, is considered by some to be very minimal. As part of this research, a detailed survey of design practices by individual state bridge design agencies throughout the United States (U.S.) was conducted for all 50 states. This is the first effort to successfully profile the use of intermediate diaphragms in all 50 states. This thesis provides details on the usage of steel intermediate diaphragms by numerous states.

When this research began in January 2013, the Alabama Department of Transportation (ALDOT) was interested in re-evaluating their practice of requiring reinforced concrete intermediate diaphragms in precast girder bridges. Initially they wanted to look into steel intermediate diaphragm alternates based on the interest from contractors. Later, in August 2013, it was decided that ALDOT would no longer specify intermediate diaphragm details and the contractors would assume responsibility for designing an adequate bracing scheme for bridges during construction. The objective of this research was to investigate temporary bracing schemes used for prestressed girder bridges across the U.S. and provide recommendations to ensure stability and safety of girders during construction by temporary bracing.

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Chapter 1: Introduction

1.1 Background

Common in bridge construction today, diaphragms are placed transverse to the girders, connecting adjacent girders to provide stability and transmit loads. Diaphragms are defined as either end diaphragms – used at the ends of girders in simply supported spans and over the supports for continuous spans – or intermediate diaphragms – used at any number of points within the span. The focus of this research is on the use of intermediate diaphragms in simple-span prestressed concrete girder bridges, specifically those with I-beams and bulb-tees.

The construction of end diaphragms in precast girder bridges, bracing the ends of adjacent girders, is typical throughout the United States (U.S.), but the use of intermediate diaphragms within spans varies widely among state bridge design agencies. Prior to August 2013, the Alabama Department of Transportation (ALDOT) specified the use of cast-in-place reinforced concrete intermediate diaphragms in bridges with precast concrete girders. When this research was initiated in January 2013, the engineers at ALDOT were interested in the feasibility and performance of steel intermediate diaphragms following requests from contractors to use steel diaphragm alternates. However, within the time of this research, in August 2013, ALDOT made the decision to take intermediate diaphragms out of their specifications and place the responsibility of ensuring the stability of girders during erection onto the contractors. The benefits and concerns of intermediate diaphragms in precast girder bridges have been debated in both research and design. Through an extensive review of existing research and detailed survey of design practices throughout the U.S., both sides of the issue were studied. This thesis provides a brief background of existing research conclusions, as well as profile the use of intermediate diaphragms across the U.S.

Intermediate diaphragms have been proven to aid in vertical load distribution of service loads between adjacent girders by reducing the maximum deflection and bending moment for each individual girder. However, these effects are not currently accounted for in *AASHTO LRFD Bridge Design Specifications* (2012); for example, effects are not included in the calculation of load distribution factors. Green et al. (2004) concluded that intermediate diaphragms provided almost 19% reduction in girder deflections for straight bridges, 11% reduction for bridges skewed 15-30°, and 6% for bridges skewed 60°. Because the design method in *AASHTO LRFD Bridge Design Specifications* (2012) currently does not include these effects, the designs are more conservative, not accounting for the slight benefit of intermediate diaphragms to reduce deflections and bending moments.

When a bridge is struck by an overheight vehicle on the roadway beneath, the effect of intermediate diaphragms in distributing that load is well understood, but the consequences are debated. The effectiveness of intermediate diaphragms in distributing an impact load is dependent the location of impact. Intermediate diaphragms are only effective at transferring the lateral load if the impact occurs at the location of the intermediate diaphragm. If the impact occurs at the location of the diaphragm, the lateral load is transferred from the impact location at the exterior girder, through the diaphragm, to the adjacent girder, and so on. This distributes a potentially devastating force for the external girder through multiple girders. If the impact is less severe, this effect could help avoid the failure of the exterior girder; however, if the impact is severe, the distribution of large forces could result in compromising multiple girders, rather than concentrating damage to the exterior girder. In design, however, spacing of intermediate diaphragms is not a function of the location of the travel lanes and the location of an impact with respect to the location of a diaphragm is unpredictable. While the effect of an intermediate diaphragm in this case has the potential to be beneficial to the safety of the structure, it is a complex interaction between the location and level of the impact, girders, and intermediate diaphragms that is not easily agreed upon.

Across the U.S., the largest variation in the use of intermediate diaphragms is whether state bridge design agencies allow steel and/or concrete for intermediate diaphragms. Steel intermediate diaphragms were proposed as an alternate to cast-in-place concrete because they can reduce construction time and simplify the construction process, but to be efficient the connections must be simple and practical in the field. Based on different experiences in the field, states have unique reasons behind the type of intermediate diaphragms they allow. For example, the Kentucky Transportation Cabinet considered steel intermediate diaphragms in reaction to issues of concrete spalling at the girder-diaphragm interface when cast-in-place intermediate diaphragms were used (Griffin, 1997).

As steel intermediate diaphragms began to gain popularity as an alternative to concrete, much research was conducted to determine the adequacy of steel intermediate diaphragms as a substitute. Research efforts by Abendroth et al. (2004) and Chandolu (2005) concluded that reinforced concrete intermediate diaphragms provide better protection in the event of an impact from an overheight vehicle at the location of the diaphragm, as measured by the decrease in the maximum stress in the exterior girder. However, if the impact occurs at a location away from the diaphragm, the type and presence of intermediate diaphragms did little to mitigate the damage. Chandolu (2005) concluded that steel and concrete intermediate diaphragms provided equivalent stability during construction.

As states like Alabama re-evaluate the traditional practice of using cast-in-place reinforced concrete intermediate diaphragms and consider alternatives like steel intermediate diaphragms or temporary bracing schemes, it is extremely helpful to understand the practices of other state bridge design agencies across the U.S. as a starting point. Additionally, it is beneficial to understand the most recent research conclusions on the subject to understand the reasoning behind intermediate diaphragm design practices. This thesis will provide a detailed summary of related research efforts and the use of intermediate diaphragms across the U.S., with a comprehensive look at steel and temporary bracing alternatives.

1.2 Objectives

The objective of this research is to present the intermediate diaphragm alternatives to cast-in-place reinforced concrete that are used in practice throughout the U.S., including steel and temporary bracing designs. This study focuses on the use of intermediate diaphragms in simple-span prestressed concrete girder bridges, specifically those with I-beams and Bulb-tees. This research seeks to address the following points:

1. Review literature and previous research on the effectiveness of intermediate diaphragms during construction, including the effects they have on lateral stability.
2. Review literature and previous research on the effectiveness of intermediate diaphragms in service, including the effects they have on vertical and horizontal live load distribution.
3. Review literature and previous research on the comparison of the performance of steel and reinforced concrete intermediate diaphragms.
4. Profile the types of intermediate diaphragms specified by all 50 state transportation agencies across the U.S.
5. Compare the details for steel intermediate diaphragms among those states specifying the use of steel intermediate diaphragms in precast concrete girder bridges.
6. Compare the temporary bracing methods specified by state transportation agencies that do not require intermediate diaphragms.

1.3 Thesis Organization

Chapter 2 presents a literature review and recap of the numerous research efforts that has been conducted on the topic of intermediate diaphragms. This chapter is organized by topic, and then chronologically organized within that subsection. Chapter 2 includes a recap of research on vertical live load (traffic) distribution, lateral live load (overheight vehicle impact) distribution, the comparison of steel and concrete intermediate diaphragms, the use of intermediate diaphragms during construction for lateral stability, and the variations in practice of intermediate diaphragm design across the U.S. In addition, this chapter includes a summary of the current design codes relating to intermediate diaphragms. The conclusions on load distribution (vertical and horizontal) from intermediate diaphragms vary, but the importance of intermediate diaphragms or temporary bracing during construction is agreed upon as essential.

Chapter 3 discusses the survey that was conducted for the state bridge design agencies throughout the U.S. and the data collected. This chapter provides a general profile of the practices of using intermediate diaphragms throughout the U.S., including all 50 states. The results in the chapter are focused on the variations in the type (steel, concrete, or temporary) of intermediate diaphragms specified in each state. The objective of this research was to look into intermediate diaphragm practices that do not use traditional cast-in-place reinforced concrete intermediate diaphragms. Chapter 4 and 5 focus on the steel and temporary intermediate diaphragms alternatives used throughout the U.S.

Chapter 4 describes the details of steel intermediate diaphragm practices throughout the U.S. This includes chapter includes discussion of the various intermediate diaphragm configurations in use, the relationship between intermediate diaphragm type and beam type in each state, alignment of diaphragms in skewed bridges, and intermediate diaphragm spacing requirements in each state.

Chapter 5 discusses the temporary bracing options specified by those states not permitting permanent intermediate diaphragms. Each of the four states in this category – Alabama, Florida, Kansas, and Texas – has unique standards for temporary bracing of girders during erection and deck construction operations. This chapter discusses the requirements and standard details used by each state, individually.

Chapter 6 includes a summary, conclusions, and recommendations.

Appendix A includes a detailed example of the Mathcad program used by the Florida Department of Transportation to determine beam stability requirements for the design of temporary bracing members and connections.

Chapter 2: Literature Review

2.1 Vertical Live Load (Traffic) Distribution

In September 1970, Sengupta and Breen (1973) began an extensive study into the effects of reinforced concrete diaphragms in prestressed concrete girder and slab bridges. Their research included both experimental lab tests and computer analysis. The study focused on service-level conditions, only, and tested both static and dynamic load effects. Sengupta and Breen concluded that the *only* significant benefit of intermediate diaphragms is their assistance in evenly distributing loads between adjacent girders from the traffic on the deck. The addition of intermediate diaphragms slightly reduced the maximum bending moments in the girders; the maximum reduction ranged from 5-8% for standard AASHTO truck loads. They suggested that it would be more economical to increase the capacity of the girders than to add intermediate diaphragms and rely on them to reduce the moments in the girders by load distribution. Such design changes, however, were not necessary because the design process in the 1969 AASHTO specifications neglected the effects of intermediate diaphragms and was already conservative.

Barr et al. (2001) studied the live load distribution factors used in bridge design, validating their accuracy and examining the effect intermediate diaphragms would have on those factors, if considered. *The Load and Resistance Factor (LRFD) Bridge*

Specifications (AASHTO 1994) were adopted by the American Association of State Highway and Transportation Officials (AASHTO) in 1994 as an alternative to the *AASHTO Standard Specifications* traditionally used.

The *AASHTO Standard Specifications* included live load distribution factors beginning in 1931, and were updated as new research became available. Live load distribution factors are used to account for transverse effects of wheel loads on girders. The design live load moment for each girder is determined by finding the maximum moment caused by a truck or lane of traffic, and factoring that by the live load distribution factor. The LRFD code introduced new expressions for live load distribution factors, which addressed the effects of girder spacing, girder stiffness, span length, skew, and slab stiffness, some of which had been previously neglected. The effects of intermediate diaphragms were not included in the expressions for live load distribution.

Barr et al. (2001) validated the accuracy of the finite element model techniques used to develop the AASHTO LRFD (1994) expressions for live load distribution factors. To study their contribution to load distribution, reinforced concrete intermediate diaphragms were included in experimental testing and finite element analysis. The influence of the intermediate diaphragms was found to be minimal compared to the effects of the variables included in the expression, such as end diaphragms and skew. It was further concluded that the expressions in AASHTO LRFD (1994) were significantly conservative, even without the inclusion of intermediate diaphragm effects. By using more precise distribution factors from a finite element model instead of the expressions in AASHTO LRFD, a bridge could have the additional capacity for approximately 39% higher live load.

Eamon and Nowak (2002) investigated the effects of edge-stiffening elements and intermediate diaphragms on ultimate capacity and load distribution using finite element modeling. Edge stiffening elements – barrier railings and sidewalks – and intermediate diaphragms were not considered in the load distribution factor calculations in AASHTO LRFD (1998). Their study sought to quantify and understand the discrepancies in the design code's prediction of behavior compared to a detailed finite element analysis. Considering secondary elements – barriers, sidewalks, and diaphragms – in the analysis led to the reduction of the live load distribution factor between 10-40% in the elastic range, 5-20% in the inelastic range, and an increase in ultimate capacity between 110-220%. The ranges of effectiveness are due to the influence that bridge geometry and element stiffness have on the structure's behavior. Further, element deterioration and the behavior of connections could affect these results, but were not considered.

Green et al. (2004) investigated the benefits of intermediate diaphragms to enhance the performance of precast bridge girders. Using finite element modeling alone, the analysis included reinforced concrete intermediate diaphragms combined with bridge skew and temperature effects. Green et al. (2004) concluded that intermediate diaphragms provided an overall benefit to the structure by stiffening precast bridge girders and reducing maximum girder deflections; however, the amount of reduction varied based on skew and temperature effects. In straight (non-skewed) bridges, deflections were reduced by about 19%; however, there was less reduction in deflections for skewed bridges, with reductions of approximately 11% and 6% possible for 15-30° skew bridges and 60° skew bridges respectively. Combining the effects of temperature changes with intermediate diaphragms also reduced deflections by 3-14%.

Cai et al. (2009) quantified the effects of reinforced concrete intermediate diaphragms on the live load distribution factors published in AASHTO LRFD Bridge Specifications (2002). The goal of this study was to generate an approach that could be used by engineers, in conjunction with the live load distribution factor method in the AASHTO LRFD code, for more accurate distribution expressions. The results were quantified in terms of the influence in load distribution due to intermediate diaphragms, denoted by R_d. Equation 2-1, below describes R_d as a ratio of the load distribution factors with and without intermediate diaphragms considered.

$$R_d = \frac{(LDF_{ND} - LDF_{WD})}{LDF_{ND}} \times 100 \qquad (2\text{-}1)$$

where

R_d = percentage influence in load distribution due to intermediate diaphragms

LDF_{ND} = load distribution factor without considering intermediate diaphragms that can be obtained from AASHTO LRFD specifications

LDF_{WD} = load distribution factor with intermediate diaphragms considered

The influence factor, R_d, varies based on the skew of the bridge and the location of intermediate diaphragms. The equations for R_d that were established by Cai et al.

(2009) vary based on the number of intermediate diaphragms and the position of the girder being considered, and are summarized in Table 2.1, below.

Table 2.1. Expressions of R_d (Cai et al. 2009)

No. of Intermediate Diaphragms in Span	Girder Position	Equation of R_d
1	Interior	$R_d = [(0.132L + 4.85) + C]S_t$
	Exterior	$R_d = (0.132L - 15.81 - C)P_L S_k$
2	Interior	$R_d = [(-0.112L + 25.81)C]S_t S_k$
	Exterior	$R_d = (-19.05 + 0.147L - C)P_L S_k$
Note: R_d = influence in load distribution due to diaphragm, C = constant, L = length of the girder, S_t = stiffness influence factor, P_L = correction factor for taking into account position of lateral loading system, S_k = skew influence factor		

As seen in Table 2.1, the equations for R_d include numerous additional variables, which are defined in Table 2.2 and Table 2.3. To account for the combination of effects of girder size/type, the number of intermediate diaphragms in the span, and the location of the girder, a constant (C) was developed, and is presented in Table 2.2. The values of C were not found for all cases, the cases not included in the study are denoted with “n.c.” and would require additional analysis to determine.

The remaining variables are detailed in Table 2.3 – the correction for skew angle (S_k), the influence factor for the stiffness of the intermediate diaphragm for interior girders (S_t), and the wheel loading position factor for exterior girders (P_L). The skew angle correction, S_k, is defined based on the angle between the longitudinal line along the bridge span and a line normal to the abutment, in degrees, and varies based on the number of intermediate diaphragms and the location of the girder. An intermediate diaphragm stiffness influence factor, S_t, is needed for interior girders to account for connections at the girder-diaphragm interface that will result in a percentage of the diaphragm being able to contribute to load distribution. The percentage of intermediate diaphragm stiffness that is effective for load distribution, S_r, that is used in the calculation of S_t is either assumed or based on analysis/testing. The full intermediate diaphragm stiffness is used for exterior girders to yield conservative results. The final factor, P_L, is used to account for variations in the width of the barrier and the cantilever portion. AASHTO LRFD (2002) requires that for analyzing exterior girders, the exterior wheel line be at least 24 inches from the interior edge of the barrier; the P_L factor is based on the distance from the center of the exterior girder to the exterior wheel line.

Table 2.2. Values of Constant C in Expressions for R_d (Cai et al. 2009)

Girder Type	**Number of Intermediate Diaphragms Per Bay in Single Span**			
	Interior Girder		**Exterior Girder**	
	1	**2**	**1**	**2**
AASHTO Type II	0	n.c.	0	n.c.
AASHTO Type III	2	n.c.	3	n.c.
AASHTO Type IV	3.5	1	5	0
Bulb Tee	n.c.	1.98	n.c.	4
Note: n.c. = not considered				

Table 2.3. Values of S_k , S_t , and P_L (Cai et al. 2009)

No. IDs	Interior Girder		Exterior Girder	
	S_k	**S_t**	**S_k**	**P_L**
1	$1 - 0.015\theta$ ($\theta \leq 30°$) $0.775 - 0.0075\theta$ ($\theta > 30°$)	$0.0264 S_r^{0.8062}$	$1 - 0.01\theta$ ($\theta \leq 30°$) 0.7 ($\theta > 30°$)	$0.45 + 0.55d$ ($0 \leq d < 3$ ft)
2	$1 - 0.0167\theta$ ($\theta \leq 30°$) $0.725 - 0.0075\theta$ ($\theta > 30°$)	$0.0873 S_r^{0.5358}$ (Type IV) $0.3024 S_r^{0.2641}$ (BT)	$1 - 0.013\theta$ ($\theta \leq 30°$) 0.6 ($\theta > 30°$)	$0.45 + 0.55d$ ($0 \leq d < 3$ ft)

Note: R_d = influence in load distribution due to diaphragm, S_t = stiffness influence factor, P_L = correction factor for taking into account position of lateral loading system, S_k = skew influence factor, d = distance between center of exterior girder to wheel line closest to edge, S_r = ratio of possible diaphragm stiffness contributing to load distribution and absolute diaphragm stiffness x 100 [either assumed or available from analysis or test], θ = skew angle (degrees)

For a specific girder, the results from Tables 2.1 -2.3 are combined to determine the influence factor describing the influence of intermediate diaphragms, R_d. Found by rearranging Equation 2-1, Equation 2-2 can used to calculate a new live load distribution factor that includes the effect of intermediate diaphragms.

$$LDF_{WD} = \left(1 - \frac{R_d}{100}\right) LDF_{ND} \tag{2-2}$$

The results of the quantification of effects of intermediate diaphragms by Cai et al. (2009) provides an approach to calculate live load distribution factors that account for the effects of intermediate diaphragms .

2.2 Lateral Load (Overheight Vehicle Impact) Distribution

Sengupta and Breen (1973) raised concerns about the potentially *negative* effects intermediate diaphragms would have in the event of an impact from an overheight vehicle or overheight load traveling on the road beneath. Based on their experimental testing of microconcrete model bridges with reinforced concrete diaphragms, they concluded that intermediate diaphragms would make interior girders more vulnerable to damage because the diaphragms transferred lateral impact loads into the interior girders, spreading the potentially damaging effects, rather than damage being isolated to the exterior girder that was struck. The effects of intermediate diaphragms in the case of an impact from an overheight vehicle would be specifically addressed by future research efforts to validate this concern. Their final recommendation was that intermediate diaphragms should *not* be provided for simply supported, prestressed concrete girder and slab bridges. This conclusion was not universally accepted, and became a basis for many future research efforts.

Abendroth et al. (1991) and Abendroth (1995) focused on the effects of intermediate diaphragms when a bridge is struck by an overheight vehicle or by an overheight load being transported, citing the disputed conclusion by Sengupta and Breen (1973). Their research efforts combined the testing of full scale models and finite element modeling. Full scale models were tested and the effects were measured by displacements and strain gages at multiple locations along the span. Four types of intermediate diaphragms were tested – reinforced concrete, shallow steel channel, deep steel channel, and a steel X-brace plus strut combination. To simulate the effects of an impact, lateral loads were applied at the bottom flange of the girder. Finally, finite element models were built and analyzed and the results of the analysis were compared with the experimental results. The largest source of variation in the two sets of results was likely bolt slippage at the connections of the steel channel intermediate diaphragms. Based on their research, Abendroth et al. (1995) concluded that vertical load distribution is mostly independent of the type and location of intermediate diaphragms, but horizontal load distribution is *highly* dependent on the type and location of intermediate diaphragm. The influence on intermediate diaphragm type and location would be further addressed in future projects.

Abendroth et al. (2004) began an extensive study in 1999 to analytically study the effects of different types of intermediate diaphragms in reducing or exacerbating damage to the girders of a prestressed concrete girder bridge that has been struck by an overheight vehicle or load. At a time when steel intermediate diaphragms were gaining popularity, this study examined whether or not steel intermediate diaphragms would provide equal protection compared to reinforced concrete intermediate diaphragms. At the time, Abendroth et al. (2004) noted that an average of 200 prestressed girder bridges were damaged each year; about 80% of them were struck by an overheight vehicle or vehicle load. Understanding the effects of intermediate diaphragms in these situations was crucial, especially with some engineers suspecting they were actually increasing the damage to the bridge. While prior research efforts estimated their modeling techniques of the impact load, Abendroth et al. (2004) based their modeling techniques on a thorough review of various crash-test publications.

Finite element models were built to represent two different bridges and three different intermediate diaphragm types. Two prototype prestressed girder bridges, designed by the Iowa Department of Transportation (Iowa DOT), were selected to represent typical designs in Iowa – one bridge skewed at approximately 20° and one bridge with negligible skew. Both bridges had similar geometry; the skew was the major difference in the designs. Three different intermediate diaphragms were considered in the theoretical study – a typical reinforced concrete intermediate diaphragm, a steel X-brace with a bottom horizontal strut, and a steel K-brace with the horizontal strut oriented at the bottom. Impact loads simulating that of an overheight vehicle or load striking the exterior girder were applied as a static load that had been magnified by a dynamic load factor. The maximum impact load was chosen as that which would induce the maximum tensile strain in the impacted girder without excessively exceeding the modulus of rupture of concrete for that girder. To compare the results from the different models, they chose to look at the induced tensile strains in the girder to measure where the damage to the girder would be worst. In their conclusions, Abendroth et al. (2004) compared the ability of the three different types of intermediate diaphragms to provide protection to the structure by reducing damage to the impacted girder. In all cases – for all locations of impact and all diaphragm types – the presence of intermediate diaphragms was beneficial to reducing the damage on the exterior girder by transferring some of the load into the interior girders and diaphragms. The influence of the *type* of diaphragm present was only significant when the impact occurred at the location of the diaphragm. While the reinforced concrete intermediate diaphragm was slightly more effective than the steel diaphragms considered, that increased benefit would only exist in the event that the impact occurred at the location of the diaphragm. However, because intermediate diaphragm locations are not based on the location of the travel lanes below and rather based on the bridge's geometry, the probability of an overheight vehicle striking the exact location of a diaphragm is unlikely. Therefore, the presence of any of the intermediate diaphragms considered would provide essentially the same degree of protection.

2.3 Comparisons of Steel and Concrete Intermediate Diaphragms

In addition to their study of intermediate diaphragms in the event of an overheight vehicle impact, Abendroth et al. (2004) addressed the trend towards allowing steel intermediate diaphragms as an alternative to reinforced concrete diaphragms. The desire for steel diaphragms was being driven by bridge contractors who wanted to use steel diaphragms because they believed they would reduce construction time and simplify the construction process. On the other hand, bridge engineers with the Iowa Department of Transportation were concerned that steel intermediate diaphragms would not provide adequate impact protection, compared to the larger mass, stiffness, and damping characteristics of a reinforced concrete intermediate diaphragm. Design for a steel intermediate diaphragm option for Iowa was developed in the early 1990s, but was not well-received by contractors due to the complexity of the connection between the diaphragm and the beams. Abendroth et al. studied both steel and reinforced concrete intermediate diaphragms and their ability to protect an impacted girder by reducing damage to that girder. The goal of this analytical study was to ensure that steel intermediate diaphragms that were becoming more widely used would provide adequate protection. Three types of intermediate diaphragms were considered – reinforced concrete, steel X-brace with a horizontal strut, and steel K-brace with a horizontal strut. Two prototype bridges were used, both with similar geometry, but one was square and the other was skewed approximately 20°. After evaluating the effects of impact loads at multiple locations on the exterior girders, the following conclusions were made about the effectiveness of the different types of intermediate diaphragms.

- When the impact occurred at the location of the intermediate diaphragm, the reinforced concrete intermediate diaphragm was more effective at limiting the damage of the exterior girder than either steel option.
- Comparing the effectiveness of steel intermediate diaphragms to each other when a bridge is struck at the location of the diaphragm, the K-brace with a horizontal strut was slightly more effective at mitigating damage to the exterior girder than the X-brace with a horizontal strut. However, the difference was not enough to recommend the use of one over the other.
- The *type* of intermediate diaphragm was only significant when the impact was coincident with the location of the intermediate diaphragm. Even if the impact is just a short distance off, the three types of diaphragms provided a positive effect, but were all similar in their ability to prevent damage.

The location of intermediate diaphragms within a span is a function of the geometry of the bridge, and not based on the location of travel lanes below. The location of an impact is more likely to be away from the location of the intermediate diaphragm; therefore any of the three intermediate diaphragms would provide essentially the same impact protection to the girder.

Chandolu (2005) studied the differences between concrete and steel intermediate diaphragms to select a steel configuration that would be an effective replacement for a reinforced concrete diaphragm. Through finite element modeling, it was determined that the primary function of intermediate diaphragms is the transfer of axial forces through the diaphragms to and from the adjacent girders. Therefore, finding steel cross sections with comparable stiffness to a reinforced concrete diaphragm was the basis for selecting trial steel configurations. The following method was used by Chandolu (2005) to define the required stiffness and required cross sectional area for the steel intermediate diaphragm.

1. Cross section dimensions of reinforced concrete intermediate diaphragm to be replaced are equal to the height of the girder web multiplied by a typical 8" diaphragm width.
2. The axial stiffness of the reinforced concrete diaphragm to be replaced is assumed to be 40% of its absolute stiffness, to account for cracking at the girder-diaphragm interface.
3. Based on (1) and (2), the required cross section for the steel intermediate diaphragm is determined by Equation 2-3.

$$A_{steel} = \frac{0.40 \times A_{conc}}{n} \tag{2-3}$$

where

A_{conc} = cross sectional area of the concrete intermediate diaphragm

n = modular ratio of steel to concrete $(E_{steel}/E_{concrete})$

This methodology was used to find two different steel diaphragm configurations – one for AASHTO girders Type II-IV, and one for Bulb Tee girders.

Considering the small web depths for AASHTO Type II-IV girders, Chandolu (2005) determined a steel channel section was most appropriate; a channel could be easily connected to the girder and most of the height of the girder web would be connected to the diaphragm. For uniformity, a channel was chosen that would fit the web of smallest girder (Type II, 15” web) but have the stiffness required for the largest girder (Type IV, 23” web); Chandolu choose to use a C15x33.9.

For Bulb Tee girders, Chandolu (2005) determined that a single channel was not possible because single steel sections were not large enough; the maximum depth of a steel channel is 18”, which is very small compared the tall webs associated with Bulb Tee girders and could result in inadequate lateral stability. Based on Equation 2-3, a steel X-brace with a bottom strut configuration, built with all MC8x20 members, was chosen.

Intermediate diaphragms are perhaps most significant when transferring lateral loads during construction, therefore, the two different steel intermediate diaphragms that had been chosen were checked by finite element analysis based on constructionstage loading. Construction loads were approximated to include the dead load due to wet concrete (112.5 psf, based on a 9” thick deck), assumed dead load of formwork (assumed to be 4 psf), and construction loads due to equipment (assumed to be 50 psf). The estimated loads, rounded up to an approximate 170 psf, were applied to the finite element as illustrated in Figure 2.1 to generate maximum forces in the bracing.

