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Structural Impact of Construction Loads

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JOINT TRANSPORTATION RESEARCH PROGRAM

INDIANA DEPARTMENT OF TRANSPORTATION
AND PURDUE UNIVERSITY

STRUCTURAL IMPACT OF CONSTRUCTION LOADS

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JOINT TRANSPORTATION RESEARCH PROGRAM

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16. Abstract

Numerous bridge construction accidents have occurred across the country because of construction loadings, which are an under-emphasized topic in many DOT specifications and design manuals. Bridge girders are least stable when they are subjected to construction loads, so it is important for design engineers and contractors to understand and design for these loads. The Indiana Department of Transportation's current Standard Specifications does not contain many construction load provisions, so this study was performed to identify and implement new requirements to proactively prevent accidents from occurring in Indiana.

Various documents were examined in this study, including AASHTO and ASCE standards in addition to several other states DOT Standard Specifications and Design Manuals. Based on these documents, new falsework and formwork design loads, including horizontal loads, impact loads, and wind load, were developed and proposed to INDOT, which currently only specifys a dead load and live load. A set of drawings showing proposed minimum lateral bracing requirements was also developed to help ensure the stability of prestressed concrete and steel girders during construction.

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EXECUTIVE SUMMARY

STRUCTURAL IMPACT OF CONSTRUCTION LOADS

Introduction

Numerous bridge construction accidents have occurred because of construction loadings, which are an underemphasized topic in many specifications and design manuals. Bridge girders are least stable during the construction phase, so it is important for bridge designers and contractors to understand and design for conditions during this phase. The Indiana Department of Transportation's current *Standard Specifications* contain limited construction load provisions and temporary bracing requirements; therefore this study was performed to identify and implement new requirements to proactively prevent construction accidents from occurring in Indiana.

Various documents were examined in this study, including AASHTO and ASCE standards and the standard specifications and design manuals of other departments of transportation. Based on these documents, new falsework and formwork design loads, including horizontal loads, impact loads, and wind load, were developed and proposed. INDOT currently specifies only construction dead load and live loads. A set of drawings showing proposed minimum lateral bracing requirements was also created to help ensure the stability of prestressed concrete and steel girders during construction.

Findings

To develop new proposed falsework and formwork design loads, the results of the literature review were studied and then discussed with a committee consisting of INDOT bridge engineers, consulting engineers, and Indiana contractors in a series of meetings. For various construction load types, appropriate specification language was developed, and if needed, analysis performed. For some of the design loads, it was agreed to simply adopt the corresponding provision in the *AASHTO Guide Design Specifications for Bridge Temporary Works*.

The following provisions are recommended for inclusion in INDOT's *Standard Specifications*:

- A dead load of 150 pcf for concrete and reinforcing steel, plus 15 psf for formwork, consistent with a current INDOT design memo.
- A live load consisting of known construction loads, a 20 psf uniform load, and a 75 plf load at overhangs, also consistent with the current design memo.
- A minimum vertical load of 100 psf and a minimum horizontal load equal to 2% of the dead load.
- An impact load requirement for falsework, potentially affected by placement or lifting operations, and of any falsework over or adjacent to traffic.
- A new wind load provision, including tables that provide a wind pressure table and a reduction factor.
- Minimum bracing requirements for prestressed and steel girders. Proposed standard drawings would require girders to be adequately braced during erection and before slab placement.

Implementation

- The new design load requirements are proposed additions to the appropriate sections of INDOT's *Standard Specifications* and *Design Manual*. Provisions would require falsework and formwork to be designed for dead load, live load, impact load, wind load, and minimum horizontal and vertical loads.
- The minimum bracing requirements for prestressed concrete and steel girders are proposed as standard drawings. Some provisions in the notes section of the drawings would be appropriate for inclusion in the *Standard Specifications*.
- Use of the bracing requirements would not relieve the contractor of responsibility for the adequacy of the bracing system and the safety of the structure.
- It is recommended that bracing systems must be designed by an engineer according to the minimum bracing requirements, and proposed details must be submitted to INDOT for review prior to erection.