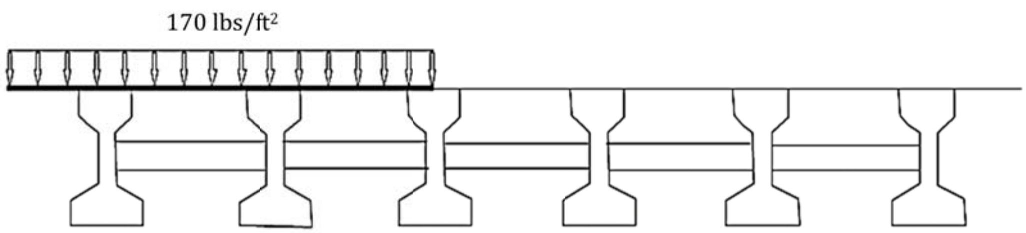

Figure 2.1. Construction loading applied to generate maximum forces in bracing (Chandolu 2005)

To determine if the suggested bracing was adequate, Chandolu checked the relative stability of the diaphragms and checked whether the load carried by the diaphragms was within the load carrying capacity of the steel members. To check stability, the stresses resulting from the finite element analysis were compared at a) the inner face of the web at the location of the diaphragm, and b) a location away from a diaphragm; the stress values were compared between the steel and concrete intermediate diaphragms. The stress values obtained from the finite element analysis were nearly the same for the concrete and steel diaphragms. Chandolu concluded that both steel intermediate diaphragm configurations provided adequate stability. Next, the diaphragms were checked for their load carrying capacity. The maximum forces in the bracing were determined by looking at various loading conditions and determining the worst case:

1. Uniformly distributed construction load of 170 psf distributed as illustrated in Figure 2.1.
2. Concentrated construction equipment load of 50 kips applied on the edge of the interior girder.
3. Different live load configurations used to determine distribution factors according to AASHTO LRFD.

Chandolu concluded that both of the steel intermediate diaphragms proposed had load carrying capacities significantly larger than the loads induced in the bracing. Therefore, in terms of stability and load carrying capacity, the steel intermediate diaphragms proposed by Chandolu were adequate alternatives to traditional reinforced concrete intermediate diaphragms.

As a final component of his research, Chandolu addressed the effectiveness of the different types of intermediate diaphragms in the event of an impact of an overheight vehicle or load. He quantified the reduction in stresses in the impacted girder using finite element analysis with the impact load applied as a static load. His results are summarized below in Table 2.4.

Table 2.4. Principle Stresses (ksi) Due to Impact Load (Chandolu 2005)

Bridge "S9L90" – AASHTO Type III Girders, Diaphragms at Midspan			
Location of Impact	Reinforced Concrete	Steel Channel	No ID
Diaphragm Location	0.45	0.8	1.4
Quarter Span	0.3	0.23	0.25
Bridge "S9L130" – BT Girders, Diaphragms at Quarter Span			
Location of Impact	Reinforced Concrete	Steel X-Brace + Strut	No ID
Diaphragm Location	0.5	0.9	3.5
Midspan	1	1.1	0.95

As the data in Table 2.4 show, the presence of an intermediate diaphragm is significantly helpful in reducing stress in the impacted girder only when the impact occurs at the location of the intermediate diaphragm. When the impact and location of the diaphragm are coincident, reinforced concrete intermediate diaphragms reduced stress by 68% (Bridge S9L90) and 86% (Bridge S9L130), compared to the case without intermediate diaphragms. The steel intermediate diaphragms provided reductions of 43% (Channel) and 74% (X-Brace + Strut). While there is a slight variation in the effectiveness of steel and concrete diaphragms when the impact occurs at the location of the diaphragm, there is a negligible effect of any diaphragm presence when the impact occurs away from the diaphragm location. As also addressed by Abendroth et al. (2004), the location of diaphragms is not typically determined by possible impact locations or the locations of travel lanes below, therefore it is more likely that an impact would occur away from the diaphragm location. As a result, the variation in performance of steel/concrete diaphragms when a bridge is struck by an overheight vehicle or load is not recommended to be used as a significant factor in the decision to use concrete or steel diaphragms in a structure.

2.4 Use of Intermediate Diaphragms During Construction for Lateral Stability

The topic of lateral stability, especially the risk of rollover, is extremely important to discuss when considering the importance of intermediate diaphragms for girder stability during construction. A stability failure of unbraced, or inadequately braced, girders after erection is both dangerous and costly. A recent stability failure of this nature occurred during construction of the Red Mountain Freeway in Mesa, Arizona.

In the midst of construction of the Red Mountain Freeway, on the morning of August 9, 2007, nine of eleven prestressed concrete, AASHTO Type V bridge girders in westbound Span 5 (5W) collapsed. Oesterle et al. (2007) investigated the failure on behalf of the Arizona Department of Transportation (ADOT). According to the report by Oesterle et al. (2007), at the time of the failure, there was not any lateral bracing in place. The girders were reportedly erected on the piers and placed onto elastomeric bearings on July 19, 2007 (3 weeks prior). All girder erection operations were completed on July 25, 2007, and no further work had been performed on the girders since, although construction activities were still in-progress on other areas of the site. Led by Oesterle et al. (2007), the CTLGroup made observations of the construction site, reviewed plans and code requirements, tested samples from the collapsed girders, performed structural analyses for lateral stability checks, then developed conclusions and recommendations based on their findings.

Figure 2.2 through Figure 2.5 show the damages post-collapse documented by Oesterle et al. (2007). It is important to note a few things about the images:

1. The collapsed girders are resting in a position (on their side) that denotes that they rolled over during the collapse. This type of lateral instability failure is referred to as “rollover”.
2. Only eight girders are visible in Figure 2.3 and Figure 2.4; the ninth girder was completely submerged in the waterway.
3. All of the fallen girders are critically damaged; this damage occurred during the collapse.

These images show how damaging a lateral instability failure of unbraced girders can be. When girders are in this condition, they must be completely replaced, which is very costly. It is also important to check the stability of adjacent girders that are still standing, but may be unstable from the event of the collapse.

Figure 2.2. Red Mountain Freeway Collapse (Oesterle et al. 2007)

Figure 2.3. Red Mountain Freeway Collapse (Oesterle et al. 2007)

Figure 2.4. Red Mountain Freeway Collapse (Oesterle et al. 2007)

Figure 2.5. Red Mountain Freeway Collapse (Oesterle et al. 2007)

To understand the collapse, it is crucial to understand the conditions of the girders prior to the collapse. Most important for this failure, Oesterle et al. (2007) noted that the “bracing conditions” of the girders were not adequate. Figure 2.6 and Figure 2.7 show the conditions of Span 5W prior to collapse, and Figure 2.8 shows the condition of walkways that would have been present in Span 5W at the time of collapse.

Figure 2.6. Girders Prior to Collapse (Oesterle et al. 2007)

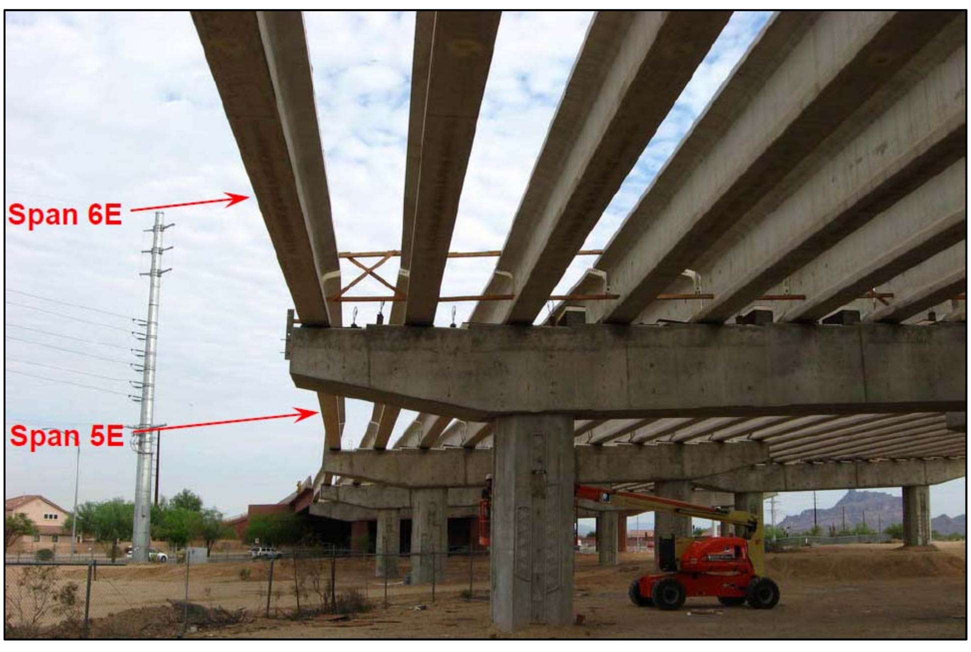

Figure 2.7. Girders Prior to Collapse (Oesterle et al. 2007)

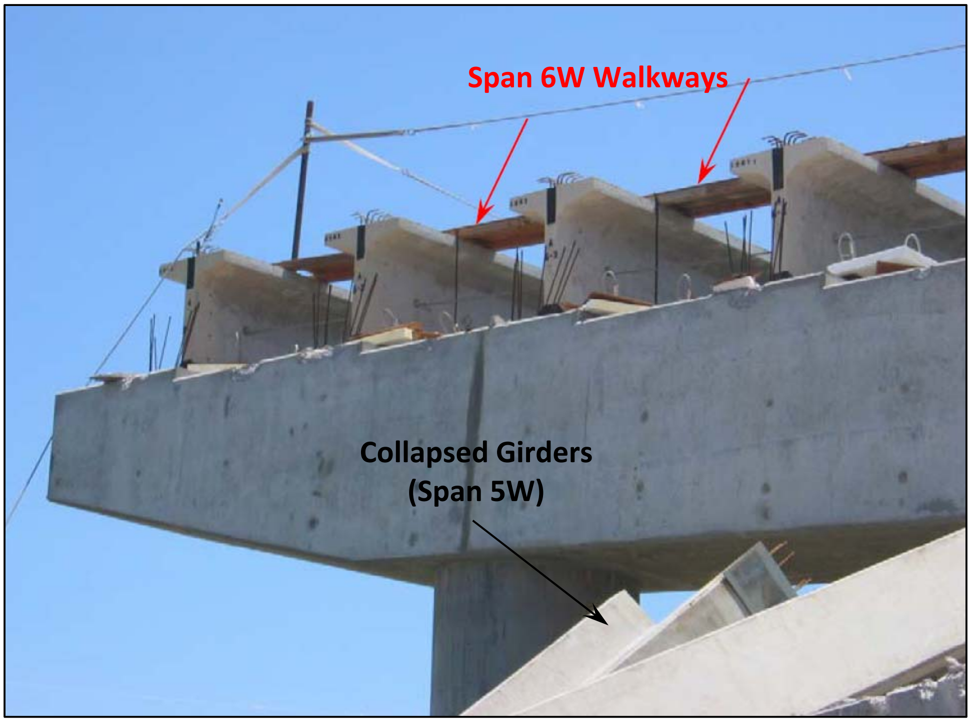

Figure 2.8. Span 6W Walkways after Collapse in Span 5W (Oesterle et al. 2007)

Figure 2.6 shows the timber horizontal struts between the webs of some of the girders in Span 5W (the span of failure). No timber struts are visible in the adjacent spans - Spans 5E, 4W, and 4E. Figure 2.7 shows the bracing conditions (timber horizontal struts and X-bracing) installed in the span adjacent to Span 5E – Span 6E. Figure 2.8 shows the wooden walkway present in the adjacent span after the collapse, a similar walkway spanned between the top flanges of adjacent girder in Span 5W at the time of the collapse. The bracing conditions visible in these images are grossly inadequate for lateral stability.

After a thorough analysis, Oesterle et al. (2007) concluded that cause of failure was the lateral instability of a single girder that led to the progressive collapse of the adjacent girders due to insufficient lateral bracing. The following details provided by Oesterle et al. (2007) supported that conclusion:

- The bracing conditions present in Span 5W at the time of collapse were inadequate to prevent overturning or sliding movements.
- The lateral instability failure of Girder A5-9 (labeled in Figure 2.6) and the resulting rollover and/or sliding failure caused the progressive collapse of the eight adjacent girders in a “domino effect”.
- The cause of lateral instability of Girder A5-9 was likely a critical combination of effects from: bearing eccentricity, initial sweep, thermal sweep, creep sweep, and slopes at supports.
- Wind loads on the day of the collapse would have had a minor effect on the girder, but could have compounded the instability and brought the girder to its failure. However, high wind loads were recorded on July 19, 2007 (the day the girders were erected in Span 5W) and could have caused eccentricity at the bearings for the girders shortly after erection.

The instability failure that resulted at the Red Mountain Freeway could have been prevented, and it is important to see this failure as a learning opportunity so that failures of this magnitude do not occur in the future. There must be regulations to ensure that girders are erected and placed on the bearing pads properly; the in-place position of girders on the bearings must be checked so that they are centered. In addition, immediately after girders are erected and positioned correctly, they must be braced for lateral stability at least at the girder ends for all adjacent girders.

In the late 1980s, Robert Mast began research on the *Lateral Stability of Long Prestressed Concrete Beams* that was broken up into two publications in the PCI Journal – Part 1 (1989) and Part 2 (1993). Part 1 focused on the stability of girders hanging from lifting loops; Part 2 focused on the stability of beams on elastic supports, such as bearing pads. Part 2 (Mast 1993) is heavily cited by recent research and still used today in calculations of lateral stability. Mast (1993) found that the failure of a prestressed girder with typical dimensions by "rollover" would occur before any lateral-torsional buckling. Further, he found that the "rollover" risk for a girder on elastic supports is a function of the support – specifically its roll stiffness – rather than a function of the beam. Mast (1993) concluded that when prestressed beams are set on elastomeric bearing pads, it is crucial that the beam's weight be concentric on the bearing pad because any amount of eccentricity, even if temporary, increases the potential for rollover. In addition, Mast (1993) suggested that the ends of beams should be braced to prevent rollover under high wind load conditions.

Expanding on the research by Mast (1993), the Florida Department of Transportation (FDOT) sponsored in-depth research at the University of Florida on the lateral stability and bracing on long-span Florida bulb-tee girders. The reports by Consolazio and Hamilton (2007) and Consolazio and Gurley (2013) were significant in the development of the *Beam Stability* program by FDOT (2013), used to determine adequate temporary bracing for prestressed girders after erection. Consolazio and Hamilton (2007) studied the interactions and combined effects of several parameters on girder stability and buckling capacity: cross sectional properties, span length, bracing stiffness, sweep, skew, slope, and bearing pad creep. Their research primarily involved numerical analyses with limited experimental testing of elastomeric steel and rubber bearing pads.

The design of bridge superstructures, including bearing pads, girders, and diaphragms is based on ideal conditions. However, after the fabrication and erection of girders, the actual conditions in the field can be different. Understanding the influence of geometric imperfections is extremely important, especially for predicting lateral stability of girders before the deck is in place. Consolazio and Hamilton (2007) illustrated the more realistic imperfections that are typical for bridge systems – sweep (Figure 2.9), skew angle (Figure 2.10), and slope (Figure 2.11).

Sweep, pictured in Figure 2.9, is horizontal bowing of a beam that can be caused by misaligned forms, prestressing with lateral eccentricity, improper girder storage, and thermal effects due to the exposure of the girder on one face more than the other. Sweep results in eccentricity of the beam's self-weight along the length.

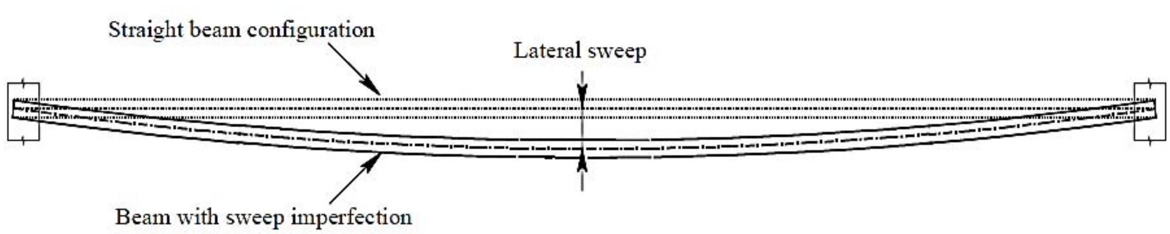

Figure 2.9. Definition of Sweep (Consolazio and Hamilton 2007)

Skew angle is a parameter of the bridge's geometry that is called for in structure's design, defined as the smaller angle between the support centerline and the roadway centerline. When the bearing pad is square on the support, the same skew angle is present between the pad and the longitudinal center of the roadway. However, lateral sweep imperfections can induce slight additional rotation at the supports. The "perfect" skew angle and the end rotation due to sweep are illustrated in Figure 2.10.

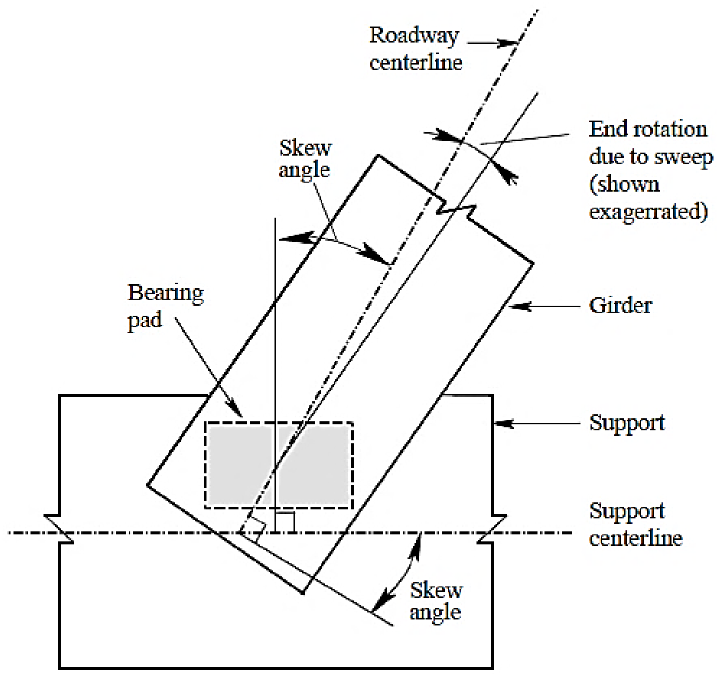

Figure 2.10. Relationship between Skew and Additional End Rotation due to Sweep Imperfections (Consolazio and Hamilton 2007)

When the bottom face of the girder does not rest flush on the top face of the bearing pads at the supports, there is a vertical angle between the longitudinal axis of the girder and the horizontal surface of the bearing pad, termed the "slope angle". This angle can result from camber of the girder created in prestressing or by the overall bridge grade. The slope angle affects the deflection and behavior of the bearing pad and its ability to resist lateral instability.

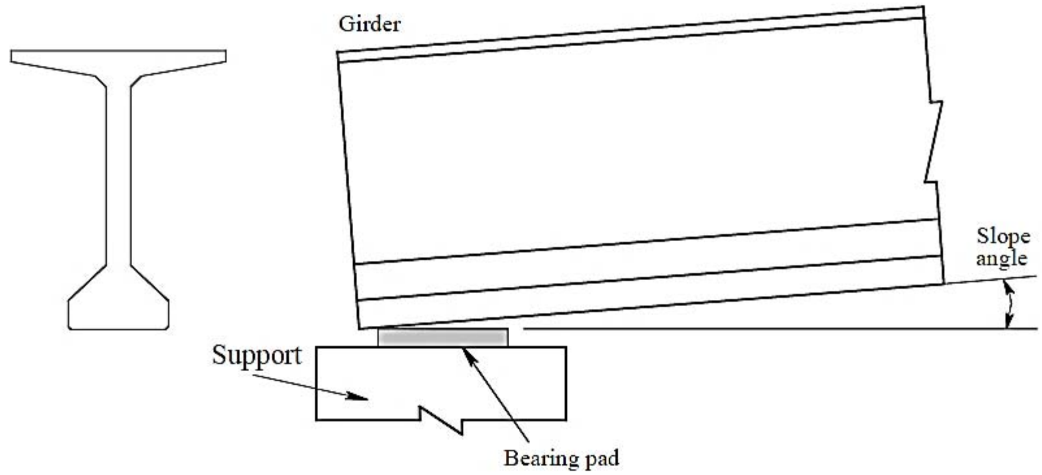

Figure 2.11. Definition of Slope Angle (Consolazio and Hamilton 2007)

The research by Consolazio and Hamilton (2007) focused on the bearing pads typically used by FDOT for simple span, prestressed concrete girder bridges – steel reinforced elastomeric bearing pads. These pads are used widely throughout the country, including in Alabama. These pads are made of multiple alternating layers of elastomeric material bonded to steel plates. The layers in combination are more ideal than either material alone; the steel layers help to limit the bulging of the bearing pad in compression, and the flexibility of the elastomer layers is important to accommodate bridge movements. An illustration of steel reinforced elastomeric bearing pad, and its general deformation behaviors are shown in Figure 2.12.

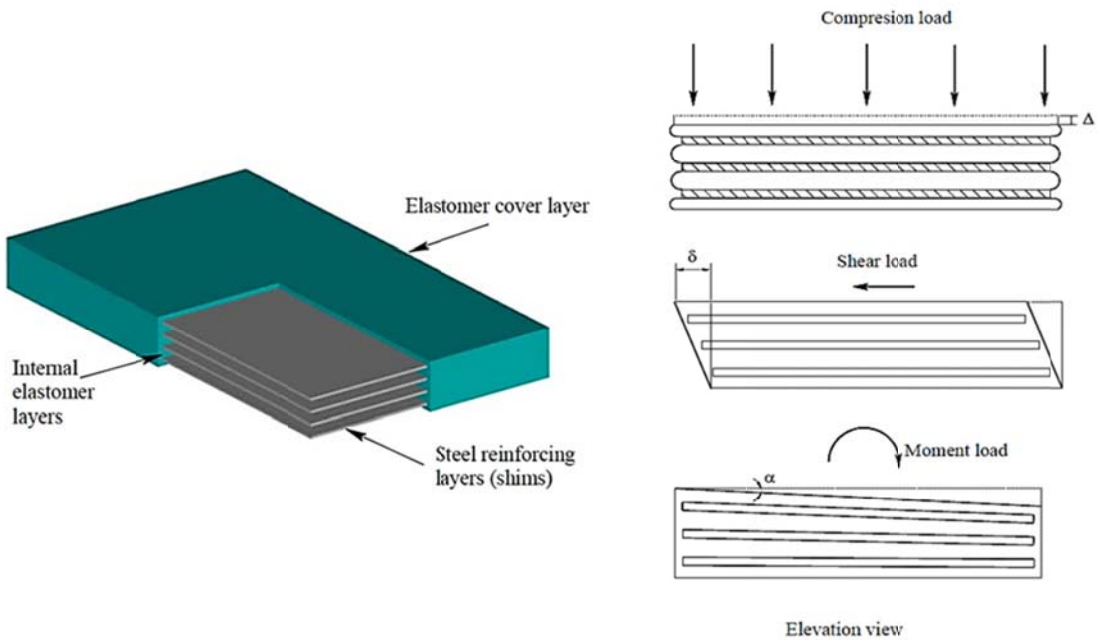

Figure 2.12. Steel Reinforced Elastomeric Bearing Pad and Deformation Behavior (Consolazio and Hamilton 2007)

When girders are set on bearing pads, the actual support boundary conditions are between fixed and free for translation and rotation. When considering lateral stability of the girders after erection, the exact degree of fixity of those parameters is based on the stiffness of the bearing pads, most importantly, the short term elastic stiffness. Experimental tests were carried out by Consolazio and Hamilton (2007) to better understand the realistic stiffness properties of bearing pads. Then, using 3-D modeling techniques with ADINA, they determined compression stiffness, shear stiffness, rotational stiffness, and torsional stiffness properties for an FDOT Type B bearing pad. Finally, the relationships between skew and rotational stiffness, and torsional angle and torsional moment were studied. The relationship they discovered between rotational stiffness and skew angle is very important to understanding the affect that skew has on the bearing pad's ability to resist rollover. As seen in Figure 2.13, as the skew angle increases, the rotational stiffness of the bearing pad significantly decreases, having a negative effect on the bearing pad's ability to resist rollover. The data in Figure 2.13 are based on FDOT's Type B elastomeric bearing pad, but Consolazio and Hamilton (2007) suggested that the general trend is the same for other elastomeric bearing pads.

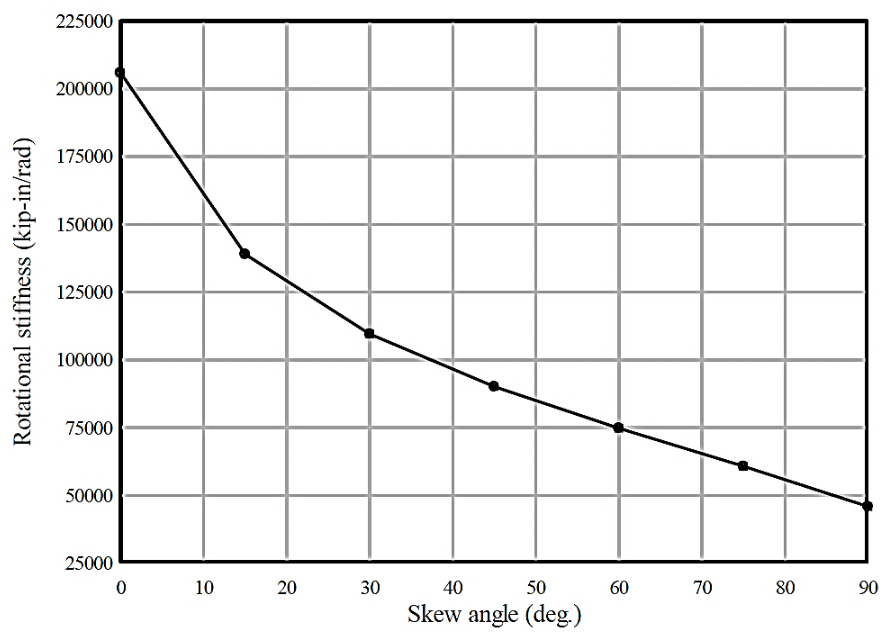

Figure 2.13. Rotational Stiffness vs. Skew Angle (Consolazio and Hamilton 2007)

Based on the data in Figure 2.13, Equation 2-4 describes the reduction in rotational stiffness for a given skew angle. This relationship can be used to find the rotational stiffness whenever experimental test data and rotational stiffness values are available. Table 2.5 combines the data of Figure 2.13 and Equation 2-4 to give the percent reduction in roll stiffness per skew which can be used to adjust the roll stiffness for any type of elastomeric bearing pad.

$$R_{skew} = \frac{K_{\theta\_skew°} - K_{\theta\_0°}}{K_{\theta\_90°} - K_{\theta\_0°}} \tag{2-4}$$

where

R_{skew} = percent reduction per skew

$K_{\theta\_skew°}$ = rotational stiffness at the skew being considered

$K_{\theta\_90°}$ = rotational stiffness for bearing pad Type B at 90° skew

$K_{\theta\_0°}$ = rotational stiffness for bearing pad Type B at 0° skew

Table 2.5. Percent Reduction Per Skew

$K_{\theta\_0°}$	$K_{\theta\_90°}$	$K_{\theta\_0°}$	$K_{\theta\_15°}$	$K_{\theta\_30°}$	$K_{\theta\_45°}$	$K_{\theta\_60°}$	$K_{\theta\_75°}$	$K_{\theta\_90°}$
206000	40000	206000	137500	110000	90000	75000	62500	40000
% Reduction, R_{skew}		0%	41%	58%	70%	79%	86%	100%

Slope and skew individually reduce the effectiveness of the bearing pad for stability. When the effects of slope and skew combine, the roll resistance of the bearing pad is further reduced due to the asymmetric and non-uniform distribution of bearing pressure, as shown in Figure 2.14. The slope mismatch between the bottom of the girder and the top of the bearing pad is typically a result of camber and bridge grade, making it essentially unavoidable. The skew between the girder and the bearing pad, however, can be eliminated by aligning the bearing pad with the longitudinal axis of the girder. Consolazio and Hamilton (2007) recommended that the bearing pad be placed unskewed relative to the girder, as shown in Figure 2.15, to improve girder stability.