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1. INTRODUCTION

In the past few decades, there have been several significant bridge construction accidents across the country caused by construction loads. Bridges are designed to carry a full design vehicle loading when they are completed, but not enough attention is given to their strength and stability during construction. Gaining a better understanding of the construction loads that act on a structure could help prevent future accidents. Many past failures have been caused by inadequately designed falsework, formwork, or bracing systems, and some departments of transportation have reacted by reviewing the construction requirements in their standard specifications, construction manuals and other related documents.

In Maryland in 1989, a Route 198 bridge under construction collapsed during deck placement, spilling wet concrete onto the road below. The failure was later attributed to inadequate shoring towers (*1*). Shortly after the accident, the Federal Highway Administration (FHWA) began studying ways to improve specifications used in designing temporary shoring in bridge construction. The American Association of State Highway and Transportation Officials (AASHTO) later used some of the FHWA's results in developing the *Guide Design Specifications for Bridge Temporary Works* (*1*).

In Colorado, during a construction project on a C-470 overpass in 2004, insufficient temporary bracing allowed a steel girder to crash down onto I-70. The fall, caused by a gust of wind, crushed a vehicle travelling on I-70 and killed a family of three (*2*). Figure 1.1 shows the collapsed girder and the remains of the vehicle. Another accident occurred in 2005 in Lansing, Illinois, when a set of steel girders laterally buckled and fell off its piers, killing one worker (*3*).

A 1986 study published in the *ASCE Journal of Construction Engineering and Management* researched the causes of dozens of falsework failures and concluded that 49% of the accidents happened during concrete pouring (*4*). The bar chart shown in Figure 1.2 illustrates the frequency of falsework failures examined in 85 cases reviewed in the study. Failures during concrete placement are four times more frequent than the next leading cause of falsework failure. This paper quantified the magnitude of this problem and even with this awareness these types of failures continue to frequently occur.

The Indiana Department of Transportation (INDOT) specifications (*5*) are currently sparse in their requirements for falsework, formwork, and bracing. It is important for INDOT to be proactive in improving its standards to minimize the risk of these types of accidents. This study was performed to find ways to improve the state's construction requirements to help prevent future collapses and failures.

A committee was formed consisting of INDOT personnel, design consultants and contractors to provide input to the project. With their direction, the focus of the project was narrowed down to two main topics that needed to be addressed to improve bridge construction safety in Indiana: falsework and formwork design loads, and temporary bracing.

INDOT's *Standard Specifications* (*5*) currently contain very little information about falsework and formwork design. Other states have expanded their requirements to define several different types of construction loads, while Indiana only specifies a dead load and a live load for construction. This study sought to evaluate other types of construction loads and incorporate them into the state's design requirements.

Figure 1.1 Collapsed C-470 girder in 2004 (*2*).

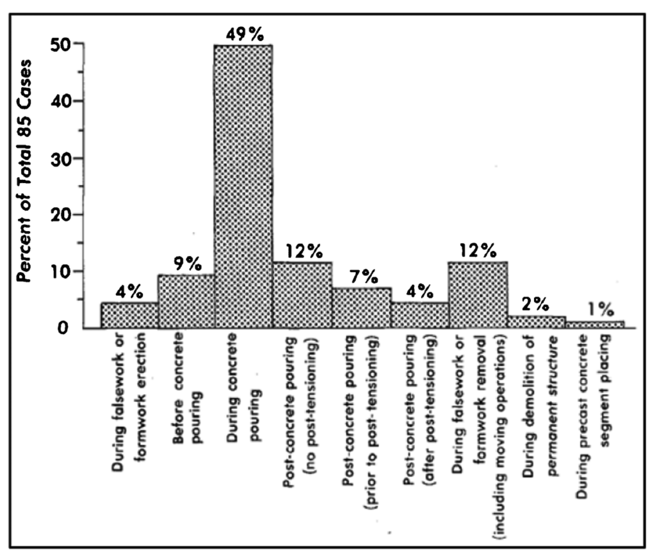

Figure 1.2 Falsework collapse by construction stage (*4*).