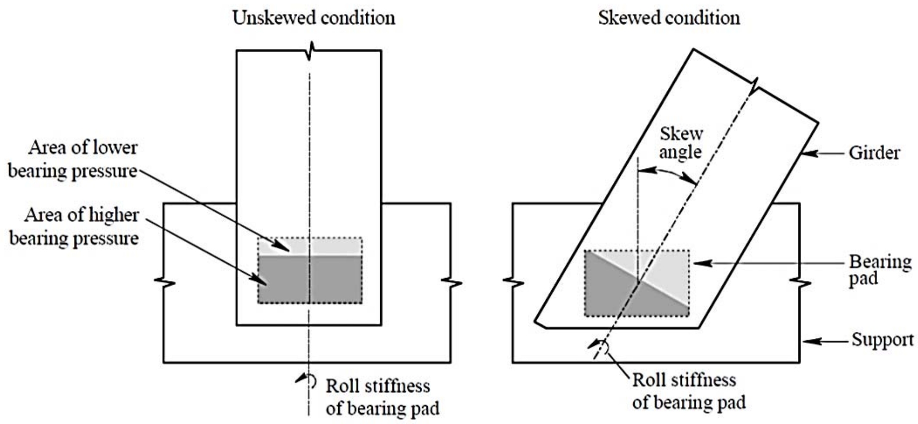

Figure 2.14. Combination of Slope and Skew (Consolazio and Hamilton 2007)

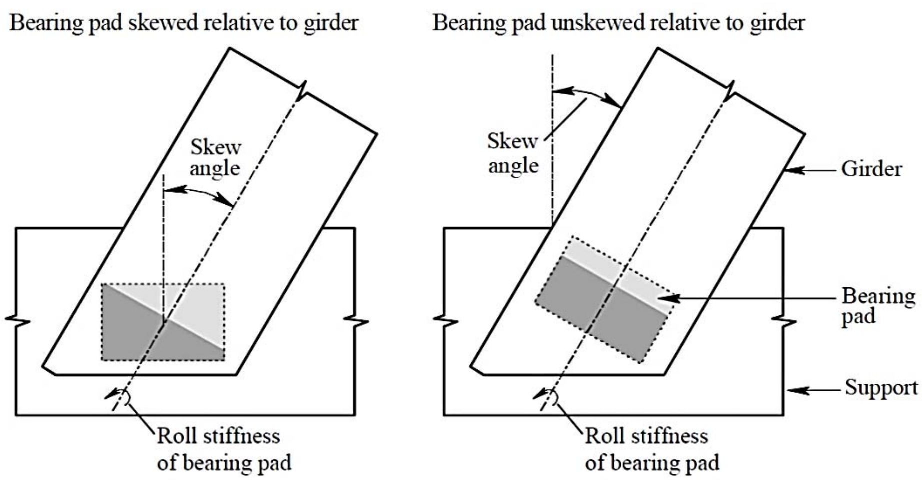

Figure 2.15. Alignment of Bearing Pad (Consolazio and Hamilton 2007)

Consolazio and Gurley (2013) followed the research by Consolazio and Hamilton (2007) and further investigated the lateral stability of long-span prestressed concrete girders, specifically looking at the effects of wind loads. Their study involved wind tunnel testing, further analysis of steel reinforced elastomeric bearing pad stiffness properties, and modeling of bridge systems to evaluate lateral stability of Florida I-beams. Unlike

Consolazio and Hamilton (2013), the focus of this study was on Florida prestressed I-Beams, rather than Florida Bulb-Tees.

Temporary bracing of girders during construction is helpful to prevent lateral instability. Consolazio and Gurley (2013) defined two basic types of bracing – anchor bracing and girder-to-girder bracing. The first girder in an erection sequence cannot be immediately braced to another girder for stability; therefore, *anchor bracing* is used to brace girder ends into the pier. Anchor bracing can be done through a variety of different methods; common methods are illustrated in Figure 2.16.

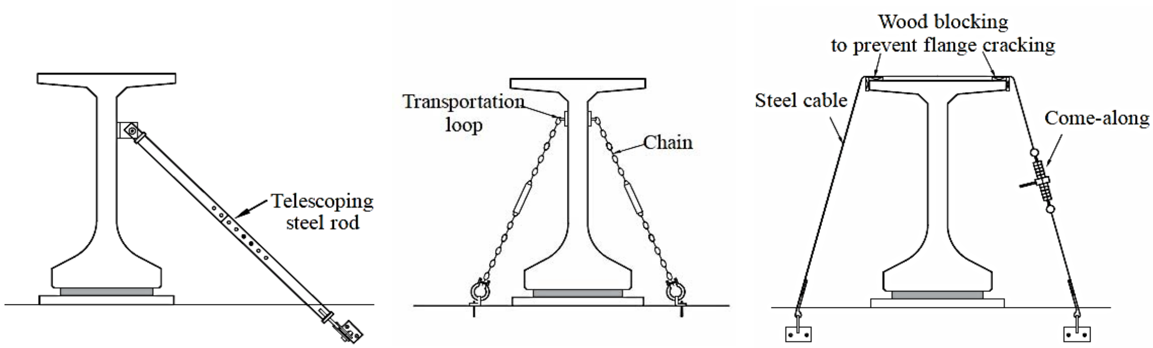

Figure 2.16. Common Anchor Bracing Methods (Consolazio and Gurley 2013)

After the erection of the initial girder, subsequent girders are connected via girder-to-girder bracing at the ends of the girder and often at intermediate points within the span. Temporary girder-to-girder bracing is typically constructed from timber or rolled-steel members and can take a variety of different configurations. Examples of common brace types illustrated by Consolazio and Gurley (2013) are shown in Figure 2.17. Individual brace designs are left to the discretion of the contractor in Florida, resulting in a wide variety of bracing layouts used in practice. The examples in Figure

2.16 and Figure 2.17 illustrate the common methods used for the temporary bracing of girders.

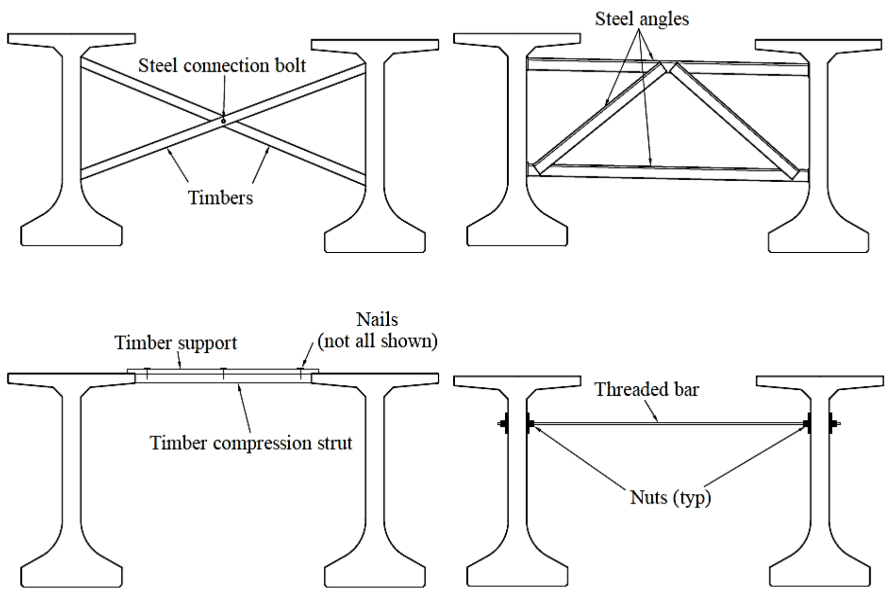

Figure 2.17. Girder-to-Girder Bracing Methods (Consolazio and Gurley 2013)

The major focus of the study by Consolazio and Gurley (2013) was the effects of wind loads on lateral stability. The designs of bridges in Florida are subject to the provisions in the FDOT Structures Design Guidelines (SDG) (FDOT 2014), which includes requirements to design for wind loads in SDG Section 2.4. The design wind pressure, P_Z, is found according to Equation 2-5.

$$P_z = (2.56 \times 10^{-6}) K_Z V^2 G C_P \quad (ksf) \tag{2-5}$$

where

K_Z = velocity pressure coefficient

V = basic wind speed (mph) [by county, according to FDOT SDG Table 2.4.1-2]

G = gust effect factor (0.85 for bridges with spans < 250 feet and a height < 75 feet; for bridges with spans > 250 feet or a height > 75 feet, G shall be evaluated according to ASCE/SEI 7)

C_P = pressure coefficient (2.2 for single I-girder, 2.75 for maximum bracing)

The pressure coefficient for girders during construction is not well understood or supported by existing research and is going to be further addressed with the publication of the 2015 FDOT SDG wind provisions. The velocity pressure coefficient, K_Z, modifies the wind pressure load, P_z, to account for the change based on the elevation. Typically the roughness of the terrain would affect the behavior of this coefficient, but the FDOT SDG conservatively assumes that the all Florida bridges are classified as Exposure C – mostly open terrain with low, scattered obstructions. The trend of K_Z used by FDOT is shown in Figure 2.18. Equation 2-6 can be used to calculate the velocity pressure coefficient, K_Z , for a specific structure.

$$K_z = 2.01 \left(\frac{z}{900}\right)^{0.2105} \geq 0.85 \tag{2-6}$$

where

z = bridge height (ft), measured to the mid-height of the beam

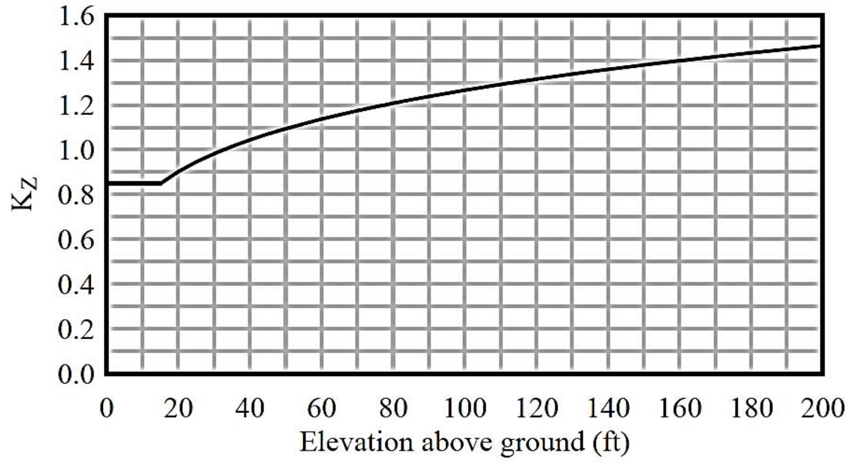

Figure 2.18. K_Z Coefficient used by FDOT (Consolazio and Gurley 2013)

Consolazio and Gurley (2013) followed up on the previous bearing pad stiffness recommendations by Consolazio and Hamilton (2007), by further developing the bearing pad rotational (roll) stiffness calculations. The roll stiffness of bearing pad can be calculated according to Equation 2-7.

$$k_{roll} = p^2(3-2p)\frac{k_{axial}W^2}{45} \tag{2-7}$$

where

k_{axial} = total axial stiffness of the bearing pad

W = width of the pad in the direction perpendicular to the roll axis

p = portion of the pad that is contact with the girder [Equation 2-8]

$$p = \sqrt{\frac{9F_{axial}}{2L\emptyset_{slope}k_{axial}}} \leq 1.0 \qquad (2\text{-}8)$$

where

F_{axial} = initial axial load (i.e., the reaction on the pad due to the girder weight)

$\emptyset_{slope}$ = girder slope angle between top of bearing pad and the bottom face of girder

L = width of the pad in the direction parallel to the roll axis

The dimensions of W and L are based upon the orientation of the roll axis (girder centerline); therefore these dimensions are a function of the skew of the girder with respect to the support centerline, as previously illustrated in Figure 2.10. Figure 2.19 illustrates the dimensions of W and L for 0° and 90° skews.

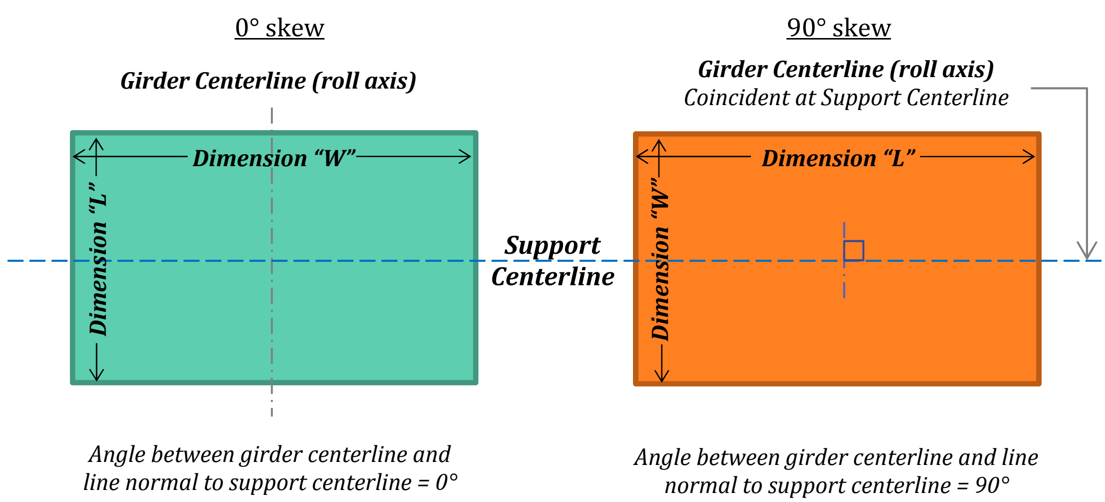

Figure 2.19. Definition of W and L for Equations 2-7 and 2-8

Skewed angles range from 0° and 90°. The reduction method proposed by Consolazio and Hamilton (2007) allows for interpolation between the roll stiffness at 0° and the much smaller roll stiffness at 90°. To calculate the rotational stiffness of the bearing pad at any skew, the rotational stiffness is first calculated according to Equation 2-7 for both 0° and 90°, separately; the dimensions of W and L are opposite for the two calculations. Then, the rotational stiffness at the specific skew can be calculated using those values and reduction method previously given in Equation 2-4 and Table 2.5. The results of the calculations by Consolazio and Gurley (2013) for all Florida bearing pad types are shown in Table 2.6. Bearing pad dimensions and variables are illustrated in Figure 2.20

Table 2.6. FDOT Bearing Pad Properties (Consolazio and Gurley 2013)

	Pad Type	L (in)	W (in)	G (psi)	n	K_{shear} (kip/ft)	K_{axial} (kip/ft)	$k_{torsion}$ (kip-ft/rad)	$k_{roll,overturning}$ (*for zero slope*) (kip-ft/rad)	$k_{roll,bending}$ (kip-ft/rad)
FBTs	A	11	24	110	2	232	71000	46.4	6330	1330
	B	14	24	110	3	222	85300	64.0	7600	2590
	C	12	23	150	3	248	72200	55.8	5900	1610
FIBs	D	8	32	110	2	225	45900	28.1	7270	458
	E	10	32	110	2	282	81400	52.4	12900	1260
	F	10	32	110	3	211	57300	39.3	9080	890
	G	10	32	150	3	288	72700	53.6	11500	1130
	H	10	32	150	4	230	56300	42.8	8910	870
	J	10	32	150	5	192	45900	35.7	7260	712
	K	12	32	150	5	230	70200	58.7	11100	1560

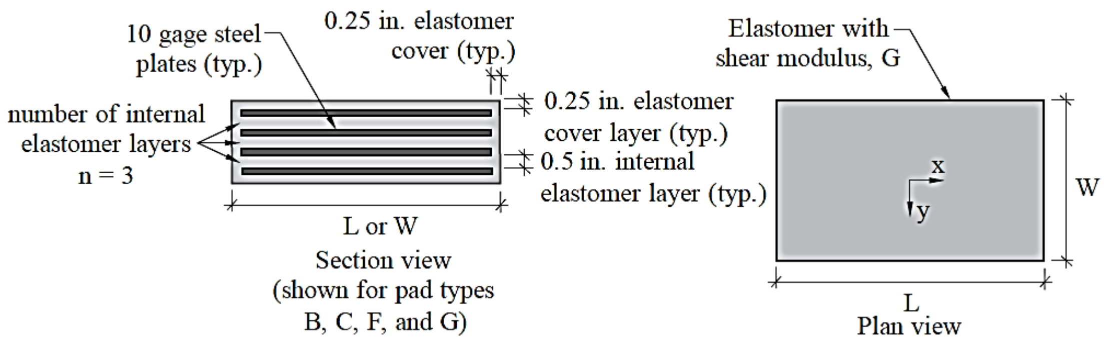

Figure 2.20. Bearing Pad Dimensions and Variables Corresponding to Table 2.6 (Consolazio and Gurley 2013)

The data in Table 2.6 give the rotational stiffness for bearing pads at 0° skew (labeled "$k_{roll,overturning}$") and at 90° skew (labeled "$k_{roll,bending}$"). The rotational stiffness for a bearing pad at any intermediate skew can be found using the data in Table 2.6, combined with Equation 2-4 and the reductions in Table 2.5.

Collectively, the research efforts by Mast (1993), Consolazio and Hamilton (2007), and Consolazio and Gurley (2013) have been extremely important to understanding the issues of lateral instability of prestressed concrete girders during construction.

2.5 Current Design Codes and Publications

The *Precast/Prestressed Concrete Institute (PCI) Bridge Design Manual (PCI BDM)* (2011) addresses diaphragms in Section 3.7. PCI does not take a definitive stance on the necessity of intermediate diaphragms or a preference in the type of diaphragm that should be used, but rather outlines common practices. The manual includes descriptions and typical details for cast-in-place concrete diaphragms, precast concrete diaphragms, steel diaphragms, and temporary diaphragms for construction.

Cast-in-place diaphragms, the most common for end and intermediate diaphragm applications, were illustrated using Figure 2.21. The PCI BDM uses the following description for the construction of cast-in-place diaphragms:

> *"Interior beams are fabricated with holes through the web to allow the top and bottom diaphragm reinforcement to pass through. Exterior beams have threaded inserts embedded in the interior face to accommodate threaded reinforcing steel, bolts or other types of anchors. In lieu of threaded inserts, some exterior beams are cast with holes through the web and a recessed pocket in the exterior face. Threaded reinforcement is passed through the hole, and secured with hand-tightened nut and washer. After the diaphragm concrete has gained some strength, the nut is tightened firmly, and the recess is coated with epoxy and patched with grout." (PCI 2011)*

While details by state can vary, Figure 2.21 is very helpful at describing the general layout of a cast-in-place concrete diaphragm.

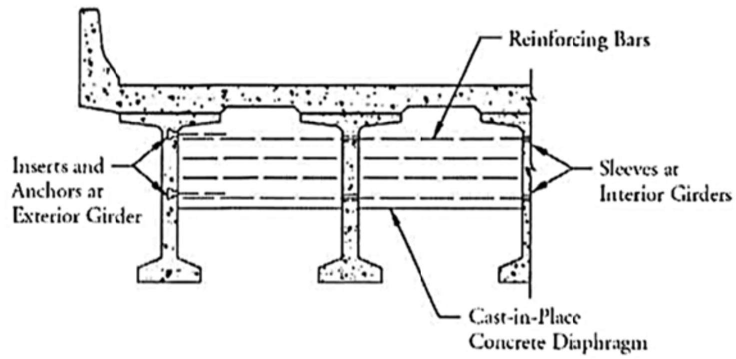

Figure 2.21. Cast-in-Place Concrete Diaphragm Details (PCI 2011)

The next diaphragm type discussed in the PCI BDM (2011) is precast concrete diaphragms. This type of diaphragm is extremely uncommon due to the complexity of fabrication and erection requirements, but PCI does give two examples – 1) an individual, separate, precast diaphragm and, 2) a secondary-cast, precast diaphragm. Individual, separate, precast diaphragms, illustrated in Figure 2.22, are cast as separate precast pieces to conform to the webs and flanges of adjacent beams and arrive lose for installation. Extra care must be taken to ensure that fabrication and erection tolerances are within acceptable limits, and connections are typically made by welding, which can be tedious during erection.

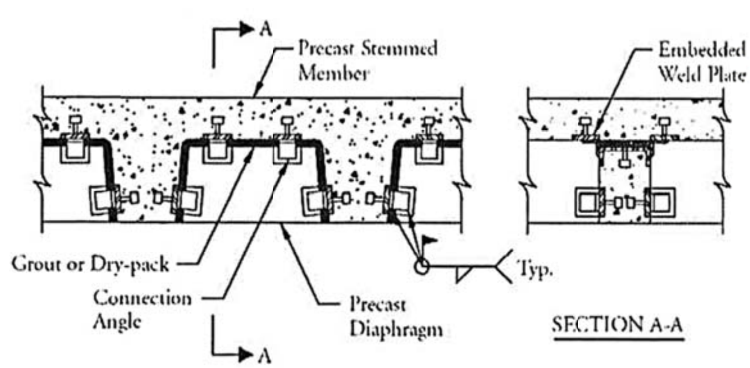

Figure 2.22. Individual, Separate, Precast Concrete Diaphragms (PCI 2011)

According to the PCI BDM (2011), secondary-cast, precast concrete diaphragms are built by casting the diaphragms directly onto an individual beam in the precast yard and the reinforcement and diaphragm-beam connection is similar to a cast-in-place diaphragm. An example of secondary-cast, precast concrete diaphragms is shown in Figure 2.23. As shown in the figure, the joint is typically at the midpoint of the diaphragm and the connection is typically accomplished by welding or mechanical splicing of exposed reinforcement. Matching the diaphragms in the field is critical and requires extra attention in the fabrication process.

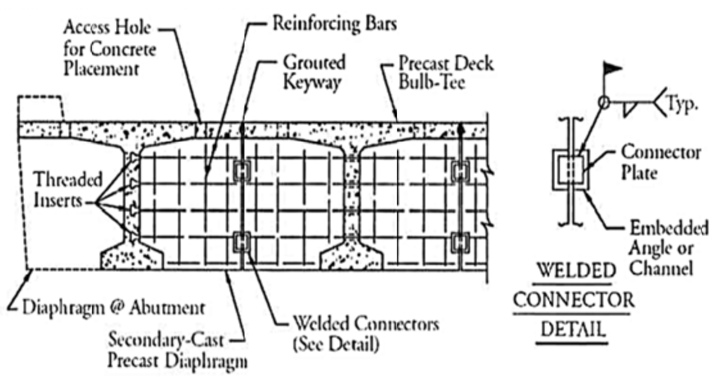

Figure 2.23. Secondary-Case, Precast Concrete Diaphragms (PCI 2011)

Both precast diaphragm options are extremely sensitive to fabrication and erection processes, making them harder to execute that the cast-in-place concrete or steel bracing options that are more popular in intermediate diaphragm design. The PCI BDM (2011) illustrates two types of steel diaphragm configurations – 1) K-braces, and 2) Delta braces; steel channel and X-brace configurations are not mentioned. The steel diaphragms illustrated in the manual are presented in Figure 2.24 (K-Brace) and Figure 2.25 (Delta Brace).

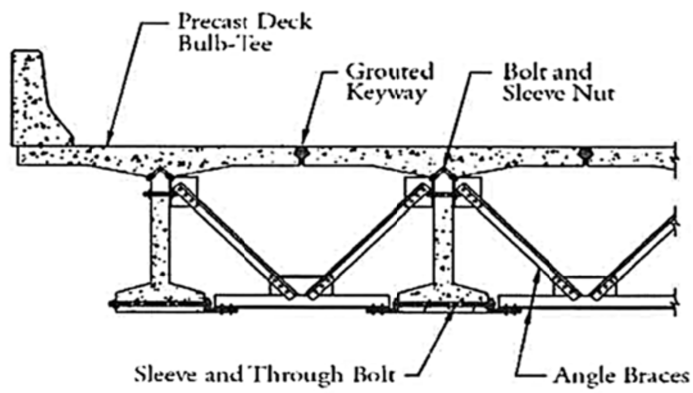

Figure 2.24. K-Brace Diaphragms (PCI 2011)

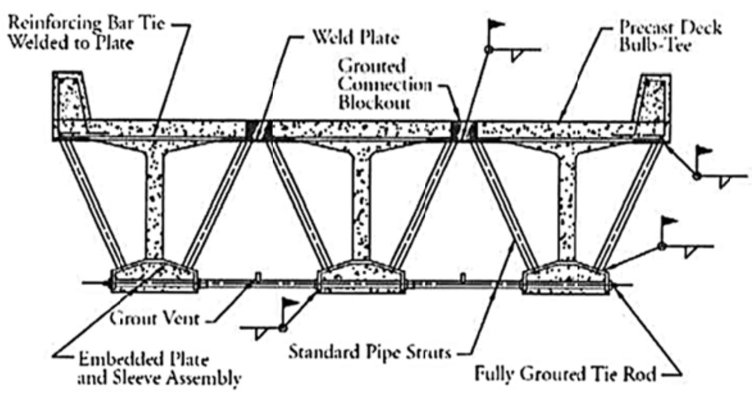

Figure 2.25. Delta Brace Diaphragms (PCI 2011)

While PCI (2011) has chosen to illustrate k-braces and delta braces as examples for steel diaphragms, these illustrations do not resemble the steel diaphragm options used by any state transportation department at the time of this research.

PCI (2011) additionally addresses the importance of diaphragms during construction and alignment of diaphragms in skewed bridges. According to PCI, forces of nature, including wind, earthquake, or thermally-induced sweep can cause unbraced, free-standing girders to topple off of their supports. Temporary braces can be installed to stabilize beams during erection and deck pouring operations, and then removed after the final connections are made. As addressed by PCI, alignment of diaphragms in skewed structures can be done so in two ways – following the skew angle, or perpendicular to the beams. The major difference between the two is in the complexity of the connection; setting diaphragms perpendicular to the beams is easier in both detailing and execution.

In general, PCI (2011) provides an in-depth overview of intermediate diaphragms, although the illustrations provided are not representative of current intermediate diaphragms used.

The 2012 *AASHTO LRFD Bridge Design Specifications* address intermediate diaphragms very briefly in section 5.13.2.2. AASHTO LRFD (2012) specifies that end diaphragms (or edge beams) should be provided at points of support to "resist lateral forces and transmit loads to points of support." It is suggested that intermediate diaphragms in curved bridges may be used, and their necessity is determined based on the geometry of the curve and diaphragms used. However, intermediate diaphragms are not discussed as necessary to all bridge structures. While the PCI BDM (2011) provides a few pages of discussion on intermediate diaphragms, AASHTO LRFD (2012) contains a few sentences, and no specifics.

2.6 Variations in Intermediate Diaphragm Design Practice

As part of their larger study, focused on the effects of intermediate diaphragms in the event a bridge is struck by a vehicle, Abendroth et al. (1991) conducted a survey with bridge design agencies in the United States and Canada to profile the types of intermediate diaphragms being required. Their survey was answered by 86% of the 64 design agencies contacted, and was able to profile the use of intermediate diaphragms by the responsive entities. The following statistics are representative of the responses they received at the time of the survey.

- *Specifying intermediate diaphragms*: 95% had specified intermediate diaphragms for prestressed girder bridges in the past; 85% still required those intermediate diaphragms
- *Type of intermediate diaphragms*: 96% used cast-in-place intermediate diaphragms for bridges with vehicular or marine traffic below; 23% also specified steel channel alternates
- *Spacing requirements*: 50% placed one intermediate diaphragm at midspan; 30% required that they be placed at one-third points within the span; 10% located them at one-quarter points

Garcia (1998) outlined current design issues for intermediate diaphragms in prestressed concrete bridges. His report outlined the use of intermediate diaphragms by the Florida Department of Transportation (FDOT), the necessity of intermediate diaphragms in skewed structures, and profiled the use of intermediate diaphragms in all 50 states and Puerto Rico at that time. In the late 1980’s, FDOT reconsidered some of their traditional design practices to ensure that their designs were as economical as possible; this included re-evaluating the use of intermediate diaphragms. To this point, research conclusions varied from those deeming intermediate diaphragm as unnecessary, to those in favor of intermediate diaphragms. It was determined by FDOT that the costs and time saved by eliminating intermediate diaphragms trumped the *possible* benefits they might have on the structure. After a successful petition to the Federal Highway Administration (FHWA) to eliminate intermediate diaphragms in tangent bridges, the design change was agreed to, but a caveat required that the design live load be increased five percent as a measure of safety. The cost to increase the strength of the girders was still more economical than building intermediate diaphragms. This design change was not applicable to bridges skewed greater than 30°. As discussed by Garcia (1998), there are three cases where the necessity of intermediate diaphragms is largely undisputed: 1) bridges with large skews, 2) curved girder bridges, and 3) during construction.

Through their survey of bridge design agencies in all 50 states and Puerto Rico, Garcia (1998) was able to profile temporary diaphragm requirements across the United States; his is survey did not include the type of intermediate diaphragms specified. He reported the following: 44 states required the use of intermediate diaphragms for all bridges, two states (TN, CO) used intermediate diaphragms in some cases, and six states (CA, FL, IL, NE, ND, TX) did not use intermediate diaphragms. It is important to note, these values are not the same as those obtained at present day for this report (Chapter 3), signaling developments and changes since the time of Garcia's research.

As a smaller element of their study analyzing the effects of intermediate diaphragms in the event the bridge is struck by an overheight vehicle, Abendroth et al. (2004) conducted a survey of design practices through the United States. The responses from their questionnaire enabled them to create a detailed profile of intermediate diaphragm practices by the 38 state transportation agencies that responded. The following results were obtained through their survey:

- *Type of intermediate diaphragms:* 36 of the 38 states that responded specified intermediate diaphragms and 14 of those states permitted structural steel to be used. Therefore, approximately 37% of those agencies that responded specify permanent steel intermediate diaphragms.
- *Purpose for using intermediate diaphragms:* 90% of states used the diaphragms to lateral stability during construction, and 70% states did not consider possible benefits during an impact event as a factor in their inclusion of diaphragms.