The state can also be proactive by improving its temporary bracing requirements of bridge girders during construction. There are currently no provisions for construction lateral bracing or details.

Although new, more stringent temporary structure requirements may lead to more time-consuming, and therefore more expensive construction requirements, the benefits of implementing new provisions far outweigh the costs. The prevention of future accidents is invaluable. Bridge collapses can cost millions of dollars in site cleanup, materials, equipment, lost time, and sometimes lead to lawsuits, injuries, or loss of life. An article from Minnesota Public Radio in 2007 estimated that the total cost of the 2005 Minneapolis I-35 collapse, which occurred during deck reconstruction, was nearly $400 million (*6*).

2. DESIGN LOAD RECOMMENDATIONS

2.1 Introduction

INDOT's current *Standard Specifications* (*5*) has limited requirements on design loads for falsework and formwork. Sections 702.13 and 702.14, covering forms and falsework, prescribe only a dead load due to the weight of the concrete and a construction live load.

In comparison with many other states' transportation agency standard specifications, and with American Association of State Highway Transportation Officials (AASHTO) and American Society of Civil Engineering (ASCE) documents, more construction load requirements need to be developed and included in the INDOT *Standard Specifications* in order to mitigate apparent shortcomings.

The standard specifications from 16 different Departments of Transportation were examined: Arizona, California, Colorado, Florida, Idaho, Illinois, Kentucky, Maryland, Michigan, Minnesota, New York, Ohio, Pennsylvania, Texas, Washington, and Wisconsin (*7–22*). The *AASHTO Guide Design Specifications for Bridge Temporary Works* (*23,24*) and *ASCE 37-02: Design Loads on Structures During Construction* (*25*) were also reviewed.

AASHTO Guide Design Specifications for Bridge Temporary Works, a 1995 document with 2008 revisions (*23,24*), provides the most comprehensive design load requirements. In discussions with the aforementioned committee, it was decided that many of the construction design loads in this document should be considered for adoption into INDOT *Standard Specifications*, with some exceptions, as noted in the following sections.

2.2 Dead Load

INDOT's current construction dead load requirement is 150 pounds per cubic foot (pcf) for vertical loads due to the weight of the concrete. This value is consistent with other agencies studied. Some agencies require 150 pcf for the density of the concrete and reinforcing steel, while others require 160 pcf, which includes the concrete, reinforcing steel, and forms. *AASHTO Bridge Temporary Works* (*23,24*) uses the latter requirement.

INDOT's *Design Memorandum No. 10-18* (*26*) prescribes several construction loads to be used in the design of bridges. Though the design of a bridge often differs from the design of its falsework, formwork, and bracing, the dead load requirement applies to both. It is recommended that the dead load be kept consistent between falsework and bridge designs. The memorandum lists a formwork weight of 15 psf "for permanent metal stay-in-place deck forms, removable deck forms, and 2-ft exterior walkway," to be added as a dead load to the weight of the concrete and reinforcing steel. This language is appropriate for inclusion into the *Standard Specifications*.

AASHTO Bridge Temporary Works (*23,24*) lists a lightweight concrete design weight of 130 pcf. However, since lightweight concrete unit weight can vary significantly among different projects, it is recommended that INDOT's *Standard Specifications* require that falsework and formwork shall not be designed with a reduced design weight for lightweight concrete. Using the normal-weight concrete design value of 150 pcf is acceptable when using lightweight concrete. It would be appropriate for a reduced design weight for lightweight concrete to be used on a particular project, if approved by INDOT.

The following requirement for construction dead load is recommended for adoption into INDOT's *Standard Specifications* (*5*):

> The combined density of concrete and reinforcing and prestressing steel shall be assumed to be not less than 150 pcf for normal-weight concrete and for lightweight concrete. Exceptions to the lightweight concrete design weight can be requested for approval from INDOT.
> The weight of formwork shall be assumed to be not less than 15 psf, including permanent metal stay-in-place deck forms, removable deck forms, and a 2-foot exterior walkway.

2.3 Live Load

INDOT's *Standard Specifications* (*5*) currently requires a live load of 50 psf on horizontal projections of surfaces for falsework and formwork design. This requirement differs from other agencies. Several state transportation agencies list the same 50 psf as Indiana for both falsework and formwork. Others, however, list a combination of point loads representing known loads, plus a 20 psf uniform load, plus a 75 plf linear load at overhangs, for falsework design, while using the 50 psf value for formwork design. INDOT *Design Memorandum No. 10-18* (*26*) uses the combination of point loads, 20 psf uniform load and 75 plf linear load, with more clarification about the location of these loads. It is recommended that the *Standard Specifications* be kept consistent with the memorandum.

An analysis was conducted in order to determine how much more conservative the proposed new requirement would be compared to the old requirement of 50 psf for both falsework and formwork. A design example found in Chapter 5 of the 2009 Kansas Department of Transportation (KDOT) *Design Manual* (*27*) shows a thorough design of a typical falsework and formwork system for a steel girder bridge. The example was used to evaluate falsework using both the old 50 psf requirement and the new proposed requirement. More information on this example and the calculations performed can be found in Appendix A. It was found that the new requirement led to a design about 10% less conservative – a significant but not extreme amount. This validates the proposed values as appropriate for adoption into INDOT's specifications.

Using the requirements in *AASHTO Bridge Temporary Works* (*23,24*) and *Design Memorandum 10-18* (*26*), the following requirement for construction live load on falsework is recommended for inclusion in the *Standard Specifications*:

> The construction live load shall consist of:
>
> - The actual weight of any equipment to be supported, applied as concentrated loads.
> - A uniform load of 20 psf, applied over the area supported and extending 2 feet past the edge of coping.
> - A 75 plf vertical force, applied at a distance of 6 inches outside the face of coping over a 30-foot length of the deck centered with the finishing machine.

It is recommended that this requirement apply only to falsework design, while the 50 psf uniform load should be used for formwork design.

2.4 Combined Dead and Live Load

AASHTO Bridge Temporary Works (*23,24*) contains a minimum design vertical load to be applied regardless of slab thickness. The value would only govern in designs with unusually thin slabs. The requirement is appropriate for INDOT to adopt:

> The minimum total design vertical load for any falsework member shall be not less than 100 psf for the combined dead and live load, exclusive of any increase for impact, regardless of slab thickness.

2.5 Impact Load

Some state's specifications require an impact load, which helps ensure a bridge's stability should an accident occur during construction. There were two types of impact loads found in the literature search – a design provision in *AASHTO Bridge Temporary Works* (*23,24*) accounting for lifting and placement operations, and a requirement in a few states' specifications for vehicular impact on falsework.

AASHTO Bridge Temporary Works (*23,24*) requires an increased design load for any members subject to placement or lifting operations. It states:

> For members subject to impact during placement operations, the design dead load must be increased by at least 30 percent of the weight of the material being placed.
>
> For members subject to impact during lifting operations, the static load due to the payload must be increased by at least 30 percent.

It is recommended that INDOT adopt this requirement from *AASHTO*. Accounting for an increased design load to include impact is consistent with the large number of failures that occur during concrete placement. As stated in the Introduction, the 1986 study *Analysis of Causes of Falsework Failures in Concrete Structures* considered over 85 failures related to falsework, and found that 49% of them occurred during the concrete placement phase (*4*).