According to the survey results presented by Abendroth et al. (1991), 23% of those agencies that responded allowed steel channel intermediate diaphragms. Approximately 13 years later, resulting from a survey by Abendroth et al. (2004), 37% of the responding state transportation agencies responded that the use of permanent steel diaphragms was permitted in their state. This comparison shows the increase in steel diaphragm usage in recent decades.

2.7 Literature Review Summary

There have been extensive research efforts to understand the performance and effectiveness of intermediate diaphragms during erection and in-service. It is widely agreed upon that intermediate diaphragms are essential immediately after girder erection and during construction operations to provide lateral stability to the girders. After the deck is constructed, and during the service-life of the structure, the importance of intermediate diaphragms is not agreed upon. A couple of research efforts have involved variations of surveys to determine typical practices for intermediate diaphragms between states. Those surveys showed wide variations in design practices, but compared to each other, indicate a trend toward steel intermediate diaphragm alternates in the last two decades. There is no national standard for the design of intermediate diaphragms and a lack of agreement on the effectiveness of intermediate diaphragms, leading to variations in intermediate diaphragm design across the U.S.

Chapter 3: Survey of Current Intermediate Diaphragms Practices

Research and design publications, discussed in Chapter 2, agree on the importance of lateral bracing during the erection of girders and construction procedures before the deck is in place and has gained strength. However, there is not a national standard for the specification of intermediate diaphragms, leading to variations in intermediate diaphragm usage across the U.S. To understand the current design trends and methodologies, a survey of the current intermediate diaphragm design practices in the U.S. was completed.

3.1 Description of Survey

To document the current use of intermediate diaphragms across the U.S., a survey was conducted through two avenues: 1) the current requirements and standards were researched through each individual design agency's website, and associated resources online, and 2) direct contact with bridge engineers within each agency. Most states (27) had adequate information available online; the remaining 23 state design agencies were contacted directly with a brief questionnaire. The following questions were asked of those agencies; responses were received from all 23 states, although follow-up requests were required for some states.

1. What type of intermediate diaphragms are specified or allowed as alternates for precast concrete girder bridges in your state (i.e., cast-in-place concrete, steel channels or cross bracing, precast)?
2. When are intermediate diaphragms required for precast concrete girder bridges in your state?
3. How were the standards listed as answers to questions 1 and 2 developed, and have they changed in the recent past? What major concerns or benefits are addressed by these standards?
4. What standard details for intermediate diaphragms do you use? Would you be willing to share those with us?

3.2 Survey Results

The results of this survey provide a profile of the intermediate diaphragm usage by all 50 U.S. state transportation departments. Standard details for intermediate diaphragms were obtained for all states that had them. A summary of the results is available in Table 3.1 and trends are illustrated by Figure 3.1 and Figure 3.2. More details on the use of steel intermediate diaphragms (Chapter 4) and temporary bracing alternatives (Chapter 5) are provided later in this thesis. The data in this chapter are meant to highlight the current trends in the type of intermediate diaphragms being used in practice by state bridge design agencies.

Table 3.1. Summary of Intermediate Diaphragms Used Across the U.S.

State	Permanent Intermediate Diaphragms				Temporary During Construction Only	No Standard
	Cast-In-Place Concrete	Steel Channel/ Bent Plate/ I-Beam	Steel K-Brace	Steel X-Brace		
Alabama					✓	
Alaska	✓					
Arizona	✓					
Arkansas	✓	✓		✓		
California	✓					
Colorado		✓				
Connecticut	✓					
Delaware	✓					
Florida					✓	
Georgia	✓					
Hawaii	✓					
Idaho						✓
Illinois		✓		✓		
Indiana		✓		✓		
Iowa		✓				
Kansas					✓	
Kentucky		✓		✓		
Louisiana	✓					
Maine						✓
Maryland	✓					
Massachusetts	✓					
Michigan	✓	✓		✓		
Minnesota		✓		✓		
Mississippi	✓					
Missouri		✓				
Montana	✓					
Nebraska	✓					
Nevada	✓					
New Hampshire						✓
New Jersey		✓		✓		
New Mexico		✓				
New York	✓	✓	✓			
North Carolina	✓	✓	✓			
North Dakota	✓					
Ohio	✓			✓		
Oklahoma	✓					
Oregon	✓					

(Table 3.1 Continued)

State	Permanent Intermediate Diaphragms				Temporary During Construction Only	No Standard
	Cast-In-Place Concrete	Steel Channel/ Bent Plate/ I-Beam	Steel K-Brace	Steel X-Brace		
Pennsylvania	✓					
Rhode Island	✓					
South Carolina		✓		✓		
South Dakota		✓		✓		
Tennessee	✓			✓		
Texas					✓	
Utah		✓	✓			
Vermont						✓
Virginia	✓	✓		✓		
Washington	✓					
West Virginia						✓
Wisconsin		✓		✓		
Wyoming	✓	✓				
Total	28	19	3	13	4	5

Figure 3.1 and Figure 3.2 show the distribution of the types of intermediate diaphragms used throughout the U.S. Figure 3.1 focuses on the geographic distribution of the types of intermediate diaphragms used by states – concrete only, concrete or steel, steel only, temporary only, and those with no standards for intermediate diaphragms. Through the survey, multiple states stated that they modeled their intermediate diaphragms off of the existing designs of a neighboring state. As seen in Figure 3.1, the multiple groupings of similar intermediate diaphragms type (same color) support that concept.

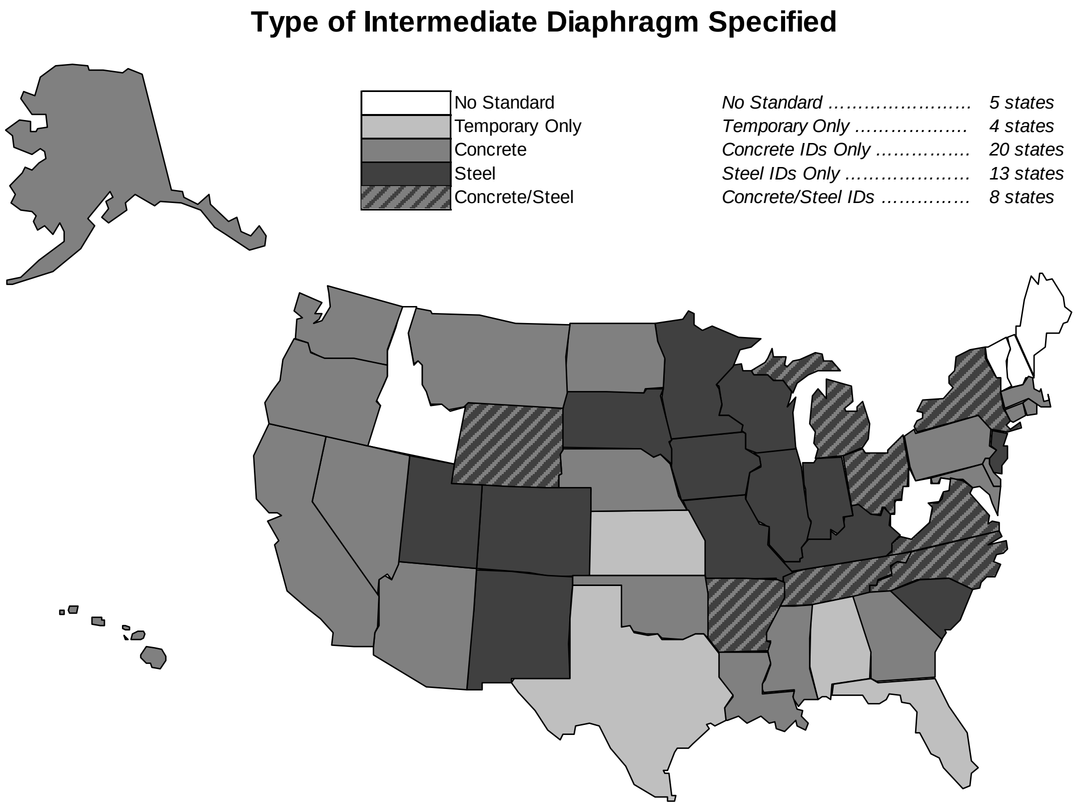

Figure 3.1. Geographic Distribution of Intermediate Diaphragm Types

Figure 3.2 shows the distribution of intermediate diaphragm practices in terms of percentages, using the same categories as Figure 3.1. The largest portion of states allows only concrete intermediate diaphragms – 40 percent of the U.S. (20 states). The next most common trend is the 26 percent of the U.S. (13 states) requiring the use of steel intermediate diaphragms. In Figure 3.1, states allowing concrete or steel are filled with the hatched pattern created from the combination of shades representing steel and concrete. States allowing concrete or steel intermediate diaphragms account for 16 percent of the U.S. (8 states). With a much smaller representation, 10 percent of the

U.S. (5 states) currently has no requirement for the type of intermediate diaphragms allowed, and 8 percent of the U.S. (4 states) allows only temporary bracing methods.

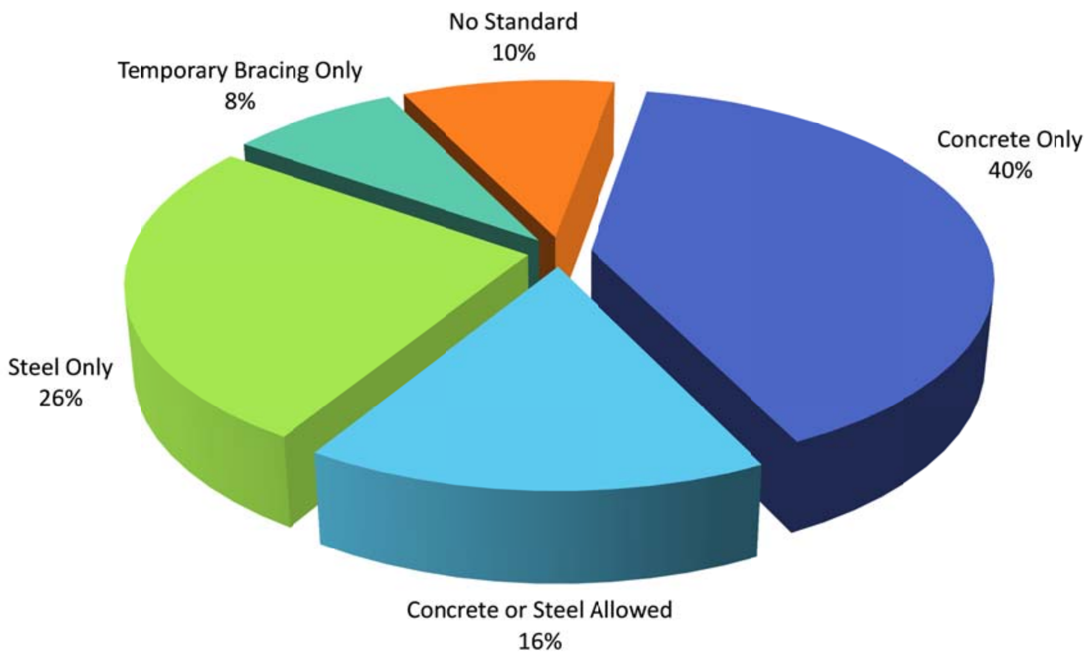

Figure 3.2. Types of Intermediate Diaphragms Used Throughout the U.S.

Comparing the data in Figure 3.2 with the data gathered by the surveys of Abendroth et al. (1991) and Abendroth et al. (2004) shows the change in intermediate diaphragm trends. The survey results presented by Abendroth et al. (1991) indicated that approximately 23% of transportation agencies permitted steel intermediate diaphragms. Through a more recent survey, Abendroth et al. (2004) indicated that approximately 37% of state transportation agencies specified steel intermediate diaphragms. Based on the survey results presented in Figure 3.2, the combination of states allowing only steel diaphragms and those states permitting steel diaphragm alternates accounts for 42% of state transportation agencies. The popularity of steel intermediate diaphragms has greatly increased in the recent decades, likely due the preference of steel intermediate diaphragms by many contractors and the re-evaluation of traditional concrete intermediate diaphragms by state design agencies.

In addition to the intermediate diaphragm design details obtained through this survey, the responses received from the questionnaire provided insight into the reasons behind the standard designs for each state. In general, there is a lack of consistency in the variety of reasons behind the design method(s) used by each state. For example:

- ***Alabama:*** The Alabama Department of Transportation (ALDOT) recently transitioned the responsibility of the bracing design to the contractor and does not specify the type of bracing. Prior to August 2013, cast-in-place reinforced concrete intermediate diaphragms were required in precast girder bridges and ALDOT provided the diaphragm design details. Effective immediately upon the change in August 2013, permanent intermediate diaphragms are no longer designed by ALDOT. ALDOT does not consider the effect of intermediate diaphragms on live load distribution to be significant, and believes the most important purpose of intermediate diaphragms is to provide stability during construction.
- ***Alaska:*** The Alaska Department of Transportation abandoned steel K-brace cross frames due to recurring constructability issues – welding, cracking, and fit-up issues. The use of cast-in-place intermediate diaphragms has

eliminated constructability issues and has been proven to be more cost effective (Elmer Marx, personal communication, May 10, 2013).

- ***Delaware:*** The Delaware Department of Transportation (DelDOT) modeled their intermediate diaphragm details from those used by the Maryland Department of Transportation. Delaware is currently revising the DelDOT *Bridge Design Manual* (DelDOT 2005). This includes looking into steel intermediate diaphragm alternatives, and re-evaluating the concrete intermediate diaphragm details that are currently used, which were developed in the early 1990s (Jason Hastings, personal communication, June 6, 2013).
- ***Florida:*** The Florida Department of Transportation (FDOT) does not use permanent intermediate diaphragms due to concerns that they increase damage to the bridge in the event of a significant impact by an overheight vehicle. FDOT prefers to have the damage concentrated at the exterior girder, rather than the load transferring to adjacent girders and increasing the number of damaged girders (Robert Robertson Jr., personal communication, May 10, 2013). FDOT requires temporary bracing during construction and has developed a *Beam* Stability program (FDOT 2013) to evaluate the lateral stability of girders during construction.
- ***Hawaii:*** The Hawaii Department of Transportation requires cast-in-place concrete intermediate diaphragms to assist in the distribution of live loads

between girders, unless an in-depth analysis concludes they aren't required (Paul Santo, personal communication, May 10, 2013).

- ***Idaho:*** The Idaho Transportation Department (ITD) does not specify details for intermediate diaphragms, but they have used both steel and concrete intermediate diaphragms. As the number of accelerated bridge construction projects by ITD has increased, they have tended towards steel intermediate diaphragms (Elizabeth Shannon, personal communication, May 14, 2013).
- ***Mississippi:*** The Mississippi Department of Transportation (MDOT) is currently giving consideration to steel cross bracing; they currently only allow cast-in-place concrete intermediate diaphragms. MDOT uses intermediate diaphragms to distribute live loads and help distribute impact forces in the case of a hit by an overheight truck (Nick Altobelli, personal communication, May 13, 2013).
- ***North Carolina:*** The North Carolina Department of Transportation specifies the use of steel intermediate diaphragms in typical environments, but requires the use of cast-in-place concrete intermediate diaphragms in corrosive environments (Brian Hanks, personal communication, June 5, 2013).
- ***Tennessee:*** The Tennessee Department of Transportation allows steel or concrete intermediate diaphragms. The intermediate steel cross frames were designed to expedite construction, and are commonly selected in lieu of the concrete alternate (Rick Crawford, personal communication, May 10, 2013).

While the type of intermediate diaphragms used varies throughout the U.S., one thing is agreed upon by all of them – intermediate diaphragms or temporary bracing is required to provide lateral stability to girders during construction. Unbraced girders are vulnerable to rollover after erection until the deck achieves a minimum required strength. There has been a move by a few state bridge design agencies, including those in Alabama, Florida, Kansas, and Texas, to eliminate permanent intermediate diaphragms and require temporary bracing only. The following chapters provide a detailed summary of the current practices of steel intermediate diaphragms (Chapter 4) and temporary bracing schemes (Chapter 5).

Chapter 4: Steel Intermediate Diaphragms

Based on the survey of intermediate diaphragm practices discussed in Chapter 3, it was determined that 21 states specify steel intermediate diaphragms, either exclusively (13 states) or along with concrete alternates (8 states). There currently is not a national standard guiding the use of steel intermediate diaphragms, leading to wide variations in design and application throughout the U.S. This chapter will cover the many elements involved in the design of steel intermediate diaphragms – configuration, connections, spacing, and alignment – highlighting the similarities and difference between states. Section 4.1 describes the different configurations of steel intermediate diaphragms and the general diaphragm-to-girder connection schemes used with each type. Section 4.2 lists, by state, the type of steel diaphragm that is specified based on prestressed girder shape and size. Section 4.3 explains the variations in intermediate diaphragm alignments based on bridge skew. Section 4.4 defines the differences in intermediate diaphragm spacing specified by each state.

4.1 Typical Configurations and Connection Schemes

Steel intermediate diaphragms are used in a few different general forms – steel channels, X-braces, or K-Braces – with varying dimensions, layouts, and connections. According the survey (Chapter 3), channels/bent plates/I-beams are specified for intermediate diaphragms in 19 states, K-brace diaphragms are specified in 3 states, and X-brace diaphragms are specified in 13 states. The illustrations in this section are generalizations summarized from individual state details.

The steel channel intermediate diaphragm illustrated in Figure 4.1 is a generalization of the 19 states (with exception of Colorado) that specify diaphragms consisting of steel channels or bent plates (17 states), or steel I-beams (2 states). Figure 4.1 is also used to show the two types of connections used to anchor the diaphragms to the girders – threaded inserts used to bolt diaphragms *into* the girder (Option 1) or formed holes with bolts passed through the girders and anchored on the opposite side (Option 2). The tightening of bolts is specified as “snug tight” in some standard details; however, others do not specify requirements.

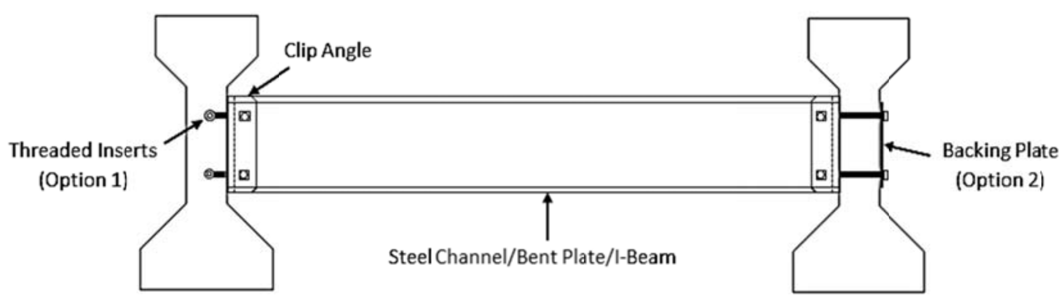

Figure 4.1. Steel Channel Intermediate Diaphragm

The details of diaphragm-to-girder connections vary widely between states, including variation between exterior and interior girder connections. The connection methods in Figure 4.1 are also used with the other steel intermediate diaphragms (X-brace and K-brace) and have been omitted from those schematics for simplicity.

Figure 4.1 is not a generalization of the I-beam intermediate diaphragm used in Colorado due to the unique hardware designed to connect the I-beam to the girder. The design used in Colorado (Figure 4.2) consists of a clip angle connecting the beam's bottom flange to the girder's web, and a unique bent plate connecting the beam's top flange to the girder's top flange. All connections to the girders are made with inserts, bolts, and lock washers.

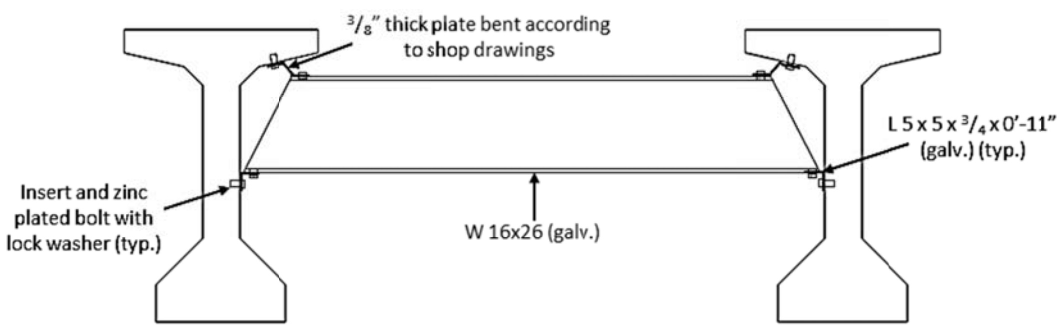

Figure 4.2. Steel Intermediate Diaphragm Used by Colorado DOT

Seven states specify the use of an X-brace without a horizontal strut (Figure 4.3) and six states call for the use of an X-brace with a bottom horizontal strut (Figure 4.4). The diagonals and struts in these configurations are formed with steel angles.

The connections for steel X-brace intermediate diaphragms vary between states; some states connect the diagonals using a single connection angle (Figure 4.3, Option 1), and others use individual clip angles for each diagonal (Figure 4.3, Option 2). The anchorage of the clip angle into or through the girders (as seen in Figure 4.1, Options 1 and 2) varies among states.

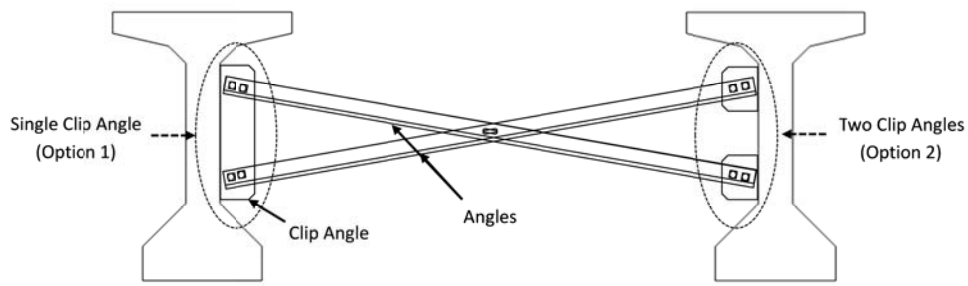

Figure 4.3. Steel X-Brace Intermediate Diaphragm

Six states use an X-brace option with a bottom horizontal strut; all 6 states use a single clip to connect the diagonals and horizontal to the girder face. Figure 4.4 shows the layout of this special X-brace diaphragm between girders. The anchorage of the clip angle into/through the girder (Figure 4.1, Options 1 and 2) varies between states.

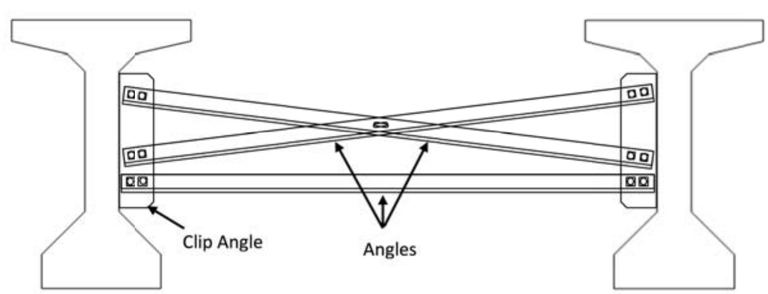

Figure 4.4. Steel X-Brace with Bottom Strut

The final type of steel intermediate diaphragm is the K-brace – made of a horizontal strut joined to two diagonals by welding all angles to a steel gusset plate. Figure 4.5 shows the layout of a K-brace with the horizontal strut placed at the top of the diaphragm system. Two states use this layout, while one state places the horizontal strut at the bottom of the system. All three states using this K-brace configuration specify a single clip angle for the connection of angles to the girder face.

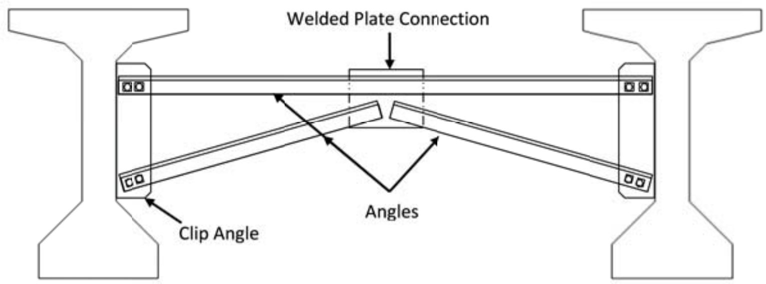

Figure 4.5. Steel K-Brace with Horizontal Oriented at Top

The four general layouts of steel intermediate diaphragms are complicated further when member dimensions are involved because the sizes of angles, bent plates, and channels used in each design vary widely between states. Some states even have multiple designs within the same category, depending on the size of the girders. Section 4.2 presents the steel intermediate diaphragm design specifics by state based girder shape and size.

4.2 Diaphragm Type Specified per Beam Type

In Chapter 3, Table 3.1 presented the various types of intermediate diaphragm combinations specified by each state to be used for precast prestressed girder bridges. Of the 21 states that specify or allow steel intermediate diaphragms, 14 use multiple configurations (Channel/Bent Plate/I-Beam, X-Brace, or K-Brace); the intermediate diaphragm configuration required is dependent on the shape and size of the prestressed girders specified in the design. In general, X-braces or K-Braces are preferred over the channel-type intermediate diaphragms for taller girders and bulb tee shapes. Table 4.1 provides details of the steel intermediate diaphragm required in each state based on the shape and size of the girder.

Table 4.1. Steel Intermediate Diaphragm Requirements Based on the Type/Size of Prestressed Girders

State	Prestressed Girder Type/Size	Configuration	Steel Section
Arkansas	AASHTO Girders ≤ 54" (Type IV)	Channel	C 15 x 33.9
	All Bulb Tees	X-Brace	L 6 x 4 x ½
Colorado	All Bulb Tees	I-Beam	W 16 x 26
Illinois	36" and 42" I-Beams	Channel	C 12 x 25 or C 12 x 30
	48" and 54" I-Beams All Bulb Tees	X-Brace	L 3½ x 3½ x ⅜
Indiana	AASHTO Type II	Channel	C 12 x 20.7
	AASHTO Type III-IV 54" Indiana Bulb Tee	Channel	MC 18 x 42.7
	≥ 60" Indiana Bulb Tee	X-Brace	L 6 x 4 x ½
Iowa	Iowa Bulb Tee "B" (36") - "D" (54")	Channel	C 15 x 33.9
	Iowa Bulb Tee "E" (63")	Bent Plate	36 x ½"
	Note: for bridges over roadway, the exterior bays have different intermediate diaphragms (W-sections) for impact protection		
Kentucky	AASHTO Type II	Channel	C 12 x 20.7
	AASHTO Type III - IV	Channel	MC 18 x 42.7
	AASHTO Type V	X-Brace	L 6 x 4 x ½
Michigan	Type I	Channel	C 10 x 15.3
	Type II	Channel	C 12 x 20.7
	Type III-IV	Channel	MC 18 x 42.7 or equiv. ⅜" Bent PL
	70" I-Girder and MI 1800 (similar)	X-Brace + Strut	Diag. L 6 x 5 x 5⁄16 Horiz. WT 6 x 13.0
Minnesota	36M and MN45 (36"-45")	Channel	C 12 x 20.7
	45M, 54M, MN54, and MN63 (45-63")	Channel	MC 18 x 42.7
	All MW (82"-96")	X-Brace + Strut	L 6 x 6 x ½ (all)
Missouri	MoDOT Type 2 - 6 (32" - 54") NU 53	Channel	C 15 x 33.9
	MoDOT Type 7 - 8 NU 63 and 70	Bent Plate	44 x 5/16"

(Table 4.1 Continued)

State	Prestressed Girder Type/Size	Configuration	Steel Section
New Jersey	45" I-Beam/Bulb Tee	Channel	C 12 x 20.7
	54" I-Beam/Bulb Tee	Channel	MC 18 x 42.7
	63" and 72" I-Beam/Bulb Tee	X-Brace + Strut	L 6 x 6 x ⅜ (all)
New Mexico	Type 36 - Type 45	Bent Plate	22 x ⅜"
	Type 54	Bent Plate	28 x ⅜"
	Type 63 - Type 72 BT-54 - BT-72 Type 63 MOD - Type 72 MOD	Bent Plate	40 x ⅜"
New York	AASHTO Type I - Type II, PCEF 39 - 47	I-Beam	W 14 x 99 (min.)
	AASHTO Type III - Type IV, PCEF 55	I-Beam	W 18 x 42.7 (min.)
	AASHTO Type V - VI, PCEF 63-79	K-Brace	L 3 x 3 x ⅜ (all)
North Carolina	Type II - Type III	Channel	MC 12 x 31
	Type IV	Channel	MC 18 x 42.7
	Bulb Tee	K-Brace	L 3 x 3 x 5/16 (all)
Ohio	60" - 72"	X-Brace + Strut	Diag. L 6 x 4 x 5⁄16Horiz. L 6 x 6 x ⅜
South Carolina	Type I Mod.	Channel	C 10 x 20
	Type II	Channel	C 12 x 25
	Type III - IV 54" Mod. Bulb Tee	Channel	MC 18 x 42.7
	63" - 78" Mod. Bulb Tee	X-Brace + Strut	L 6 x 3½ x 5⁄16 (all)
South Dakota	< 63"	Channel	Standard Details N/A
	≥ 63	X-Brace	Standard Details N/A
Tennessee	All Bulb Tee and I-Beams	X-Brace	L 6 x 4 x ½
Utah	42" - 58"	Channel	MC 18 x 42.7
	66" - 98"	K-Brace	Diag. L 3 x 3 x ⅜ (min) Horiz. L 5 x 5 x ⅜ (min)
Virginia	PCBT 29	Channel	MC 8 x 22.8
	PCBT 37	Channel	C 12 x 20.7
	PCBT 45 and 53	Channel	C 15 x 33.9
	61" - 93"	X-Brace	L 6 x 4 x ½
Wisconsin	28"	Channel	C 10 x 15.3
	36", 36W", 45W"	Channel	C 12 x 20.7
	45", 54", 54W"	Channel	MC 18 x 42.7 or equiv. ⅜" Bent PL
	70", 72W", 82W"	X-Brace + Strut	Diag. L 6 x 4 x 5⁄16 Horiz. WT 6 x 13.0
Wyoming	All	Channel	C 12 x 20.7

Table 4.1 details the different intermediate diaphragm configurations required for the type and size of the prestressed girders. Most states (14 of the 21 states designing steel intermediate diaphragms) require a change from channels/bent plates/I-beams to an X-brace or K-brace for larger girders. The specific girder size for this transition varies between states, but in general the transition to cross bracing schemes is made for girders deeper than 54-63 inches. Table 4.1 also provides the typical steel sections specified per girder size. The most common steel sections used are the C 12 x 20.7 (8 states) and MC 18 x 42.7 (9 states). Bent plate dimensions and angle sections used for cross bracing schemes vary widely between states and there is not a common section.

4.3 Alignment of Diaphragms in Skewed Bridges

The next major variation in steel intermediate diaphragm designs concerns the alignment of diaphragms in a bridge that has a skewed alignment. Skew is defined as the smaller angle between a line normal to the support centerline and the longitudinal centerline of the girders; skew varies from 0° to 90°. When intermediate diaphragms are placed within a span, they are arranged in a continuous line or staggered. For 0° skew, the intermediate diaphragms are aligned in a continuous line, as illustrated in Figure 4.6.

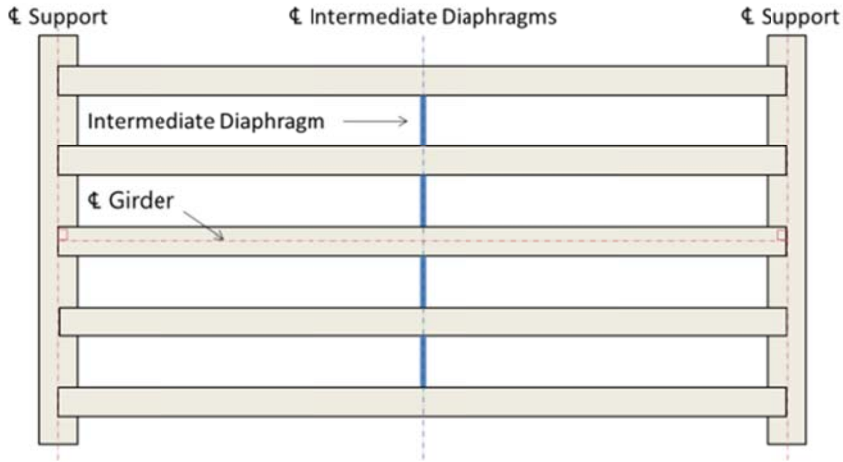

Figure 4.6. Continuous Line of Intermediate Diaphragms, 0° Skew

For skewed alignments, intermeidate diaphragms are inserted in a continuous line (Figure 4.7) or staggered (Figure 4.8). Intermediate diaphragms are placed in a continuous line for skewed girders, as illustrated in Figure 4.7, by creating connection angles to match the desired angle between the girder and intermediate diaphragm. When intermediate diaphragms are staggered, as illsutrated in Figure 4.8, the diaphragms are placed normal to the girders and the diaphragms in adjacent bays are offset according to the skew angle.

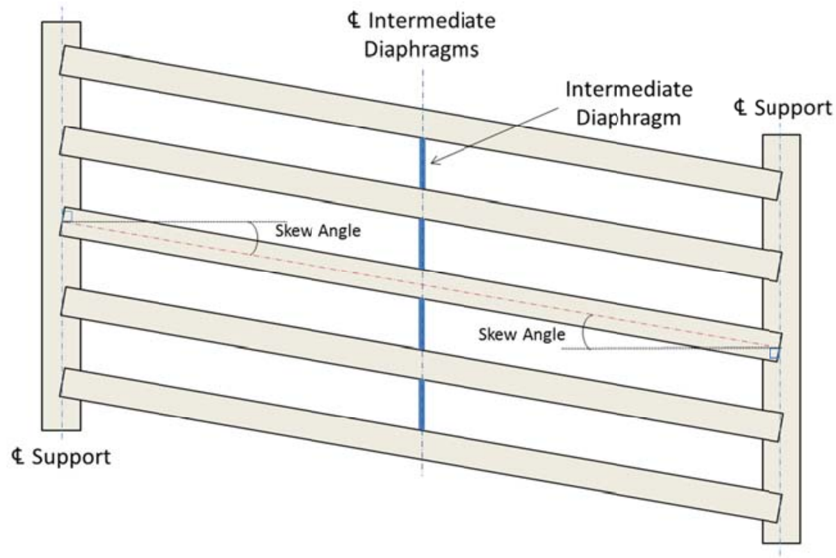

Figure 4.7. Continuous Line of Intermediate Diaphragms, Skewed Girders

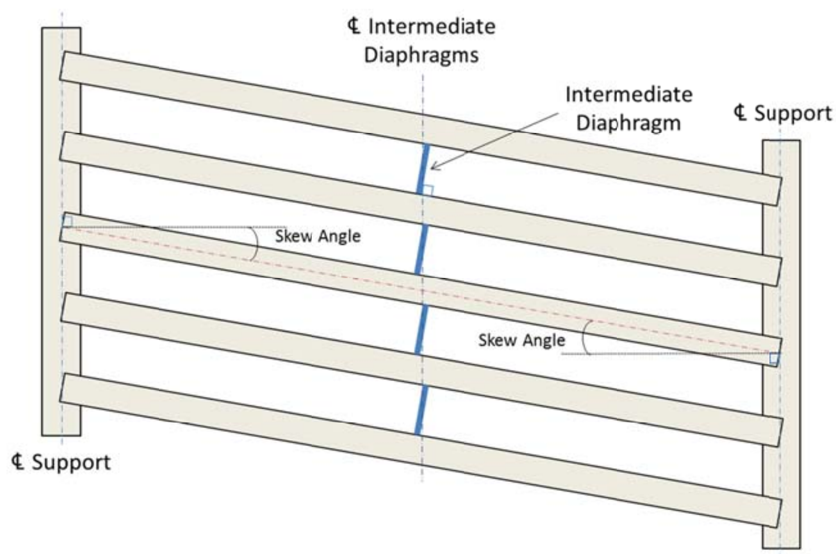

Figure 4.8. Staggered Line of Intermediate Diaphragms, Skewed Girders

The transition from continuous to staggered alignments to account for skew varies between states. Table 4.2 and Figure 4.9 provide details about the relationship between skew angle and diaphragm alignment for each state.

Table 4.2. Alignment of Steel Intermediate Diaphragms Based on Skew Angle

State	Skew	Alignment
Arkansas	0° > 0°	Continuous Line Staggered
Colorado	≤ 10° > 10°	Continuous Line Staggered
Illinois	≤ 20° > 20°	Continous Line Staggered
Indiana	0° > 0°	Continuous Line Staggered
Iowa	≤ 7°30′ > 7°30′	Continuous Line Staggered
Kentucky	0° > 0°	Continuous Line Staggered
Michigan	≤ 10° > 10°	Continuous Line Staggered
Minnesota	≤ 20° > 20°	Continuous Line Staggered
Missouri	≤ 20° > 20°	Continuous Line Staggered
New Jersey	≤ 10° > 10°	Continuous Line Staggered
New Mexico	0° > 0°	Continuous Line Staggered*

State	Skew	Alignment
New York	≤ 20° > 20°	Continuous Line Staggered
North Carolina	≤ 20° > 20°	Continuous Line Staggered
Ohio	≤ 10° > 10°	Continuous Line Staggered
South Carolina	≤ 20° > 20°	Continuous Line Staggered
South Dakota	Standard Details N/A	
Tennessee	≤ 15° > 15°	Continuous Line Staggered
Utah	All	Continuous Line
Virginia	≤ 20° > 20°	Continuous Line Staggered
Wisconsin	≤ 10° > 10°	Continuous Line Staggered
Wyoming	≤ 20° > 20°	Continuous Line Staggered

* For skew 0-8° there is a modified staggered alignment, but it is similar to Figure 4.8

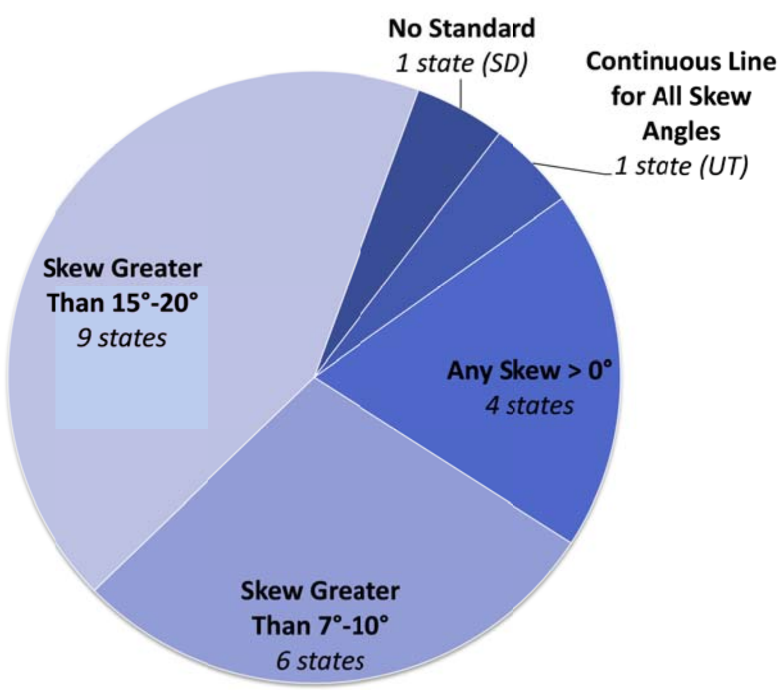

Figure 4.9. Transition from Continuous to Staggered Alignment

Table 4.2 lists the alignment requirements in each state based on the skew angle of the bridge. Figure 4.9 describes the trends in intermediate diaphragm alignment by grouping states with similar requirements. For bridges that have a 0° skew, it is typical for the intermediate diaphragms to be aligned in a continuous line, perpendicular to the girders. Bridges skewed between 0-20° have varying alignments of intermediate diaphragms based on state requirements and the severity of the skew. One state (Utah) uses a continuous alignment regardless of skew angle, using connection angles made specifically to match the skew of the girders. Nine states transition from continuous alignments to staggered alignments between 15-20°. Six states make the transition in alignment from continuous to staggered between 7-10°. Four states used staggered alignments for any skew greater than 0°.

4.4 Spacing of Diaphragms within Span

The next important characteristic of intermediate diaphragm is the spacing of intermediate diaphragms within spans. Table 4.3 summarizes the spacing requirements by state for steel intermediate diaphragms within a span, based on the span length.

Table 4.3. Summary of Locations of Steel Intermediate Diaphragms

State	Span Length	Location
Arkansas	All	Midspan
Colorado	All	Midspan *
Illinois	≤ 90’ > 90’	$^{1}/_{3}$ Points $^{1}/_{4}$ Points
Indiana	< 80’ 80’ – 120’ > 120’	None Midspan $^{1}/_{3}$ Points

(Table 4.3 Continued)

State	Span Length	Location
Iowa	< 125′ 125′ – 135′	Midspan 20′ Each Side of Midspan
Kentucky	< 40′ 40′ – 80′ > 80′	None Midspan $^{1}/_{4}$ Points
State	**Span Length**	**Location**
Michigan	All	Midspan
Minnesota	< 45′ 45′ – 90′ > 90′	None Midspan $^{1}/_{3}$ Points
Missouri	≤ 90′ > 90′	Midspan $^{1}/_{3}$ Points **
New Jersey	≤ 80′ > 80′	Midspan $^{1}/_{3}$ Points
New Mexico	≤ 100′ > 100′	Midspan $^{1}/_{3}$ Points
New York	< 65′ 65′ – 100′ > 100′	None Midspan $^{1}/_{3}$ Points
North Carolina	< 40′ 40′ – 100′ > 100′	None Midspan $^{1}/_{3}$ Points
Ohio	≤ 80′ > 80′	Midspan $^{1}/_{4}$ Points
South Carolina	≤ 40′ > 40′	None Midspan *
South Dakota	All	Midspan
Tennessee	< 40′ 40′ – 80′ > 80′	None Midspan $^{1}/_{3}$ Points
Utah	< 80′ 80′ – 120′ 120′ – 160′ > 160′	Midspan $^{1}/_{3}$ points $^{1}/_{4}$ points $^{1}/_{5}$ points
Virginia	≤ 40′ 40′ – 80′ ≥ 80′	None Midspan Equally spaced w/ max. spacing of 40′
Wisconsin	≤ 80′ > 80′	Midspan $^{1}/_{3}$ points
Wyoming	≤ 40′ 40′ – 80′ ≥ 80′	None Midspan $^{1}/_{3}$ points
* Minimum requirement ** Maximum spacing of 50′ allowed		

The standards for locations of steel intermediate diaphragms within a span vary widely between states. Some states have a minimum span length for which it has been determined an intermediate diaphragm is not required, other states specify at least one intermediate diaphragm at midspan regardless of span length. These requirements are determined at the bridge design agency's discretion, just as is the type of intermediate diaphragm used. There is no national standard or minimum requirement for either element of the design.

4.5 Summary of Current Steel Intermediate Diaphragm Design Practices

The design of steel intermediate diaphragms for precast girder bridges throughout the U.S. varies widely. Channels/bent plates/I-beams are specified for intermediate diaphragms in 19 states, K-brace diaphragms are specified in 3 states, and X-brace diaphragms are specified in 13 states. The diaphragm-to-girder connection schemes vary between state designs. Of the 21 states that specify or allow steel intermediate diaphragms, 14 use multiple configurations (Channel/Bent Plate/I-Beam, X-Brace, or K-Brace); the intermediate diaphragm configuration required is dependent on the shape and size of the prestressed girders specified in the design. Most commonly, channel-type diaphragms are specified for shallow girders and cross bracing schemes (X-brace or K-brace) are required for deeper girders. The specific girder size for this transition varies between states, but in general the transition to cross bracing schemes is made for girders deeper than 54-63 inches. The alignment of intermediate diaphragms (continuous/staggered) based on skew angle also varies between states.

Finally, the relationship between span length and intermediate diaphragm spacing also varies throughout the U.S. In the absence of a national design standard for intermediate diaphragms, requirements are set at the independent discretion of the state departments of transportation, leading to wide variations across the U.S.

Chapter 5: Temporary Diaphragms for Erection and Construction

The benefits of intermediate diaphragms in prestressed girder bridges after the deck has cured are debated, as discussed in Chapter 2. However, the bracing of girders during erection and construction before the deck strengthens is widely considered necessary. Based on the survey of intermediate diaphragm practices discussed in Chapter 3, it was determined that four states – Alabama, Florida, Kansas, and Texas – specify temporary bracing during construction instead of permanent intermediate diaphragms. All four states have unique temporary bracing standards. This chapter will discuss the details of temporary bracing methods in each state individually. Section 5.1 will discuss how the Alabama Department of Transportation (ALDOT) addresses the need for temporary bracing of girders during construction. Section 5.2 will discuss the temporary bracing standards specified by the Florida Department of Transportation (FDOT) and the FDOT *Beam Stability* Mathcad Program (2013). Section 5.3 will describe the temporary bracing details used by the Kansas Department of Transportation (KDOT). Section 5.4 will describe the temporary bracing details specified by the Texas Department of Transportation (TxDOT).

5.1 Alabama Department of Transportation (ALDOT)

Prior to August 2013, ALDOT specified cast-in-place reinforced concrete intermediate diaphragms in prestressed concrete girder bridges. However, the standards were revised August 1, 2013 to no longer specify the use of permanent intermediate diaphragms for such bridges. Cast-in-place end diaphragms – called “edge beams” at interior joints and “end walls” at the abutments – are still specified at the girder ends. ALDOT does not consider intermediate diaphragms to be necessary to the structure in its service life. Contractors are now responsible for ensuring girder stability during erection and construction and assessing the need for temporary bracing. ALDOT has not published standard details for temporary bracing, and there are no published guidelines for evaluating the adequacy of a contractor’s bracing design. The goal of this chapter is to highlight the temporary bracing means and methods used by FDOT, KDOT, and TxDOT as suggestions for the future development of temporary bracing guidelines by ALDOT.

5.2 Florida Department of Transportation (FDOT)

In Florida, the bridge designer is responsible for calculating the required bracing, but the contractor is responsible for designing the steel bracing members and connections to resist those loads. FDOT reviews contractor bracing plans in cases where the construction affects public safety. Typically these temporary bracing systems are used during construction then removed.

According the FDOT *Instructions for Design Standards* (FDOT 2014b), bridge designers are required to provide the wind load variables and assumed construction loads, calculate the design loads for the temporary bracing, and submit those sets of values in three tables, shown in Table 5.1. These values can be calculated with the aid of the FDOT *Beam Stability* Mathcad Program (2013). The forces and values submitted by the bridge designer are used by contractors to design bracing members and connections.

Table 5.1. Prestressed Beam Temporary Bracing Data Tables (FDOT 2014b)

(A) TABLE OF WIND LOAD VARIABLES	
WIND SPEED, BASIC (MPH)	
WIND SPEED, CONSTRUCTION INACTIVE (MPH)	
WIND SPEED, CONSTRUCTION ACTIVE (MPH)	
VELOCITY PRESSURE EXPOSURE COEFFICIENT	
GUST EFFECT FACTOR	

(B) TABLE OF ASSUMED CONSTRUCTION LOADS (UNFACTORED)	
BUILD-UP (PLF)	
FORM WEIGHT (PSF)	
FINISHING MACHINE TOTAL WEIGHT (KIPS)	
FINISHING MACHINE WHEEL LOCATION BEYOND EDGE OF DECK OVERHANG (INCHES)	
DECK WEIGHT (PSF)	
LIVE LOAD (PSF)	
LIVE LOAD AT EXTREME DECK EDGE (PLF)	

(C) TABLE OF TEMPORARY BRACING VARIABLES							
SPAN NO.	L_B, MAXIMUM UNBRACED LENGTH (FT)	HORIZONTAL FORCE AT EACH BEAM END AND ANCHOR BRACE (KIP)	HORIZONTAL FORCE AT EACH INTERMEDIATE SPAN BRACE (KIP)	OVERTURNING FORCE AT EACH BEAM END AND ANCHOR BRACE (KIPXFT)	OVERTURNING FORCE AT EACH INTERMEDIATE SPAN BRACE (KIPXFT)	BRACE ENDS PRIOR TO CRANE RELEASE (YES/NO)	TOTAL LINES OF BRACING

According to the FDOT *Instructions for Design Standards* (IDS) (FDOT 2014b), these tables (Table 5.1) are required to be included in the plans for all bridges with prestressed concrete I-beam superstructures. Requirements for the variables and assumptions needed for Table 5.1(a) and Table 5.1(b) are discussed in the FDOT *Structures Design Guidelines* (SDG) (FDOT 2014c). Permitted values for the wind load variables and assumed construction loads are listed in Table 5.2.

Table 5.2. Permitted Load Assumptions

(A) TABLE OF WIND LOAD VARIABLES	
WIND SPEED, BASIC (MPH)	110-150 MPH, DEPENDENT ON COUNTY....................... *FDOT SDG (2014) TABLE 2.4.1-2*
WIND SPEED, CONSTRUCTION INACTIVE (MPH)	60% OF BASIC WIND SPEED .... *FDOT SDG (2014) SECTION 2.4.3*
WIND SPEED, CONSTRUCTION ACTIVE (MPH)	20 MPH OR EXPECTED WIND SPEED, IF HIGHER .... *FDOT SDG (2014) SECTION 2.4.3*
VELOCITY PRESSURE EXPOSURE COEFFICIENT	CALCULATED BASED ON HEIGHT (EQN. 2-6) .... *FDOT SDG (2014) EQN. 2-2*
GUST EFFECT FACTOR	0.85 FOR BRIDGE W/ SPANS < 250FT AND HEIGHT < 75FT; OTHERWISE, EVALUATE ACCORDING TO ASCE/SEI 7-05 SECTION 6.5.8 .... *FDOT SDG (2014) SECTION 2.4.1 D*

(B) TABLE OF ASSUMED CONSTRUCTION LOADS (UNFACTORED)	
BUILD-UP (PLF)	50 PLF .... *FDOT IDS (2014)*
FORM WEIGHT (PSF)	20 PSF RECOMMENDED ....*FDT SDG (2014) TABLE 2.2-1*
FINISHING MACHINE TOTAL WEIGHT (KIPS)	6.4-16 KIPS, DEPENDENT ON BRIDGE WIDTH .... *FDOT SDG (2014) 2.13.1 A*
FINISHING MACHINE WHEEL LOCATION BEYOND EDGE OF DECK OVERHANG (INCHES)	2.5 INCHES .... *FDOT BEAM STABILITY (2014)*
DECK WEIGHT (PSF)	CALCULATE BASED ON DECK DIMENSIONS
LIVE LOAD (PSF)	20 PSF .... *FDOT SDG (2014) 2.13.1 B*
LIVE LOAD AT EXTREME DECK EDGE (PLF)	75 PLF .... *FDOT SDG (2014) 2.13.1 D*

The data in Table 5.1(a) and Table 5.1(b), based on the permitted assumptions in Table 5.2, are used in the calculations required to obtain the values needed for Table 5.1(c) – "The Table of Temporary Bracing Variables." These values are easily calculated using the FDOT *Beam Stability* Mathcad Program (2013). An annotated example using the latest program version – version 2.1 (2013) – is included in Appendix A. Figure 5.1 outlines the procedure used by the FDOT *Beam Stability* Mathcad Program (2013) to calculate the temporary bracing variables required to be submitted by the bridge designer.

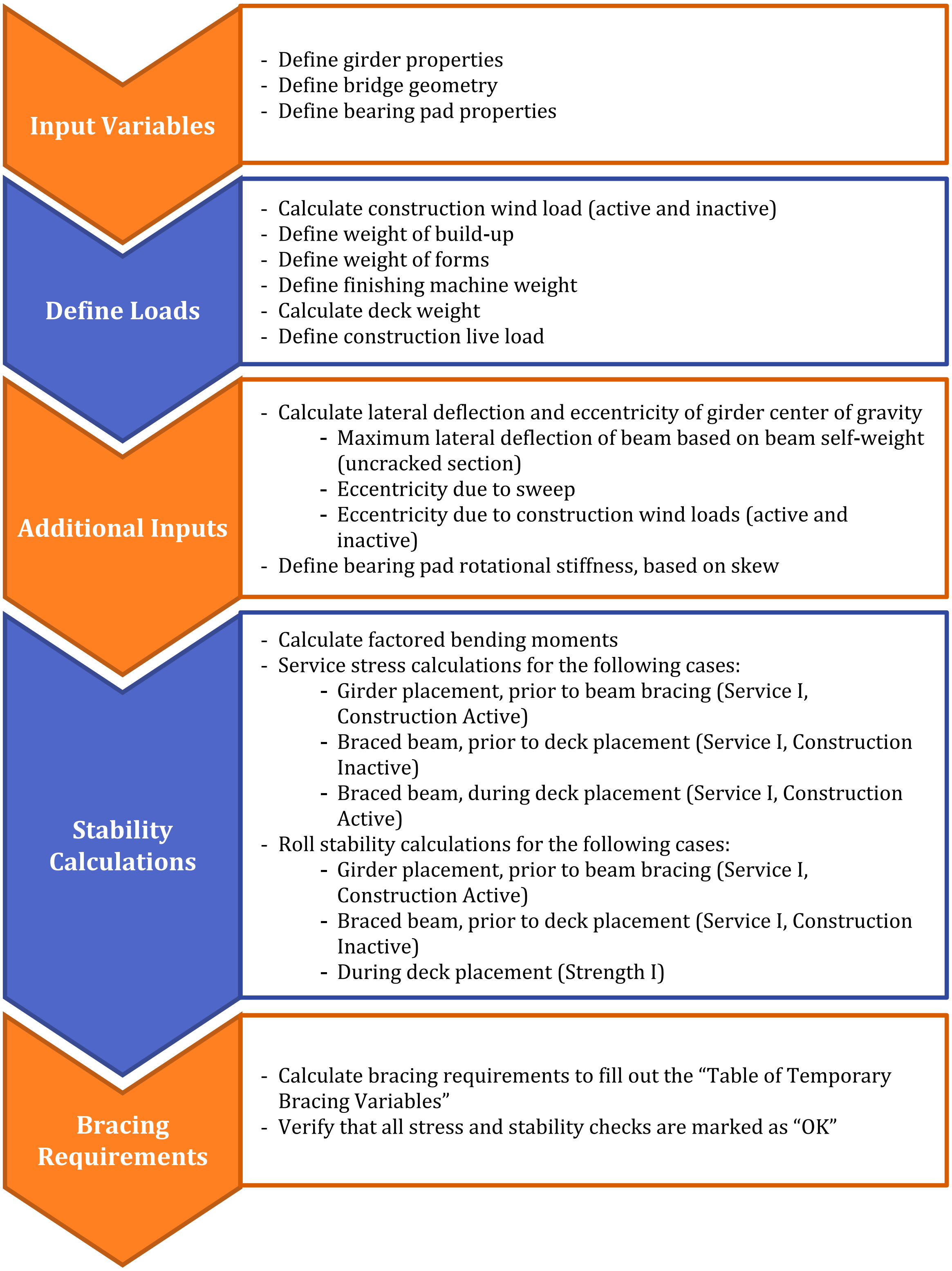

Figure 5.1. Outline of FDOT *Beam Stability* Mathcad Program

The outputs of the FDOT *Beam Stability* Mathcad Program (2013) are used to complete Table 5.1(c) - “The Table of Temporary Bracing Variables.” It is the bridge designer’s responsibility to calculate these values, but the contractor is responsible for designing adequate bracing members and connections based on the values submitted in Table 5.1. If the actual construction loads exceed the values assumed by the engineer in Table 5.1(b), the contractor is required to re-calculate the bracing requirements.

In addition to the standard tables shown in Table 5.1(a) through Table 5.1(c), the following “Beam Temporary Bracing Notes” (Figure 5.2) are required to be included in the plans. The notes to the contractor in Figure 5.2 clarify the responsibilities of the contractor to design temporary bracing adequate for the design values in Table 5.1(a) through Table 5.1(c).

BEAM TEMPORARY BRACING NOTES:

Based on investigation of the beam stability, temporary bracing as shown in the 'TABLE OF TEMPORARY BRACING VARIABLES' and Design Standard Index No. 20005 is required. The Table and following information is provided to aid the Contractor in design of beam temporary bracing:

1. *Design the bracing members and connections to transfer both compressive and tensile forces equal to the horizontal forces given in the 'TABLE OF TEMPORARY BRACING VARIABLES'. Also design bracing members and connections to be capable of resisting the overturning forces given in the Table, non-simultaneously with horizontal forces. Assume that horizontal bracing forces are applied perpendicular to the beam web at mid-height of the beam, and assume that overturning bracing forces are applied at the centerline of the beam at the top of the top flange.*
2. *The horizontal brace forces have been determined by application of the Construction Inactive Wind Load as listed in the 'TABLE OF WIND LOAD VARIABLES'. The overturning brace forces have been determined by application of the Construction Active Wind Load as listed in the 'TABLE OF WIND LOAD VARIABLES' plus the assumed construction loads shown in the 'TABLE OF ASSUMED CONSTRUCTION LOADS'. It is the Contractor's responsibility to re-calculate the bracing requirements if the actual construction loads exceed the assumed loads shown, or if the finishing machine wheel location from the edge of the deck overhang exceeds the value listed.*
3. *The temporary bracing at the ends of the beams shall be installed prior to crane release if indicated in the 'TABLE OF TEMPORARY BRACING VARIABLES'. Beams shall not be left un-braced during non-work hours. Bracing shall remain in place until bridge deck concrete reaches 2500 psi.*
4. *The exposure period (defined as the time period for which temporary load cases of the superstructure exist) is assumed to be less than one year. Horizontal bracing forces, as specified in the 'TABLE OF TEMPORARY BRACING VARIABLES', are not valid if the exposure period is more than one year; for this case the Contractor shall re-calculate bracing requirements.*
5. *Horizontal and overturning forces are factored per the Strength III limit state for construction.*

Figure 5.2. Required Beam Temporary Bracing Notes (FDOT 2014b)

In addition to the “Prestressed Beam Temporary Bracing Data Tables” (Table 5.1) and the “Beam Temporary Bracing Notes” (Figure 5.2), FDOT also specifies standard details for temporary bracing in the *Design Standards eBooklet* (FDOT 2014a). Figure 5.3

summarizes the allowable temporary steel diaphragm configurations for both end and intermediate locations. These details include permitted layouts and alignment but do not specify member sizes or connection details for the bracing. Those specific details are required to be determined by the contractor based on the load requirements specified by the bridge designer.

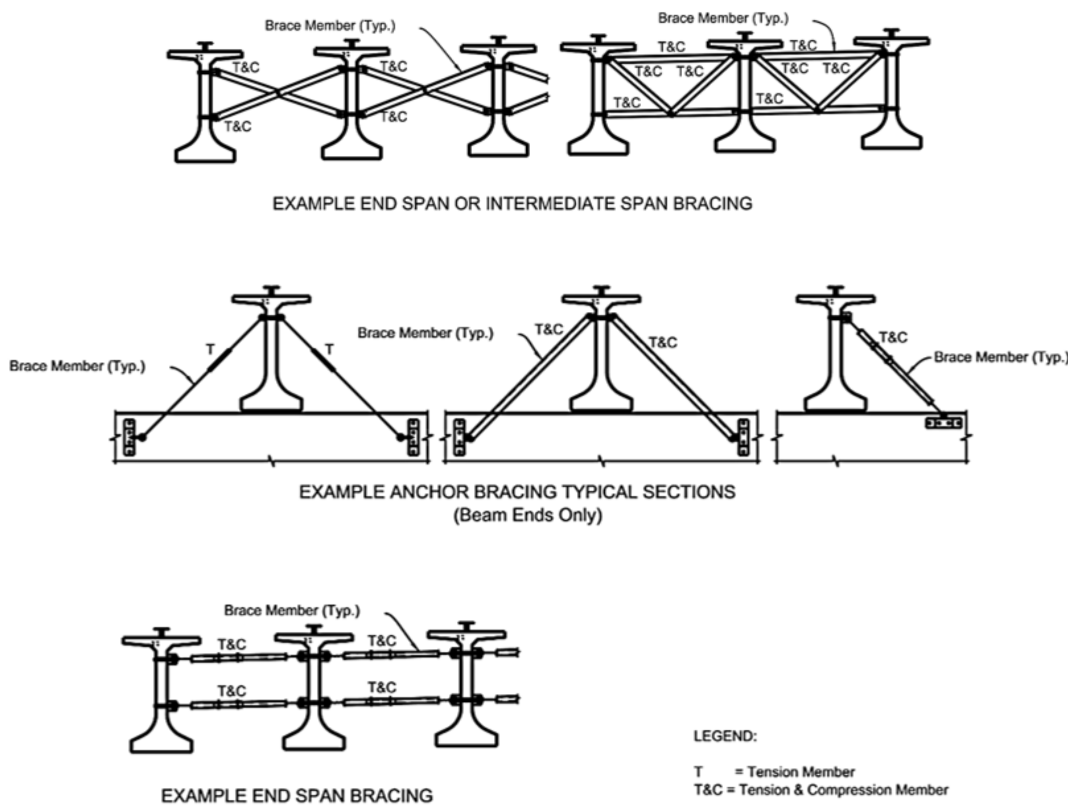

Figure 5.3. Permitted Temporary Bracing Configurations (FDOT 2014a)

The first girder erected in the span requires anchor bracing at both support locations. All subsequent beams are braced against the “anchor beam” sequentially using any of the end span bracing configurations permitted. If intermediate bracing is required according to “The Table of Temporary Bracing Variables” submitted by the bridge designer, FDOT permits the use of temporary steel cross bracing at intermediate points within the span, either with an X-brace or a K-brace with top horizontal strut. All bracing – end or intermediate – is placed perpendicular to the girders for all possible skew angles.

5.3 Kansas Department of Transportation (KDOT)

The standards for diaphragms defined by KDOT are outlined in Section 3.5.2.10 of the KDOT *LRFD Design Manual* (KDOT 2012). Cast-in-place permanent diaphragms are required at all supports. KDOT specifies steel temporary diaphragms in the form of bent plates for typical prestressed girder bridges. For heavily skewed bridges, cast-in-place intermediate diaphragms are permitted to be used instead of steel bent plates. Shop drawings of temporary diaphragms must be submitted by the contractor to the KDOT Bridge Section for review and approval. Temporary diaphragms are required to be placed prior to any superstructure concrete and are to be left in place until the concrete end diaphragms and concrete deck have cured. The steel diaphragms are not permitted to be left in the structure; they are property of the contractor and must be removed from the site at the completion of the project.

The standard details for temporary intermediate diaphragms used in Kansas are shown in Figure 5.4. KDOT specifies the use of a bent 5/16-inch thick steel plate, bent according to “Section A” in Figure 5.4. The details show connections using bolts through the girder, which is preferred, but threaded insert connections are permitted for heavily skewed girders. Temporary intermediate diaphragms are placed perpendicular to the girders, regardless of skew angle, and typically they are placed in all bays. However, in bridges with an even number of beams, KDOT permits diaphragms to be placed in the outer bays and every other interior bay, connecting the beams in pairs.

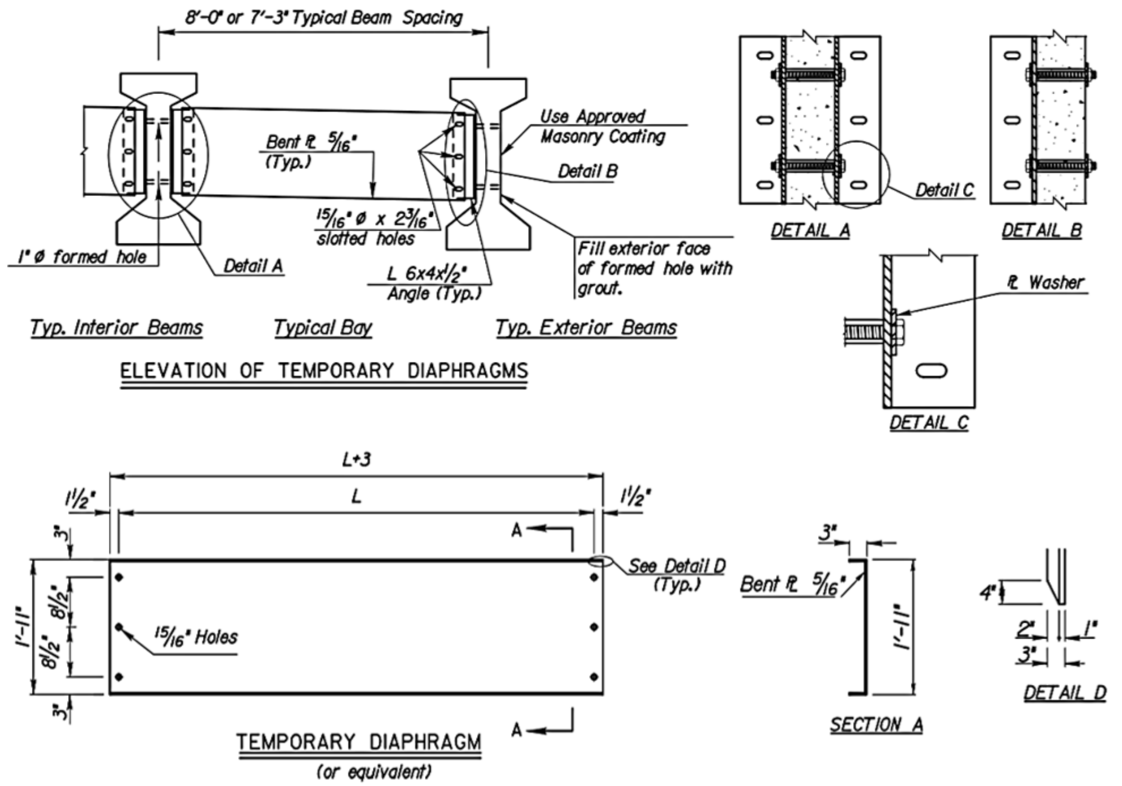

Figure 5.4. KDOT Temporary Intermediate Diaphragm Detail (KDOT 2012)

The spacing of temporary intermediate diaphragms is defined by KDOT based on the length of the span being braced. For spans less than 40 feet long, temporary steel diaphragms are not required. For spans ranging from 40 to 80 feet in length, two temporary steel diaphragms are required in the span, located at third-points. Spans greater than 80 feet in length but less than 120 feet require three temporary steel diaphragms, located at quarter-points. Spans equal to, or greater than, 120 feet in length require special intermediate bracing, to be determined by KDOT.

5.4 Texas Department of Transportation (TxDOT)

The Texas Department of Transportation (TxDOT) has standards for erection and bracing requirements that are detailed in the TxDOT *Bridge (English) Standards* (2014). The bracing specified by TxDOT is required to be placed immediately after the erection of each girder. The first girder in each sequence is braced with a diagonal brace, which combines a diagonal timber bracing member with a wire rope. Figure 5.5 shows the portions of the standard that illustrate the diagonal bracing elements. The text pictured in Figure 5.6 contains the annotations associated with the drawings in Figure 5.5.

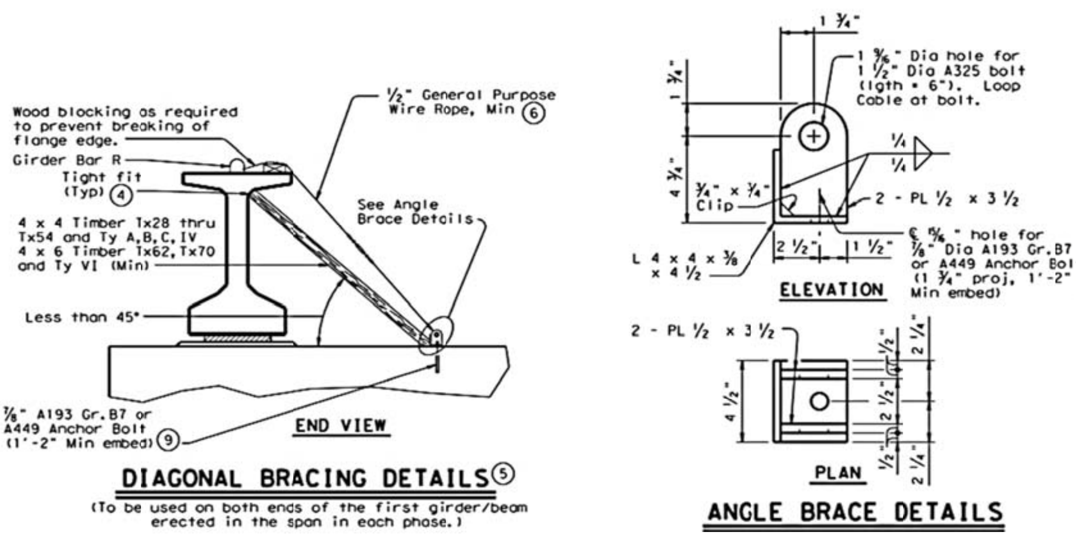

Figure 5.5. TxDOT Diagonal Anchor Bracing Standard (TxDOT 2014)

(2) Place and weld #5 bars as shown during erection. If forming deck with prestressed panels, bars can be temporarily removed, one at a time, during panel erection. Re-install bar prior to additional panel erection. Bars can rest on panels and be bent down and welded to Girder Bars R (See Sheet 2 of 2).

(3) Clear distance between spacers must not exceed 3'. Nail together with 16d nails.

(4) Use wedges as necessary to obtain tight fit. Nail wedges to timbers.

(5) Pressure treated landscape timbers can not be used.

(6) All hardware used with cable must be able to develop a minimum 25 kips breaking strength. Use thimbles at all loops in cable. Install cable clamps with saddles bearing against the live end and U-bolts bearing aginst the dead end.

(7) It is acceptable to tie anchor bolts to cap reinforcement.

(8) Prior to installing, field bend strap to lay flush on both girders' top flange and slope between flange tips.

(9) Anchor bolt may be drilled and epoxied in place. Provide 25k minimum pullout. Core drill hole.

Figure 5.6. Notes for Temporary Bracing Standards (TxDOT 2014)

After the first girder in the sequence is braced using the diagonal anchor bracing, subsequent girders are braced using horizontal bracing that also combines permanent and temporary elements. Figure 5.7 shows the details relevant to the erection bracing in the span after the first girder is placed. This bracing is used at both end and intermediate points within the span. The top bracing – both rebar and strap – remains in place, while the bottom bracing – timber struts – must be removed. The notes in Figure 5.6 also relate to the annotations in Figure 5.7.

There are two horizontal bracing options, labeled in Figure 5.7. The difference between the details is only the steel top bracing; Option 1 requires a steel strap and Option 2 requires standard rebar. Option 1 is only allowed when the slab is formed using precast deck panels; it is not permitted when the slab is formed using permanent metal deck forms or plywood. When Option 1 is used and the slab is formed with precast panels, the panels are placed on top of the steel strap. Option 2 is permitted when the slab is formed with precast panels, permanent metal deck forms, or plywood.

When Option 2 is used and the slab is constructed using precast panels, the rebar is bent in the field to allow the panels to rest on the adjacent girder flanges, snug beneath the rebar. However, when Option 2 is used and the slab is constructed using permanent metal deck forms or plywood, the rebar is not bent for slab placement.

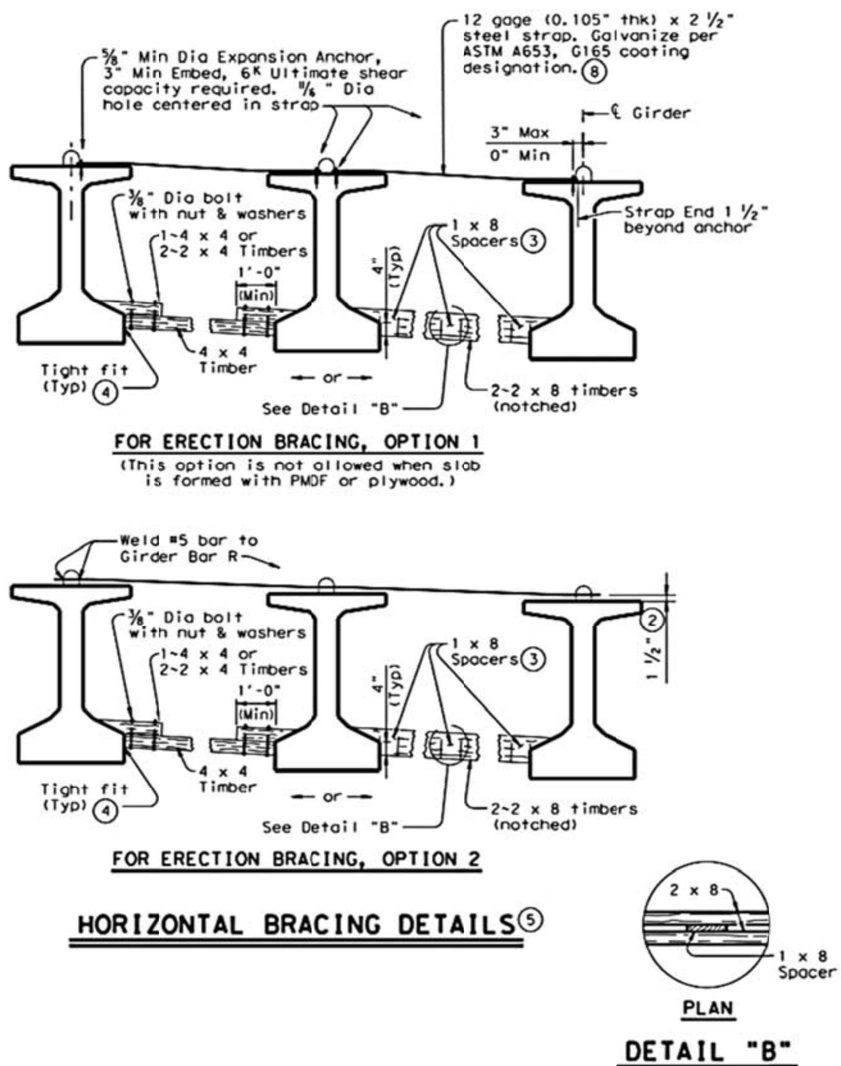

Figure 5.7. TxDOT Erection Bracing Standards (TxDOT 2014)

All bracing is placed perpendicular to the girders, regardless of skew angle. The spacing requirement for the horizontal bracing varies based on girder size, bracing option, and slab overhang. The first and last bracing in the span is placed 4’-0” inward from the support centerline at each end; the spacing of bracing between the first and last is measured inward from those brace locations. Table 5.3 summarizes the minimum bracing spacing requirements specified by TxDOT.

Table 5.3. Maximum Bracing Spacing Permitted by TxDOT (2014)

TxDOT I-Girder Depth	Option #1 (Steel Strap)		Option #2 (No. 5 Bar)	
	Slab Overhang* Less than 4’-0”	Slab Overhang* 4’-0” and Greater	Slab Overhang* Less than 4’-0”	Slab Overhang* 4’-0” and Greater
28”	¼ points	¼ points	¼ points	⅛ points
34”	¼ points	¼ points	¼ points	⅛ points
40”	¼ points	⅛ points	¼ points	⅛ points
46”	¼ points	⅛ points	¼ points	⅛ points
54”	¼ points	⅛ points	¼ points	⅛ points
62”	¼ points	⅛ points	¼ points	⅛ points
70”	¼ points	⅛ points	¼ points	⅛ points
** Slab overhang is measured from centerline of girder. If the overhang varies in the span, use the largest.*				

For Option 1 or Option 2 bracing, if the slab overhang is less than 4’-0”, bracing is required at quarter-points at a minimum. If the slab overhang is 4’-0” or greater, spacing at eighth-points is typical for both bracing options; a larger spacing – quarter-points – is permitted for 28” and 34” I-girders.

5.5 Summary of Temporary Intermediate Diaphragm Practices

Four states have transitioned from permanent intermediate diaphragms to temporary bracing schemes that remain in-place during erection and construction only.

However, the means and methods of bracing are unique for each state. The physical diaphragm configurations and materials vary, as well as spacing and how they are regulated by the state department of transportation. The temporary bracing schemes that have already been established by FDOT, KDOT, and TxDOT are a valuable starting point for states considering re-evaluating their practice of permanent intermediate diaphragms.

In Alabama, the contractor is solely responsible for ensuring the stability of girders during erection and construction procedures. There is no standard for temporary bracing published by ALDOT, and there is currently no specified protocol for checking the adequacy of a contractor's temporary bracing plans.

In Florida, the bridge designer is responsible for calculating the required bracing loads and indicating those loads on the plans, but the contractor is responsible for designing the steel bracing members and connections to resist those loads. The FDOT *Beam Stability* Mathcad Program (2013) was developed to aid designers in determining construction and wind loads on girders; the outputs of the program are used to provide contractors with the temporary bracing requirements. FDOT provides standard details that illustrate the permitted configurations for anchor, end, and intermediate temporary bracing of prestressed girders, but these schematics are merely to illustrate geometry and placement of bracing. The contractor is responsible for determining member sizes and connections that will provide adequate bracing for the girders. If the construction affects public safety, FDOT reviews contractor bracing plans.

In Kansas, KDOT provides detailed specifications, including member sizes and connection details, for temporary steel intermediate diaphragms that are to be used by contractors. The spacing of diaphragms is also dictated by the temporary diaphragm standard. Shop drawings of temporary diaphragms are required to be submitted by contractors to the KDOT Bridge Section for review and approval.

In Texas, TxDOT provides detailed standards for erection bracing that is a combination of timber and steel. Two bracing options are permitted, determined by the slab forming method – precast deck panels, permanent metal deck forms, or plywood. The standards include all member sizes, connection details, and spacing requirements.

Chapter 6: Conclusions and Recommendations

6.1 Summary

Diaphragms are placed transverse to bridge girders, connecting adjacent girders together to provide stability and transmit loads. They are commonly located at each end of the span and at intermediate points within the span. The construction of end diaphragms in precast girder bridges, bracing the ends of adjacent girders at the supports, is typical throughout the United States (U.S.), but the use of diaphragms at intermediate points varies widely among state bridge design agencies. The focus of this research is on the use of intermediate diaphragms in simple-span prestressed concrete girder bridges, specifically those with I-beams and Bulb-tees.

Prior to August 2013, the Alabama Department of Transportation (ALDOT) specified the use of cast-in-place, reinforced concrete intermediate diaphragms in bridges with precast concrete girders. When this research was initiated in January 2013, engineers at ALDOT were interested in the feasibility and performance of steel intermediate diaphragms following requests from contractors to use steel diaphragm alternates. However, in August 2013, ALDOT revised their existing standards, removing intermediate diaphragms from their specifications and placing the responsibility of ensuring the stability of girders during erection and deck construction onto the contractors. To provide a resource of alternatives to reinforced concrete intermediate diaphragms, this report profiled the practices of permanent steel diaphragms and temporary bracing options used throughout the U.S.

There has been much research to understand the performance and effectiveness of intermediate diaphragms during construction and in-service. Details on the state of research and important studies and conclusions are discussed in Chapter 2. It has been well established that intermediate diaphragms are essential immediately after girder erection and during construction operations to provide lateral stability to the girders. However, the necessity of intermediate diaphragms after the deck is constructed and during the service-life of the structure is not agreed upon. Several research efforts have involved surveys to profile trends in intermediate diaphragm practices between states. Those surveys showed wide variations in design practices and a trend toward steel intermediate diaphragm alternates in the last two decades. The lack of agreement on the effectiveness and purpose of intermediate diaphragms has led to a lack of cohesion in intermediate diaphragm design and implementation across the U.S.

To understand the current design trends and methodologies, a detailed survey of the current intermediate diaphragm design practices in the U.S. was completed. The methodology and results of the survey are detailed in Chapter 3. Table 6.1 briefly summarizes the results of the survey, based on the data from all 50 state design agencies. Many states permit multiple intermediate diaphragm types, for various reasons.

Table 6.1. Intermediate Diaphragms Specified Throughout U.S.

Type of Intermediate Diaphragm Specified	No. of States
Permanent Cast-In-Place Concrete	28 states
Permanent Steel Channel/Bent Plate/I-Beam	19 states
Permanent Steel K-Brace	3 states
Permanent Steel X-Brace	13 states
Temporary Bracing During Construction Only	4 states
No Standard for Intermediate Diaphragms	5 states

Chapter 4 discusses the various permanent steel intermediate diaphragms used through the U.S. Channels/bent plates/I-beams are specified for intermediate diaphragms in 19 states, K-brace diaphragms are specified in 3 states, and X-brace diaphragms are specified in 13 states. Of the 21 states that specify or allow steel intermediate diaphragms, 14 use multiple configurations (Channel/Bent Plate/I-Beam, X-Brace, or K-Brace); the intermediate diaphragm configuration required is dependent on the shape and size of the prestressed girders specified in the design. Most commonly, channel-type diaphragms are specified for shallow girders and cross bracing schemes (X-brace or K-brace) are required for deeper girders. The placement and installation of steel intermediate diaphragms also varies widely. Factors such as alignment with respect to the girder and skew angle, diaphragm-to-girder connections, and the spacing of intermediate diaphragms within a span, are not standard between states. In the absence of a national design standard for intermediate diaphragms, requirements are set at the independent discretion of the state departments of transportation.

Four states have transitioned from permanent intermediate diaphragms to temporary bracing schemes that remain in-place during erection and construction only. The means and methods of temporary bracing are unique for each of the four states. The physical diaphragm configurations and materials vary, as well as spacing and how they are regulated. The temporary bracing schemes that have already been established by the Florida Department of Transportation (FDOT), Kansas Department of Transportation (KDOT), and Texas Department of Transportation (TxDOT) are a valuable starting point for states re-evaluating their practice of using permanent intermediate diaphragms. Details from FDOT, KDOT, and TxDOT are included and discussed in Chapter 5.

Both steel intermediate diaphragm standards and temporary bracing standard vary widely throughout the U.S. The absence of a national standard and disagreements on the effectiveness and purpose of intermediate diaphragms in-service have led to a lack of cohesion throughout the U.S. Regardless of the debate about the effectiveness of intermediate diaphragms in-service, it is imperative that the girders are braced immediately upon being erected onto their supports. Erected, unbraced girders are highly susceptible to instability, endangering both the structural integrity of the bridge, worker safety, and public safety.

6.2 Conclusions

The following conclusions can be made about the effectiveness and use of intermediate diaphragms throughout the U.S.

1. Bracing of girders during erection and deck construction operations is critical to ensure stability of the girders, worker safety, and public safety.
2. The use of permanent steel intermediate diaphragms and temporary steel bracing options are becoming more popular with state transportation agencies.

6.3 Recommendations

The following recommendations are based on data collected through this research, and provide ideas for future avenues of research that should be addressed.

1. ALDOT may consider developing standards or recommendations to aid contractors in designing an adequate temporary bracing system. A review of the details already in practice by FDOT, KDOT, and TxDOT by ALDOT engineers and contractors is suggested as a starting point towards the development of standards that will suit Alabama.
2. Existing bridge codes do not provide standards for the design of intermediate diaphragms and future bridge design codes should be revised to include more detailed specifications to advise state transportation departments. These codes should address the most crucial stages of construction where bracing girders for stability is essential – from the girder placement through the deck pour operations.
3. This research effort focused on documenting the details of steel intermediate diaphragms and temporary bracing standards. An in-depth survey into the

details of concrete intermediate diaphragms across the U.S. should be completed.

4. Temporary bracing of girders during construction, instead of permanent intermediate diaphragms, has been a long standing practice by the Florida Department of Transportation (FDOT). FDOT has developed a detailed *Beam Stability* Mathcad program (FDOT 2013) designed to estimate the wind and construction loads on girders and calculate bracing requirements to provide adequate lateral stability. The only details specific to the state of Florida used in the calculations are the girder and bearing pad dimensions and properties. If updated to include properties relevant to girders and bearing pads used by ALDOT, the *Beam Stability* Mathcad program could provide a valuable resource to check the adequacy of temporary bracing methods in Alabama.

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Appendix A: Example Using *Beam Stability v2.1* (FDOT 2013)

These are calculations for the Lateral Stability of Precast Concrete Bridge Girders during construction.

Instructions for use of this program:

1) Input the items under the girder properties, geometry, and loads sections highlighted in tan. For the girders listed in the "Girder Type" pull-down menu, un-highlighted girder properties are automatically defined. For any other girder types, properties must be manually defined. The number of intermediate bracing points, from zero to six, represents any intermediate bracing that is to be present between the points of bearing. A value of zero represents no intermediate bracing points between the bearing points.

2) Check that the stress and stability checks (highlighted in yellow) read "OK." The check for stability at girder placement may read "Not OK," but for this case, girders must be braced prior to crane release.

3) If requirement 2 is not met, revise the number of intermediate brace points.

4) The bracing forces and maximum un-braced length are given at the end of calculations.

The calculations in this program are the result of the following research/publications:

American Association of State Highway and Transportation Officials (AASHTO). "AASHTO LRFD Bridge Design Specifications." Washington, D.C., 2012.

Consolazio, Gary R. , and Kurtis R. Gurley. "Bridge Girder Drag Coefficients and Wind-Related Bracing Recommendations (FDOT BDK 75-977-33)." Final Report, Civil and Coastal Engineering , University of Florida, Gainesville, FL , 2013.

Consolazio, Gary R. , and H.R. (Trey) Hamilton III. "Lateral Bracing of Long-Span Florida Bulb-Tee Girders (FDOT BD 545-36)." Final Report, Civil and Coastal Engineering, University of Florida, Gainesville, FL, 2007.

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Source of Original Program:

Florida Department of Transportation (FDOT). "Structures Design Programs Library." *Beam Stability v2.1.* http://www.dot.state.fl.us/structures/proglib.shtm, December 2, 2013.

*** The formatting of this program has been modified for this report, and annotations have been added to explain calculations and variables used within ***

Girder Variables:

Girder Type	Girder := 36-in Florida-I Beam
Unit weight of Concrete	$w_c := 150 \cdot pcf$
Concrete Strength	$f_{c'} := 8.5 \cdot ksi$
Modulus of Elasticity Correction Factor (= 0.9 reduction for Florida lime rock coarse aggregate)	$K_1 := 0.9$
Effective Prestressing Force (may assume all losses have occurred)	$P_e := 1508.6 \cdot kip$
Eccentricity of Prestressing	$e_p := 11.51 \cdot in$

Bridge Geometry:

Beam Span Length (centerline to centerline bearing)	$L := 90 \cdot ft$
Number of Intermediate Bracing Points, (from 0 to 20)	$n_b := 1$
Sweep Tolerance	$tol_S := \frac{\frac{1}{8} \cdot in}{10 \cdot ft}$
Initial imperfection of bracing	$e_b := .25 \cdot in$
Eccentricity due to setting the beams off-center on the pads (0.25 in recommended)	$e_{set} := .25 \cdot in$
Skew Angle between beam and bearing pad (0 and 60 degrees)	$\phi := 0 \cdot deg$
Beam Spacing	$S := 10 \cdot ft$
Number of Beams in cross section (from 2 to 12)	$n_{beam} := 9$
Overhang Length (measured from centerline of exterior beam)	$OH := 4.542 \cdot ft$
Deflection of Deck Limit at Edge of Cantilever (.25 in. recommended)	$\delta_{max} := .25 \cdot in$
Deck thickness ($t_{slab} + t_{mill}$)	$t_d := 8.5 \cdot in$

Bearing Pad Properties:

Bearing Pad Type	BearingPad := D
Transverse Tilt Angle of Support in Radians (Bearing Pad Construction Tolerance, 0.01 recommended)	$\alpha := .01$

Loads:

FDOT SDG Chapter 2 - "Loads and Load Factors"

Basic Wind Speed	$V_B := 100 \cdot mph$	*(FDOT SDG Table 2.4.1-2)*
Wind Speed Factor for Construction Inactive Wind Speed	$R_E := 0.6$	
	$V := R_E \cdot V_B = 60 \cdot mph$	
Construction Active Wind Speed	$V_E := 20 \cdot mph$	*(20 mph recommended)*
Construction Wind Load Factor	$\gamma := 1.25$	
Gust effect factor	$G := 0.85$	
Pressure Coefficient, single girder	$C_{pg} := 2.2$	*(FDOT IDS 2014)*
Pressure Coefficient, maximum bracing (forms in place)	$C_{pbr} := 1.1$	
Bridge Height, measured to mid-height of beam	$Height := 20.5 \cdot ft$	
Velocity Pressure Exposure Coefficient	$K_z := \max\left[2.01 \cdot \left(\frac{Height}{900 \cdot ft}\right)^{.2105}, 0.85\right] = 0.907$	
Construction Active Wind Load for single girder	$w_{wE} := 0.00256 \cdot K_z \cdot G \cdot C_{pg} \cdot \left(\frac{V_E}{mph}\right)^2 \cdot psf = 1.736 \cdot psf$	
Construction Inactive Wind Load for single girder	$w_w := 0.00256 \cdot K_z \cdot G \cdot C_{pg} \cdot \left(\frac{V}{mph}\right)^2 \cdot psf = 15.626 \cdot psf$	
Weight of build-up	$w_b := 50 \cdot plf$	
Weight of forms between beams	$w_f := 20 \cdot psf$	*(20 psf. recommended)*
Weight of overhang forms	$w_{OH} := 20 \cdot psf$	*(20 psf. recommended)*
Live loads during deck pour *(FDOT SDG 2.13.1)*	$w_l := 20 \cdot psf$	$P_l := 75 \cdot plf$

Total Weight of finishing machine $w_{fm} := 16 \cdot kip$

Recommended values are:

26'-32' Wide: 6.4 k	*56'-68' Wide: 12k*
32'-44' Width: 10 k	*68'-80' Wide: 13k*
44'-56' Width: 11 k	*80'-120' Wide: 16k*

Wheel Location of finishing machine in relation to edge of overhang, positive is to exterior of overhang edge, negative is to interior of overhang edge	$d_{fm} := 2.5 \cdot in$	*(+2.5 in. recommended)*
Worker platform width	$d_{wp} := 2 \cdot ft$	*(2 ft. recommended)*
Dead Load of Worker platform	$w_{wp} := 20 \cdot psf$	*(20 psf. recommended)*

Girder Properties

Reference to Excel Properties file — Properties := READFILE("BeamProp.xls" ,"Excel")

Unbraced Length of Beam — $L_b := \frac{L}{n_b + 1} = 45\,ft$

Height — $h := Properties_{Girder,1} \cdot in = 36 \cdot in$

Top flange width — $b_t := Properties_{Girder,2} \cdot in = 48 \cdot in$

Bottom flange width — $b_b := Properties_{Girder,3} \cdot in = 38 \cdot in$

Modulus of Elasticity — $E_c := 33000 \cdot K_1 \cdot .145^{1.5} \cdot \left(\frac{f_{c'}}{ksi}\right)^{0.5} \cdot ksi = 4.781 \times 10^3 \cdot ksi$

Shear Modulus — $G_{shear} := .416667 \cdot E_c = 1.992 \times 10^3 \cdot ksi$

Area of Concrete — $A_c := Properties_{Girder,4} \cdot in^2 = 806.58 \cdot in^2$

Moment of Inertia, about x-axis — $I_x := Properties_{Girder,5} \cdot in^4 = 127564 \cdot in^4$

Moment of Inertia, about y-axis — $I_y := Properties_{Girder,6} \cdot in^4 = 81131 \cdot in^4$

Distance from CG to top of beam — $y_t := Properties_{Girder,7} \cdot in = 19.51 \cdot in$

Distance from CG to bottom of beam — $y_b := Properties_{Girder,8} \cdot in = 16.49 \cdot in$

Torsional Constant — $J := Properties_{Girder,9} \cdot in^4 = 28654 \cdot in^4$

Section Moduli

Section moduli about x-axis — $S_t := \frac{I_x}{y_t} = 6538 \cdot in^3$ — $S_b := \frac{I_x}{y_b} = 7736 \cdot in^3$

Section moduli about y-axis — $S_{yt} := \frac{2 \cdot I_y}{b_t} = 3380 \cdot in^3$ — $S_{yb} := \frac{2 \cdot I_y}{b_b} = 4270 \cdot in^3$

Self-weight of beam and deck — $w := A_c \cdot w_c = 840.188 \cdot plf$ — $w_d := t_d \cdot w_c = 106.25 \cdot psf$

Lateral Deflection and Eccentricity of Girder Center of Gravity:

Maximum Lateral Deflection of Uncracked Section

$$z_o := \frac{w \cdot L^4}{120 \cdot E_c \cdot I_y} = 2.046 \cdot in$$

This is the theoretical maximum lateral deflection of the beam based on beam self-weight if cracking did not occur

Eccentricity due to Sweep

$$e_s := \min\left(1.5 \cdot in, L \cdot tol_S\right) \frac{2}{3} = 0.75 \cdot in$$

Based on the sweep tolerance and 1.5" limit per the Specifications, this is maximum sweep that could occur, the 2/3 factor is included because the average location of the CG over the length of the beam is 2/3 of the maximum sweep

Eccentricity due to construction inactive wind speed

$$e_w := \frac{w_w \cdot h \cdot L^4}{120 \cdot E_c \cdot I_y} = 0.114 \cdot in$$

Eccentricity due to wind loading at construction active wind speed, girder only

$$e_{wE} := \frac{w_{wE} \cdot h \cdot L^4}{120 \cdot E_c \cdot I_y} = 0.013 \cdot in$$

Lateral deflection due to wind, based on uncracked section

Bearing Pad Rotational Stiffness

Effect of Skew on Stiffness (coefficient)

$Ang := (0\ \ 15\ \ 30\ \ 45\ \ 60)$

$Stiffness := (0\ \ .41\ \ .58\ \ .7\ \ .79)$

Range of stiffness coefficients per skew per FDOT Structures Research Projects BD 545-36 & BDK 75-977-33.
Skew adjustment explained by FDOT:

The draft research report for project BDK 75-977-33 'Bridge Girder Drag Coefficients and Wind Related Bracing Recommendations' includes an equation for roll stiffness of bearing pads in chapter 6 (equation 6.29). The equation is valid for rectangular bearing pads bent about an orthogonal axis. Roll stiffnesses of skewed bearing pads cannot be calculated based on the equation presented. Research was done for rotational stiffness of skewed bearing pads in research project BD 545-36 "Lateral Bracing of Long-Span Florida Bulb-Tee Girders". The chart below shows the rotational stiffness of a type B (14"x24") bearing pad according to skew angle.

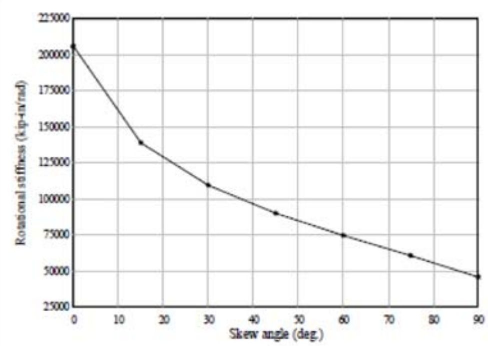

Figure 5.10 Rotational stiffness vs. skew angle

Figure 5.10 shows data for only one bearing pad dimension. It is assumed that the graph shape is similar for bearing pads with different dimensions. In order to apply the graph shape to different bearing pad shapes with different stiffnesses at 0 and 90 degrees, the following equations were used:
Based on Figure 5.10: Percent Reduction Per Skew, R = (Kϑ.Skew - Kϑ.0)/(Kϑ.90 - Kϑ.0)
Rotational Stiffness for Other Bearing Pad Shapes: Kϑ = Kϑ.0 + R x (Kϑ.90 - Kϑ.0)
Rotational Stiffness for 75 and 90 degree skew are not needed because they are not included in the beam stability program.

Per Figure 5.10:				Skew						
	Kθ at 0°	Kθ at 90°	Kθ90 - Kθ0	0	15	30	45	60	75	90
Kθ:	206000	40000	-166000	206000	137500	110000	90000	75000	62500	40000
Percent Reduction Per Skew, R:				0%	41%	58%	70%	79%	86%	100%

Rotational Stiffness at 0 and 90 degrees per BDK 75-977-33 'Bridge Girder Drag Coefficients and Wind-Related Bracing Recommendations' Draft Final Report Appendix D (p 162)

FDOT Bearing Pad Type:

				Skew				
	Kθ at 0°	Kθ at 90°	Kθ90 - Kθ0	0	15	30	45	60
A	6330	1330	-5000	6330	4267	3438	2836	2384
B	7600	2590	-5010	7600	5533	4703	4099	3646
C	5900	1610	-4290	5900	4130	3419	2902	2515
D	7270	458	-6812	7270	4459	3331	2510	1894
E	12900	1260	-11640	12900	8097	6168	4766	3714
F	9080	890	-8190	9080	5700	4344	3357	2617
G	11500	1130	-10370	11500	7221	5503	4253	3316
H	8910	870	-8040	8910	5592	4260	3292	2565
J	7260	712	-6548	7260	4558	3473	2684	2093
K	11100	1560	-9540	11100	7163	5583	4433	3571

Bearing Pad Rotational Stiffness (Continued)

Reference to Excel Properties file

$PropertiesBP := READFILE("BearingPadProp.xls", "Excel")$

*** If custom bearing pad is used, input axial stiffness, length and width ***

Axial Bearing Pad Stiffness

$k_{axial} := PropertiesBP_{BearingPad,1} \cdot \frac{kip}{ft} = 45900 \cdot \frac{kip}{ft}$

Bearing Pad Width

$b := PropertiesBP_{BearingPad,2} \cdot in = 32 \cdot in$

Bearing Pad Length

$a := PropertiesBP_{BearingPad,3} \cdot in = 8 \cdot in$

Distance from Bottom of Beam to Roll Axis (half bearing pad thickness)

$h_r := .5 \cdot PropertiesBP_{BearingPad,4} \cdot in = 0.953 \cdot in$

Elastomer Shear Modulus

$G_{bp} := .85 \cdot PropertiesBP_{BearingPad,5} \cdot psi = 93.5\,psi$

The elastomer shear modulus used for calculations should be 15% less than plan value (AASHTO 14.7.5.2) because the material is not homogenous

Slope Angle

$\phi_{slope} := Properties_{Girder,10} = 0.025$

The slope is the predicted slope between the bearing pad and beam before the deck is poured. The predicted maximum camber is assumed, with a 0.005 construction tolerance and maximum grade of 2%. Inputting a project-specific value will greatly reduce conservatism.

Roll Stiffness

$$p_0 := \min\left(1, \sqrt{\frac{9 \cdot .5 \cdot w \cdot L}{2 \cdot a \cdot \phi_{slope} \cdot k_{axial}}}\right) = 0.472$$

$$p_1 := \min\left(1, \sqrt{\frac{9 \cdot .5 \cdot w \cdot L}{2 \cdot b \cdot \phi_{slope} \cdot k_{axial}}}\right) = 0.236$$

$$K_{\theta.0} := \left[p_0^{\,2} \cdot \left(3 - 2 \cdot p_0\right) \cdot \frac{k_{axial} \cdot b^2}{45}\right] = 39815.533 \cdot \frac{kip \cdot in}{rad}$$

$$K_{\theta.1} := \left[p_1^{\,2} \cdot \left(3 - 2 \cdot p_1\right) \cdot \frac{k_{axial} \cdot a^2}{45}\right] = 764.76 \cdot \frac{kip \cdot in}{rad}$$

Rotational Stiffness
[Per BDK 75-977-33]

$$K_{\theta} := 2\left[K_{\theta.0} + \mathrm{linterp}\left(Ang^T, Stiffness^T, \frac{\phi}{deg}\right) \cdot \left(K_{\theta.1} - K_{\theta.0}\right)\right] = 79631.067 \cdot \frac{kip \cdot in}{rad}$$

Coefficient for Reaction at Bracing Based on Number of Brace Points

Intermediate (i):

$$k_{vi} := \begin{pmatrix} 0.000 & 0.000 & 0.000 & 0.000 & 0.000 & 0.000 & 0.000 & 0.000 & 0.000 & 0.000 & 0.000 \\ 0.629 & 0.838 & 0.941 & 1.003 & 1.044 & 1.072 & 1.096 & 1.113 & 1.127 & 1.137 & 1.144 \\ 0.557 & 0.737 & 0.830 & 0.881 & 0.917 & 0.943 & 0.964 & 0.979 & 0.989 & 1.000 & 1.010 \\ 0.584 & 0.770 & 0.866 & 0.921 & 0.955 & 0.983 & 0.996 & 1.010 & 1.024 & 1.031 & 1.038 \\ 0.584 & 0.773 & 0.859 & 0.910 & 0.945 & 0.971 & 0.988 & 1.005 & 1.014 & 1.022 & 1.031 \\ 0.598 & 0.773 & 0.866 & 0.917 & 0.948 & 0.979 & 0.989 & 1.000 & 1.010 & 1.020 & 1.031 \\ 0.601 & 0.782 & 0.878 & 0.926 & 0.950 & 0.974 & 0.986 & 0.998 & 1.010 & 1.022 & 1.022 \\ 0.618 & 0.797 & 0.880 & 0.935 & 0.962 & 0.976 & 0.989 & 1.003 & 1.003 & 1.017 & 1.017 \\ 0.634 & 0.819 & 0.897 & 0.943 & 0.959 & 0.974 & 0.989 & 1.005 & 1.005 & 1.020 & 1.020 \\ 0.653 & 0.825 & 0.910 & 0.945 & 0.962 & 0.979 & 0.996 & 0.996 & 1.014 & 1.014 & 1.014 \\ 0.680 & 0.850 & 0.926 & 0.964 & 0.983 & 0.983 & 1.002 & 1.002 & 1.002 & 1.020 & 1.020 \\ 0.722 & 0.886 & 0.948 & 0.969 & 0.989 & 1.010 & 1.010 & 1.010 & 1.010 & 1.010 & 1.010 \\ 0.759 & 0.916 & 0.960 & 0.983 & 1.005 & 1.005 & 1.005 & 1.005 & 1.005 & 1.005 & 1.005 \\ 0.794 & 0.938 & 0.986 & 1.010 & 1.010 & 1.010 & 1.010 & 1.010 & 1.010 & 1.010 & 1.010 \\ 0.850 & 0.979 & 1.005 & 1.031 & 1.031 & 1.031 & 1.031 & 1.031 & 1.031 & 1.031 & 1.031 \\ 0.880 & 1.017 & 1.044 & 1.044 & 1.044 & 1.044 & 1.044 & 1.044 & 1.044 & 1.017 & 1.017 \\ 0.935 & 1.051 & 1.081 & 1.081 & 1.081 & 1.051 & 1.051 & 1.051 & 1.051 & 1.022 & 1.022 \\ 0.989 & 1.113 & 1.113 & 1.113 & 1.082 & 1.082 & 1.082 & 1.051 & 1.051 & 1.051 & 1.051 \\ 1.077 & 1.142 & 1.142 & 1.142 & 1.110 & 1.110 & 1.077 & 1.077 & 1.044 & 1.044 & 1.044 \\ 1.134 & 1.203 & 1.203 & 1.168 & 1.168 & 1.134 & 1.099 & 1.099 & 1.065 & 1.065 & 1.065 \\ 1.190 & 1.263 & 1.227 & 1.190 & 1.154 & 1.118 & 1.118 & 1.082 & 1.082 & 1.082 & 1.046 \end{pmatrix}$$

Creation of the "k" matrices: FDOT analyzed RISA models with varying numbers of beams and bracing to determine reactions and moments. Then, divided the load applied and span length to determine coefficients. (Christina Freeman, P.E., FDOT)

Rows: Relate to number of intermediate bracing points (from 0 to 20)

Columns: Relate to number of beams (from 2 to 12)

$$K_{vi} := k_{vi_{n_b, n_{beam}-2}} = 1.113$$

End (e):

$$H := \frac{h}{ft} = 3 \qquad Z := \frac{S}{ft} = 10$$

$$k_{ve_1} := .1548634 + .130797 \cdot \ln(H) + .000832 \cdot H \cdot Z - .00381 \cdot Z \cdot n_{beam} - .00561 \cdot H \cdot n_{beam}^{\ 2} = -1.383$$

$$k_{ve_2} := .0907444 + .200578 \cdot \ln(H) + .001155 \cdot H \cdot Z - 2.1 \cdot 10^{-8} \cdot e^{(Z)} \cdot e^{\left(n_{beam}\right)} - .00345 \cdot H \cdot n_{beam}^{\ 2} = -4.241$$

$$k_{ve_3} := \Big[.0805514 - .03341 \cdot H + .214357 \cdot \ln(H) - 3.8 \cdot 10^{-7} \cdot e^{(Z)} + 3.5 \cdot 10^{-8} \cdot e^{n_{beam}} + .000832 \cdot H \cdot Z + .002484 \cdot H \cdot n_{beam} - .00026 \cdot Z \cdot n_{be}$$
$$+ 2.07 \cdot 10^{-13} \cdot e^{(Z)} \cdot e^{n_{beam}} - .00012 \cdot H \cdot n_{beam}^{\ 2}$$

$$k_{ve_4} := .0580771 + .049056 \cdot \ln(H) + .000312 \cdot H \cdot Z - .00143 \cdot Z \cdot n_{beam} - .0021 \cdot H \cdot n_{beam}^{\ 2} = -0.518$$

Coefficient for Reaction.... (Continued)

$$k_{ve_5} := .034036 + .075244 \cdot \ln(H) + .000434 \cdot H \cdot Z - 8 \cdot 10^{-9} \cdot e^{(Z)} \cdot e^{(n_{beam})} - .00129 \cdot H \cdot n_{beam}{}^{2} = -1.612$$

$$k_{ve_6} := \left[\begin{array}{l} .0301988 - .01256 \cdot H + .080445 \cdot \ln(H) - 1.4 \cdot 10^{-7} \cdot e^{(Z)} + 1.31 \cdot 10^{-8} \cdot e^{n_{beam}} + .000313 \cdot H \cdot Z + .000934 \cdot H \cdot n_{beam} - 9.9 \cdot 10^{-5} \cdot Z \\ + 7.8 \cdot 10^{-14} \cdot e^{(Z)} \cdot e^{n_{beam}} - 4.5 \cdot 10^{-5} \cdot H \cdot n_{beam}{}^{2} \end{array} \right.$$

$$k_{ve_7} := 0.0513364 + .048951 \cdot \ln(H) - .031 \cdot \ln\left(\max\left(.1, n_b\right)\right) - .02063 \cdot \ln(H) \cdot \ln\left(\max\left(.1, n_b\right)\right) + .000173 \cdot H \cdot Z - .00105 \cdot Z \cdot n_{beam} + .000172 \cdot$$
$$+ -.00188 \cdot H \cdot n_{beam}{}^{2} - 7.9 \cdot 10^{-6} \cdot H \cdot n_b{}^{3} + .000522 \cdot H \cdot n_{beam} \cdot n_b + .002314 \cdot n_{beam} \cdot n_b$$

$$k_{ve_8} := 0.0507789 + .075099 \cdot \ln(H) - .02716 \cdot \ln\left(\max\left(.1, n_b\right)\right) - .03163 \cdot \ln(H) \cdot \ln\left(\max\left(.1, n_b\right)\right) + .00024 \cdot H \cdot Z - .0011 \cdot Z \cdot n_{beam} + .000296 \cdot$$
$$+ -.00114 \cdot H \cdot n_{beam}{}^{2} - 8.1 \cdot 10^{-6} \cdot H \cdot n_b{}^{3} + .000454 \cdot H \cdot n_{beam} \cdot n_b + .001287 \cdot n_{beam} \cdot n_b$$

$$k_{ve_9} := .0298736 - .00755 \cdot H + .073344 \cdot \ln(H) - .01962 \cdot \ln\left(\max\left(.1, n_b\right)\right) - .02721 \cdot \ln(H) \cdot \ln\left(\max\left(.1, n_b\right)\right) - 1.3 \cdot 10^{-7} \cdot e^{Z} + 7.38 \cdot 10^{-9} \cdot e^{n_b}$$
$$+ .000175 \cdot H \cdot Z + .000561 \cdot H \cdot n_{beam} - 5.4 \cdot 10^{-5} \cdot Z \cdot n_{beam} + .000434 \cdot Z \cdot n_b + 4.21 \cdot 10^{-14} \cdot e^{Z} \cdot e^{n_{beam}} - 2.5 \cdot 10^{-5} \cdot H \cdot n_{beam}{}^{2} + 1.69$$
$$+ -1.3 \cdot 10^{-5} \cdot H \cdot n_{beam} \cdot n_b - 4.8 \cdot 10^{-6} \cdot n_{beam} \cdot n_b$$

$$k_{ve_{10}} := .0271265 + .03591 \cdot \ln(H) - .01161 \cdot \ln(H) \cdot \ln\left(\max\left(.1, n_b\right)\right) + 8.87 \cdot 10^{-5} \cdot H \cdot Z - .00056 \cdot Z \cdot n_{beam} + 4.86 \cdot 10^{-5} \cdot Z \cdot n_b + 5.94 \cdot 10^{-6} \cdot$$
$$+ -.00035 \cdot H \cdot n_{beam} \cdot n_b - .00077 \cdot n_{beam} \cdot n_b$$

$$k_{ve_{11}} := .0271625 + .055 \cdot \ln(H) - .01771 \cdot \ln(H) \cdot \ln\left(\max\left(.1, n_b\right)\right) + .000125 \cdot H \cdot Z - .0006 \cdot Z \cdot n_{beam} + 8.45 \cdot 10^{-5} \cdot Z \cdot n_b - .00058 \cdot H \cdot n_{beam}{}^{2}$$
$$+ 8.92 \cdot 10^{-5} \cdot H \cdot n_{beam} \cdot n_b - .00049 \cdot n_{beam} \cdot n_b$$

$$k_{ve_{12}} := .006385 - .00433 \cdot H + .058732 \cdot \ln(H) - .01881 \cdot \ln(H) \cdot \ln\left(\max\left(.1, n_b\right)\right) - 7.4 \cdot 10^{-8} \cdot e^{Z} + 3.75 \cdot 10^{-9} \cdot e^{n_{beam}} - 7.5 \cdot 10^{-7} \cdot e^{n_b} + 9.$$
$$+ .000301 \cdot H \cdot n_{beam} - 2.6 \cdot 10^{-5} \cdot Z \cdot n_{beam} + .000127 \cdot Z \cdot n_b + 2.14 \cdot 10^{-14} \cdot e^{Z} \cdot e^{n_{beam}} - 1.4 \cdot 10^{-5} \cdot H \cdot n_{beam}{}^{2} + 1.41 \cdot 10^{-6} \cdot H \cdot n_b{}^{3}$$
$$+ -2.8 \cdot 10^{-6} \cdot H \cdot n_{beam} \cdot n_b - 3.3 \cdot 10^{-6} \cdot n_{beam} \cdot n_b$$

$$k_{ve_{13}} := .0312766 + .022184 \cdot \ln(H) - .01043 \cdot \ln\left(\max\left(.1, n_b\right)\right) - .00566 \cdot \ln(H) \cdot \ln\left(\max\left(.1, n_b\right)\right) + 4.48 \cdot 10^{-5} \cdot H \cdot Z - .00028 \cdot Z \cdot n_{beam} + 1.3$$
$$+ -.00046 \cdot H \cdot n_{beam}{}^{2} + 2.1 \cdot 10^{-5} \cdot H \cdot n_{beam} \cdot n_b + .000162 \cdot n_{beam} \cdot n_b$$

$$k_{ve_{14}} := .0310998 + .037269 \cdot \ln(H) - .01082 \cdot \ln\left(\max\left(.1, n_b\right)\right) - .00978 \cdot \ln(H) \cdot \ln\left(\max\left(.1, n_b\right)\right) + 6.58 \cdot 10^{-5} \cdot H \cdot Z - .0003 \cdot Z \cdot n_{beam} \ldots = -$$
$$+ 2.1 \cdot 10^{-5} \cdot Z \cdot n_b + -.00033 \cdot H \cdot n_{beam}{}^{2} - 2.7 \cdot 10^{-8} \cdot H \cdot n_b{}^{3} + 3.04 \cdot 10^{-5} \cdot H \cdot n_{beam} \cdot n_b + .000142 \cdot n_{beam} \cdot n_b$$

$$k_{ve_{15}} := .0147435 - .00242 \cdot H + .037228 \cdot \ln(H) - .00477 \cdot \ln\left(\max\left(.1, n_b\right)\right) - .0093 \cdot \ln(H) \cdot \ln\left(\max\left(.1, n_b\right)\right) - 3.5 \cdot 10^{-8} \cdot e^{Z} + 2.21 \cdot 10^{-9} \cdot e^{n_b}$$
$$+ -3.6 \cdot 10^{-13} \cdot e^{n_b} + 5.64 \cdot 10^{-5} \cdot H \cdot Z + .000187 \cdot H \cdot n_{beam} - 1.6 \cdot 10^{-5} \cdot Z \cdot n_{beam} + 2.84 \cdot 10^{-5} \cdot Z \cdot n_b + 1.17 \cdot 10^{-14} \cdot e^{Z} \cdot e^{n_{beam}} \ldots$$
$$+ -8.4 \cdot 10^{-6} \cdot H \cdot n_{beam}{}^{2} + 6.64 \cdot 10^{-8} \cdot H \cdot n_b{}^{3} - 6 \cdot 10^{-7} \cdot H \cdot n_{beam} \cdot n_b - 4.9 \cdot 10^{-7} \cdot n_{beam} \cdot n_b$$

Coefficient for Reaction.... (Continued)

$$Bm := \text{if}\left(n_{beam} = 2, 0, \text{if}\left(n_{beam} = 3, 1, \text{if}\left(n_{beam} \geq 4, 2, \text{"Err"}\right)\right)\right) = 2$$

$$Br := \text{if}\left(n_b = 0, 0, \text{if}\left(n_b = 1, 1, \text{if}\left(n_b \geq 2 \wedge n_b \leq 4, 2, \text{if}\left(n_b \geq 5 \wedge n_b \leq 9, 3, \text{if}\left(n_b \geq 10 \wedge n_b \leq 20, 4, \text{"Err"}\right)\right)\right)\right)\right) = 1$$

$$nkve := \begin{pmatrix} 1 & 4 & 7 & 10 & 13 \\ 2 & 5 & 8 & 11 & 14 \\ 3 & 6 & 9 & 12 & 15 \end{pmatrix}$$

$$Nkve := nkve_{Bm, Br} = 6$$

$$K_{vi} := kvi_{n_b, n_{beam}-2} = 1.113$$

$$K_{ve} := kve_{Nkve} = 0.093$$

$$k_m := \begin{pmatrix}
0.125 & 0.125 & 0.125 & 0.125 & 0.125 & 0.125 & 0.125 & 0.125 & 0.125 & 0.125 & 0.125 \\
0.312 & 0.207 & 0.155 & 0.124 & 0.114 & 0.107 & 0.102 & 0.099 & 0.096 & 0.093 & 0.091 \\
0.570 & 0.388 & 0.297 & 0.242 & 0.206 & 0.181 & 0.161 & 0.146 & 0.134 & 0.125 & 0.121 \\
1.028 & 0.684 & 0.513 & 0.411 & 0.345 & 0.300 & 0.266 & 0.241 & 0.220 & 0.203 & 0.189 \\
1.575 & 1.063 & 0.809 & 0.655 & 0.552 & 0.480 & 0.425 & 0.383 & 0.348 & 0.322 & 0.298 \\
2.278 & 1.515 & 1.136 & 0.909 & 0.760 & 0.656 & 0.581 & 0.521 & 0.470 & 0.432 & 0.401 \\
3.062 & 2.053 & 1.546 & 1.245 & 1.044 & 0.902 & 0.795 & 0.709 & 0.644 & 0.588 & 0.541 \\
4.016 & 2.670 & 2.003 & 1.599 & 1.335 & 1.150 & 1.010 & 0.903 & 0.819 & 0.746 & 0.684 \\
5.048 & 3.372 & 2.542 & 2.038 & 1.704 & 1.470 & 1.285 & 1.150 & 1.036 & 0.944 & 0.873 \\
6.249 & 4.154 & 3.111 & 2.489 & 2.077 & 1.788 & 1.569 & 1.394 & 1.262 & 1.148 & 1.061 \\
7.530 & 5.027 & 3.775 & 3.022 & 2.524 & 2.174 & 1.909 & 1.697 & 1.538 & 1.400 & 1.283 \\
8.986 & 5.982 & 4.480 & 3.572 & 2.979 & 2.562 & 2.247 & 1.994 & 1.805 & 1.641 & 1.515 \\
10.517 & 7.006 & 5.258 & 4.221 & 3.525 & 3.022 & 2.651 & 2.355 & 2.133 & 1.940 & 1.777 \\
12.214 & 8.125 & 6.081 & 4.862 & 4.054 & 3.487 & 3.058 & 2.714 & 2.439 & 2.233 & 2.044 \\
14.001 & 9.328 & 7.001 & 5.601 & 4.674 & 4.003 & 3.510 & 3.136 & 2.820 & 2.564 & 2.347 \\
15.931 & 10.613 & 7.943 & 6.350 & 5.295 & 4.532 & 3.971 & 3.545 & 3.186 & 2.894 & 2.670 \\
17.959 & 11.981 & 8.967 & 7.194 & 6.003 & 5.142 & 4.509 & 4.002 & 3.597 & 3.293 & 3.014 \\
20.162 & 13.404 & 10.053 & 8.036 & 6.702 & 5.736 & 5.026 & 4.458 & 4.032 & 3.663 & 3.351 \\
22.433 & 14.934 & 11.201 & 8.954 & 7.467 & 6.423 & 5.600 & 4.999 & 4.493 & 4.082 & 3.765 \\
24.891 & 16.548 & 12.411 & 9.922 & 8.274 & 7.082 & 6.205 & 5.504 & 4.978 & 4.523 & 4.137 \\
27.404 & 18.244 & 13.683 & 10.938 & 9.122 & 7.808 & 6.841 & 6.107 & 5.489 & 4.986 & 4.561
\end{pmatrix}$$

Coefficient for Bending Moment in Girder based on # of Brace Points

$$K_M := k_{m_{n_b, n_{beam}-2}} = 0.099$$

Calculation of Bending Moments:

Unfactored vertical load during deck placement for beam (not including finishing machine)

Includes self-wt of girder, build-up, forms, wet concrete deck, and construction live loads

$$w_{D.ext} := w + w_b + w_f \cdot .5 \cdot S + w_{OH} \cdot OH - .5 \cdot b_t \cdot \left(w_f + w_{OH}\right) + w_{wp} \cdot d_{wp} + w_d \cdot (.5 \cdot S + OH) \ldots + w_l \cdot \left(.5S + OH + d_{wp}\right) = 2.29 \cdot klf$$

$$w_{D.int} := w + w_b + \left(w_d + w_l\right) \cdot S + w_f \cdot \left(S - b_t\right) = 2.273 \cdot klf$$

Strength I Torsional Distributed Overhang Moment during deck placement

Includes all construction loads except finishing machine

$$\begin{aligned} M_c := {} & 1.25 \cdot \left(w_d + w_f\right) \cdot \left(.5 \cdot S - .5 \cdot b_t\right) \cdot \left(-.5b_t\right) \ldots \\ & + \left(1.25 \cdot w_d + 1.5w_{OH}\right) \cdot \left(OH - .5 \cdot b_t\right) \cdot \left(.25 \cdot b_t + .5 \cdot OH\right) \ldots \\ & + 1.5 \cdot w_{wp} \cdot d_{wp} \cdot \left(OH + .5 \cdot d_{wp}\right) \ldots \\ & + 1.5 \cdot w_l \cdot \left(.5 \cdot S - .5 \cdot b_t\right) \cdot \left(-.5 \cdot b_t\right) \ldots \\ & + 1.5 \cdot w_l \cdot \left(OH - .5b_t + d_{wp}\right) \cdot \left(.25 \cdot b_t + .5 \cdot OH + .5 \cdot d_{wp}\right) \end{aligned} = 1.14 \cdot \frac{kip \cdot ft}{ft}$$

Strength I Torsional Finishing Machine and 75 plf Live Load Moment

Finishing Machine Moment at each exterior girder

$$M_{fm} := 1.5 \cdot \left[.5 \cdot w_{fm} \cdot \left(OH + d_{fm}\right) + 20 \cdot ft \cdot P_l \cdot \left(OH + .5 \cdot d_{wp}\right)\right] = 69.474 \cdot kip \cdot ft$$

Lateral Moment Due to Construction Inactive wind speed

$$M_w := K_M \cdot w_w \cdot h \cdot L_b^{\,2} = 113 \cdot kip \cdot in$$

Vertical Moment due to girder self-weight

$$M_g := \frac{w \cdot L^2}{8} = 10208 \cdot kip \cdot in$$

Lateral Moment Due to Construction Active Wind speed, braced condition

$$M_{wE} := K_M \cdot w_{wE} \cdot h \cdot L_b^{\,2} = 13 \cdot kip \cdot in$$

Lateral Moment Due to Construction Active Wind speed, unbraced condition

$$M_{wE.u} := .125 \cdot w_{wE} \cdot h \cdot L^2 = 63 \cdot kip \cdot in$$

Vertical Moment due to self-weight and construction loads during deck placement

$$M_{gD} := \max\left[\frac{w_{D.ext} \cdot L^2}{8} + \frac{\left(.5w_{fm} + P_l \cdot 20 \cdot ft\right) \cdot L}{4}, \frac{w_{D.int} \cdot L^2}{8}\right] = 30336 \cdot kip \cdot in$$

Service Stress Check for Girder Placement, prior to beam bracing (Service I, Constructive Active):

Camber (approx.)

$$\delta_c := \frac{L^2 \cdot \left(P_e \cdot e_p - \frac{5w \cdot L^2}{48}\right) \cdot 2}{8 \cdot E_c \cdot I_x} = 4.235 \cdot in$$

Assumes creep factor is 2.0

Distance from Center of Gravity to Roll Axis

$$y := y_b + h_r + \delta_c \cdot \frac{2}{3} = 20.266 \cdot in$$

The camber is multiplied by 2/3 because the average location of the CG over the length of the beam is 2/3 of the maximum camber

Radius of Stability

$$r := \frac{K_\theta}{w \cdot L} = 87.757\,ft$$

Per Mast Part 2, r is the height at which the total beam weight could be placed to cause neutral equilibrium with the spring for a given small angle

Stress sign convention is tension=positive, compression=negative

Stress at Top of Beam, Tension

$$f_{ttE} := -\frac{P_e}{A_c} + \frac{P_e \cdot e_p}{S_t} - \frac{M_g}{S_t} + \frac{M_{wE.u}}{S_{yt}} = -0.757 \cdot ksi$$

Stress at Top of Beam, Compression

$$f_{tcE} := -\frac{P_e}{A_c} + \frac{P_e \cdot e_p}{S_t} - \frac{M_g}{S_t} - \frac{M_{wE.u}}{S_{yt}} = -0.795 \cdot ksi$$

Compression Check

$$Ck_{E.t.comp} := if\left(f_{tcE} \le 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{tcE} \ge -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Tension Check

$$Ck_{E.t.tens} := if\left(f_{ttE} \le 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{ttE} \ge -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Stress at Bottom of Beam, Tension

$$f_{btE} := -\frac{P_e}{A_c} - \frac{P_e \cdot e_p}{S_b} + \frac{M_g}{S_b} + \frac{M_{wE.u}}{S_{yb}} = -2.781 \cdot ksi$$

Stress at Bottom of Beam, Compression

$$f_{bcE} := -\frac{P_e}{A_c} - \frac{P_e \cdot e_p}{S_b} + \frac{M_g}{S_b} - \frac{M_{wE.u}}{S_{yb}} = -2.81 \cdot ksi$$

Compression Check

$$Ck_{E.b.comp} := if\left(f_{bcE} \le 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{bcE} \ge -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Tension Check

$$Ck_{E.b.tens} := if\left(f_{btE} \le 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{btE} \ge -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Check for stress at girder placement

$$Ck_{stress.plcmnt} := if\left(min\left(Ck_{E.t.comp}, Ck_{E.t.tens}, Ck_{E.b.comp}, Ck_{E.b.tens}\right) = 1, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Roll Stability Check for Girder Placement, prior to beam bracing (Service I, Constructive Active):

Modulus of Rupture

$$f_r := 7.5 \cdot \sqrt{f_{c'}} \cdot \sqrt{psi} = 691.466 \cdot psi$$

Lateral Cracking Moment

$$M_{lat} := \min\left[\frac{(f_r - f_{ttE}) \cdot I_y}{\left(\frac{b_t}{2}\right)}, \frac{(f_r - f_{btE}) \cdot I_y}{\left(\frac{b_b}{2}\right)}\right] = 4897.261 \cdot kip \cdot in$$

Y-direction moment that causes cracking

Rotation Angle at Cracking

$$\theta_{cr} := \frac{M_{lat}}{M_g} = 0.48 \cdot rad$$

ϑ and ϑf are adapted from Mast Part 2 to include effects of erection wind load

Rotation Angle at Failure

$$\theta_f := \min\left[.0001 \cdot \frac{.5 \cdot w \cdot L}{kip} + .038, \frac{5 \cdot z_o \cdot \alpha + \left[(5 \cdot z_o \cdot \alpha)^2 + 10 \cdot z_o \cdot \left(e_s + e_{set} + e_{wE} + \alpha \cdot z_o + 2.5 \cdot e_{wE} \cdot \alpha + y \cdot \alpha + \frac{w_{wE} \cdot h^2}{2 \cdot w}\right)\right]^{.5}}{5 \cdot z_o}\right] = 0.042$$

Final Rotation

$$\theta := \frac{\alpha \cdot r + e_s + e_{set} + e_{wE} + \frac{w_{wE} \cdot h^2}{2 \cdot w}}{r - y - z_o} = 0.0113 \cdot rad$$

Factor of Safety for Cracking (Unbraced Beam)

$$FS_{cr} := \frac{r \cdot (\theta_{cr} - \alpha)}{z_o \cdot \theta_{cr} + e_s + e_{set} + e_{wE} + y \cdot \theta_{cr} + \frac{w_{wE} \cdot h^2}{2 \cdot w}} = 41.8$$

Factor of Safety for Failure (Unbraced Beam)

$$FS_f := \frac{r \cdot (\theta_f - \alpha)}{z_o \cdot (1 + 2.5 \cdot \theta_f) \cdot \theta_f + e_s + e_{set} + e_{wE} \cdot (1 + 2.5 \cdot \theta_f) + y \cdot \theta_f + \frac{w_{wE} \cdot h^2}{2 \cdot w}} = 16.2$$

Factors of safety are adapted from Mast Part 2 to include effects of erection wind load

Maximum Wind Load

$$W_{max.0} := \left[123 \cdot e^{\frac{-L}{100 \cdot ft}} \cdot \left(1 + 15 \cdot e^{\frac{-h}{22 \cdot in}}\right) - 750 \cdot e^{\frac{-h}{16 \cdot in}} - 16\right] \cdot psf$$

Equation 8.2 per Final Report, FDOT Contract BDK75 977-33

Wind Factor of Safety

$$FS_{wind} := \frac{W_{max.0}}{w_{wE}} = 58.172$$

Check for stability at girder placement

$$Ck_{stab.plcmnt} := \text{if}\left[(\theta \geq 0) \wedge (FS_{cr} \geq 1) \wedge (FS_f \geq 1.5) \wedge (FS_{wind} \geq 1.5), \text{"OK"}, \text{"Not OK"}\right] = \text{"OK"}$$

Service Stress Check for braced beam, prior to deck placement (Service I, Construction Inactive):

Sign convention is tension=positive, compression=negative

Stress at Top of Beam, Tension

$$f_{tt} := -\frac{P_e}{A_c} + \frac{P_e \cdot e_p}{S_t} - \frac{M_g}{S_t} + \frac{M_w}{S_{yt}} = -0.743 \cdot ksi$$

Stress at Top of Beam, Compression

$$f_{tc} := -\frac{P_e}{A_c} + \frac{P_e \cdot e_p}{S_t} - \frac{M_g}{S_t} - \frac{M_w}{S_{yt}} = -0.809 \cdot ksi$$

Compression Check

$$Ck_{B.t.comp} := \text{if}\left(f_{tc} \le 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{tc} \ge -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Tension Check

$$Ck_{B.t.tens} := \text{if}\left(f_{tt} \le 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{tt} \ge -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Stress at Bottom of Beam, Tension

$$f_{bt} := -\frac{P_e}{A_c} - \frac{P_e \cdot e_p}{S_b} + \frac{M_g}{S_b} + \frac{M_w}{S_{yb}} = -2.769 \cdot ksi$$

Stress at Bottom of Beam, Compression

$$f_{bc} := -\frac{P_e}{A_c} - \frac{P_e \cdot e_p}{S_b} + \frac{M_g}{S_b} - \frac{M_w}{S_{yb}} = -2.822 \cdot ksi$$

Compression Check

$$Ck_{B.b.comp} := \text{if}\left(f_{bc} \le 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{bc} \ge -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Tension Check

$$Ck_{B.b.tens} := \text{if}\left(f_{bt} \le 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{bt} \ge -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Check for stress at braced condition

$$\boxed{Ck_{stress.braced} := \text{if}\left(\min\left(Ck_{B.t.comp}, Ck_{B.t.tens}, Ck_{B.b.comp}, Ck_{B.b.tens}\right) = 1, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}}$$

Roll Stability Check for braced beam, prior to deck placement (Service I, Construction Inactive):

Initial Rotation

$$\theta_w := \frac{\alpha \cdot r + e_s + e_{set} + \min(e_b, e_w)}{r - y - z_o} = 0.011$$

Rotation Limits

$$\theta_{w.max} := \min(\theta_{cr}, 5 \cdot deg) = 0.087$$

It makes sense to prevent cracking of the beam, as the strength of the beam is compromised once cracking occurs. A reasonable upper bound limit is 5 degrees. Per Mast Part 2, cracking occurs in many beams at 5 degrees.

Wind Load Rotation Check

$$FS_{\theta w} := \frac{\theta_{w.max}}{\theta_w} = 7.724$$

Check for stability at braced condition

$$Ck_{stab.braced} := \text{if}(FS_{\theta w} \geq 1, \text{"OK"}, \text{"Not OK"}) = \text{"OK"}$$

Service Stress Check for braced beam, during deck placement (Service I, Construction Active):

Sign convention is tension=positive, compression=negative

Stress at Top of Beam, Tension

$$f_{ttD} := -\frac{P_e}{A_c} + \frac{P_e \cdot e_p}{S_t} - \frac{M_{gD}}{S_t} + \frac{M_{wE}}{S_{yt}} = -3.851 \cdot ksi$$

Stress at Top of Beam, Compression

$$f_{tcD} := -\frac{P_e}{A_c} + \frac{P_e \cdot e_p}{S_t} - \frac{M_{gD}}{S_t} - \frac{M_{wE}}{S_{yt}} = -3.858 \cdot ksi$$

Compression Check

$$Ck_{D.t.comp} := \text{if}\left(f_{tcD} \leq 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{tcD} \geq -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Tension Check

$$Ck_{D.t.tens} := \text{if}\left(f_{ttD} \leq 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{ttD} \geq -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Stress at Bottom of Beam, Tension

$$f_{btD} := -\frac{P_e}{A_c} - \frac{P_e \cdot e_p}{S_b} + \frac{M_{gD}}{S_b} + \frac{M_{wE}}{S_{yb}} = -0.191 \cdot ksi$$

Stress at Bottom of Beam, Compression

$$f_{bcD} := -\frac{P_e}{A_c} - \frac{P_e \cdot e_p}{S_b} + \frac{M_{gD}}{S_b} - \frac{M_{wE}}{S_{yb}} = -0.196 \cdot ksi$$

Compression Check

$$Ck_{D.b.comp} := \text{if}\left(f_{bcD} \leq 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{bcD} \geq -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Tension Check

$$Ck_{D.b.tens} := \text{if}\left(f_{btD} \leq 6 \cdot \sqrt{\frac{f_{c'}}{psi}} \cdot psi \wedge f_{btD} \geq -0.6 \cdot f_{c'}, 1, 0\right) = 1$$

Check for stress at deck placement condition

$$Ck_{stress.deck} := \text{if}(\min(Ck_{D.t.comp}, Ck_{D.t.tens}, Ck_{D.b.comp}, Ck_{D.b.tens}) = 1, \text{"OK"}, \text{"Not OK"}) = \text{"OK"}$$

Roll Stability Check during Deck Placement (Strength I):

Lateral Cracking Moment

$$M_{latD} := \min\left[\frac{\left(f_r - f_{ttD} + \frac{M_{wE}}{S_{yt}}\right)\cdot I_y}{.5\cdot b_t}, \frac{\left(f_r - f_{btD} + \frac{M_{wE}}{S_{yb}}\right)\cdot I_y}{.5\cdot b_b}\right] = 3778.651\cdot kip\cdot in$$

Y-direction moment that causes cracking

Rotation Angle at Cracking

$$\theta_{crD} := \frac{M_{latD}}{M_{gD}} = 0.125\cdot rad$$

Initial Rotation

$$\theta_{i.D} := \frac{\alpha\cdot r + e_s + e_{set}}{r - y - z_o} = 0.011$$

The initial rotation is caused by the imperfections in the girder and girder support. Additionally, it can be expected that the construction loads will cause the maximum "play" in the bracing to be achieved, which results in an eccentricity of eb. The initial rotation is the maximum rotation that is seen at the bracing points. Any additional rotation is between the bracing points in the form of torque. The torque is caused by the construction live loads acting on the overhang of the bridge, eccentric to the centerline of the exterior girder.

Torque due to construction live loads

$$T_D := \max\left(\left|M_{fm}\right|, \left|.5M_c\cdot L_b\right|, \left|M_{fm} + .5M_c\cdot L_b\right|\right) = 95.155\cdot kip\cdot ft$$

Twist due to construction live loads

$$\phi_D := \frac{T_D\cdot\left(.5\cdot L_b\right)^2}{G_{shear}\cdot J\cdot L_b} = 0.0027$$

Deflection at cantilever due to twist

$$\delta_D := OH\cdot\tan\left(\phi_D\right) = 0.147\cdot in$$

Total Rotation

$$\theta_D := \theta_{i.D} + \phi_D = 0.014$$

Rotation Limits

$$\theta_{D.max} := \min\left(\theta_{crD}, 5\cdot deg\right) = 0.087$$

Deck Placement Rotation Check

$$\boxed{Ck_{stab.deck} := \text{if}\left(\delta_D \le \delta_{max} \wedge \theta_D \le \theta_{D.max}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}}$$

Required Brace Stiffness

Anchor Brace

$$k_{ra} := \left[\frac{1 - 39\cdot e^{\frac{-L}{48\cdot ft}} + \frac{\sqrt{\frac{w_w}{2\cdot psf}}}{37000}\cdot\left[\left(\frac{L}{ft}\right)^2 - \frac{405\cdot L}{ft} + 50000\right] + \frac{h\cdot w_w\cdot plf}{48\cdot w\cdot in\cdot psf}}{\left(\frac{100 - \frac{h}{in}}{n_{beam}\cdot 125000} + \frac{\sqrt{\frac{w_w}{2\cdot psf}}\cdot .01\cdot L}{37000\cdot n_{beam}\cdot ft}\right)}\right]\cdot\frac{kip\cdot ft}{rad} = -5.169\times 10^4\cdot\frac{kip\cdot ft}{rad}$$

$$\omega := \text{if}\left(n_b = 0, 1, \text{if}\left(n_b = 1, 1.4, \text{if}\left(n_b = 2, 1.6, 1.7\right)\right)\right) = 1.4$$

Moment-resisting brace

$$a_{br} := \frac{\left(1 - .004\cdot\frac{L}{ft}\right)\cdot\sqrt{\frac{w_w}{2\cdot psf}}}{1000000} = 1.789\times 10^{-6}$$

$$b_{br} := \omega\cdot 620\cdot e^{\frac{-L}{30\cdot ft}} + 39\cdot e^{\frac{-L}{48\cdot ft}} - \left(8\times 10^{-6}\right)\cdot\left(\frac{L}{ft}\right)^2\cdot\sqrt{\frac{w_w}{2\cdot psf}} + .0011\cdot\frac{L}{ft}\cdot\sqrt{\frac{w_w}{2\cdot psf}} + .1\cdot\sqrt{\frac{w_w}{2\cdot psf}} - \frac{h\cdot w_w\cdot plf}{48\cdot w\cdot in\cdot psf} - 1 = 48.557$$

$$c_{br} := 39000000\cdot e^{\frac{-L}{48\cdot ft}} - 1000000 - 8\cdot\left(\frac{L}{ft}\right)^2\cdot\sqrt{\frac{w_w}{2\cdot psf}} + 5100\cdot\frac{L}{ft}\cdot\sqrt{\frac{w_w}{2\cdot psf}} - 900000\cdot\sqrt{\frac{w_w}{2\cdot psf}} - \frac{20833.33\cdot h\cdot w_w\cdot plf}{w\cdot in\cdot psf} = 3.553\times 10^6$$

$$k_{br} := \max\left(\frac{-b_{br} + \sqrt{b_{br}^2 - 4\cdot a_{br}\cdot c_{br}}}{2\cdot a_{br}}, \frac{-b_{br} - \sqrt{b_{br}^2 - 4\cdot a_{br}\cdot c_{br}}}{2\cdot a_{br}}\right)\cdot\frac{kip\cdot ft}{rad} = -7.337\times 10^4\cdot\frac{kip\cdot ft}{rad}$$

Wind Capacity of Single Anchored Girder

$$W_{max} := \frac{W_{max.0} + 11\cdot e^{\frac{-L}{22\cdot ft}}\cdot\frac{k_{ra}}{\left(\frac{kip\cdot ft}{rad}\right)}\cdot psf}{1.5} = -6.273\times 10^3\cdot psf$$

$$V_{max} := \min\left(\sqrt{\frac{W_{max}}{0.00256\cdot K_z\cdot G\cdot C_{pg}\cdot psf}}\cdot mph, V\right) = 1.202i\times 10^3\cdot mph$$

Bracing Requirements:

Factored Horizontal Force at Each Beam End and Anchor Brace, at midheight of beam

$$F_e := w_w \cdot \gamma \cdot h \cdot L_b \cdot K_{ve} \cdot \frac{C_{pbr}}{C_{pg}} = 0.12 \cdot kip$$

(Strength III, Construction Inactive)

Factored Horizontal Bracing Force at Each Intermediate SpanBrace (if present), at mid-height of beam

$$F_i := if\left(n_b = 0, \text{"N/A"}, w_w \cdot \gamma \cdot h \cdot L_b \cdot K_{vi} \cdot \frac{C_{pbr}}{C_{pg}}\right) = 1.467 \cdot kip$$

Factored Overturning Force at Each Beam End and Anchor Brace, at top of beam

$$M_e := max\left(\left|1.0 \cdot M_{fm}\right|, \left|M_c \cdot L_b \cdot K_{ve}\right|, \left|1.0 \cdot M_{fm} + M_c \cdot L_b \cdot K_{ve}\right|\right) = 74.23 \cdot kip \cdot ft$$

(Strength I)

Factored Overturning Force at Each Intermediate Span Brace (if present), at top of beam

$$M_i := if\left(n_b = 0, \text{"N/A"}, max\left(\left|1.0 \cdot M_{fm}\right|, \left|M_c \cdot L_b \cdot K_{vi}\right|, \left|1.0 \cdot M_{fm} + M_c \cdot L_b \cdot K_{vi}\right|\right)\right) = 126.639 \cdot kip \cdot ft$$

Note: The distribution of the finishing machine in the equations for overturning force has conservatively been set to 1.0. Finite element analysis indicates a lower distribution factor may be appropriate and a parametric study for various beam types and relative bracing stiffness is in progress. In the meantime, designers may adjust the distribution as they see fit.

Summary of Results

Verification of Bracing Adequacy

Stress Checks

$Ck_{stress.plcmnt}$ = "OK"

$Ck_{stress.braced}$ = "OK"

$Ck_{stress.deck}$ = "OK"

Stability Checks

$Ck_{stab.plcmnt}$ = "OK"

If Ckstab.plcmnt is "Not OK," the girder must be braced prior to crane release.

$Ck_{stab.braced}$ = "OK"

$Ck_{stab.deck}$ = "OK"

Temporary Bracing Variables

Maximum Unbraced Length	$L_b = 45\,ft$
Factored Horizontal Force at Each Beam End and Anchor Brace, at mid-height of beam	$F_e = 0.12 \cdot kip$
Factored Horizontal Bracing Force at Each Intermediate SpanBrace (if present), at mid-height of beam	$F_i = 1.467 \cdot kip$
Factored Overturning Force at Each Beam End and Anchor Brace, at top of beam	$M_e = 74.23 \cdot kip \cdot ft$
Factored Overturning Force at Each Intermediate Span Brace (if present), at top of beam	$M_i = 126.639 \cdot kip \cdot ft$

Wind Load Variables

Basic Wind Speed	$V_B = 100 \cdot mph$
Construction Inactive Wind Speed	$V = 60 \cdot mph$
Construction Active Wind Speed	$V_E = 20 \cdot mph$
Velocity Pressure Exposure Coefficient	$K_z = 0.907$
Gust effect factor	$G = 0.85$

Assumed Construction Loads

Weight of build-up	$w_b = 50 \cdot plf$
Form Weight	$w_f = 20 \cdot psf$
Finishing Machine Total Weight	$w_{fm} = 16 \cdot kip$
Finishing Machine Wheel Location Beyond Edge of Deck Overhang	$d_{fm} = 2.5 \cdot in$
Deck Weight	$w_d = 106.25 \cdot psf$
Live load	$w_l = 20 \cdot psf$
Live Load at Extreme Deck Edge	$P_l = 75 \cdot plf$

Required Brace Stiffness

Required Roll Anchor Stiffness

$$k_{ra} = -5.169 \times 10^4 \cdot \frac{kip \cdot ft}{rad}$$

*Typical effective anchor bracing stiffnesses vary from 500 to 1600 kip*ft/rad. Multiple anchor braces may be used to increase the system anchor capacity.*

Required Brace Stiffness for Moment-Resisting Brace System

$$k_{br} = -7.337 \times 10^4 \cdot \frac{kip \cdot ft}{rad}$$

*Typical moment-resisting bracing stiffnesses vary from 15,000 to 600,000 kip*ft/rad.*