A vehicle impact provision is contained in some states' specifications. It applies to the design of any falsework potentially subjected to collision, and lists minimum design requirements for any such members. Designs adhering to this section would help ensure the stability of falsework over or adjacent to traffic, reducing the potential for collapse should vehicular impact occur. It is recommended that Indiana adopt the following language:

> Falsework over or adjacent to roadways or railroads which are open to traffic shall be designed and constructed so that the falsework will be stable if subjected to impact by vehicles. Falsework posts which support members that cross over a roadway or railroad shall be considered as adjacent to roadways or railroads. Other falsework posts shall be considered as adjacent to roadways or railroads only if they are located in the row of falsework posts nearest to the roadway or railroad, and the horizontal distance from the traffic side of the falsework to the edge of pavement or to a point 10 feet from the centerline of track is less than the total height of the falsework and forms.
>
> Falsework shall not be considered adjacent to roadways or railroads if it is protected from traffic by an approved barrier. If not properly protected, the appropriate loads are as follows:
>
> - Falsework posts adjacent to roadways or railroads shall consist of either steel with a minimum section modulus about each axis of 9.5 inches cubed or sound timbers with a minimum section modulus about each axis of 250 inches cubed.
> - Each falsework post adjacent to roadways or railroads shall be mechanically connected to its supporting footing at its base, or otherwise laterally restrained, so as to withstand a force of not less than 2,000 pounds applied at the base of the post in any direction except toward the roadway or railroad track. The posts also shall be mechanically connected to the falsework cap or stringer. The mechanical connection shall be capable of resisting a load in any horizontal direction of not less than 1,000 pounds.

2.6 Horizontal Load

In the design of temporary structures, horizontal loads occur because of wind load and horizontal construction loads. Many documents contain the following provision that gives a minimum horizontal load, not including the wind load. The section is appropriate for adoption in Indiana:

> The horizontal design load shall consist of the sum of any actual horizontal loads due to equipment, construction sequence, or other causes, excluding the specified wind load, but in no case shall the horizontal design load be less than 2 percent of the total dead load to be supported at the point under consideration.

2.7 Wind Load

INDOT's *Standard Specifications* do not contain a wind load requirement for falsework and formwork design. *AASHTO Bridge Temporary Works* (*23,24*) and the specifications from California and Washington (*8,21*) are relatively progressive and contain simplified tables listing design wind pressure values depending on the height zone of the falsework. A similar table for INDOT was developed and is recommended for adoption.

The design wind load was developed based on the method prescribed in *ASCE 7-10* (*28*). A 115-mph design wind speed was used to find appropriate wind pressure values. A wind pressure reduction factor was adapted from *ASCE 37-02* (*25*) and is also recommended to account for the low probability of a 50-year design wind speed occurring in a construction period of no more than 5 years. A thorough explanation of the development is included in Appendix B. The resulting requirements, recommended for adoption in Indiana, are as follows:

> The minimum horizontal load to be allowed for wind on falsework is dependent on the Exposure category of the falsework. Falsework shall be assigned one of the following Exposure categories:
>
> - Exposure B: Has terrain with buildings, forest, or surface irregularities 20 ft or more in height covering at least 20 percent of the area extending 1 mile or more from the site.
> - Exposure C: Has terrain which is flat and generally open, extending ½ mile or more from the site in any full quadrant.
> - Exposure D: Represents the most severe exposure in areas with basic wind speeds of 80 miles per hour (mph) or greater and has terrain which is flat and unobstructed facing large bodies of water over one mile or more in width relative to any quadrant of the construction site.
>
> The following method for calculating design wind loads only applies for the design of falsework categorized as Exposure B or C, and with a height no more than 75 feet above the ground. Design wind loads on falsework categorized as Exposure D, or on falsework taller than 75 feet, shall be calculated according to *ASCE 7-10 – Minimum Design Loads for Buildings and Other Structures*, or the most recent version of *ASCE 7*.

TABLE 2.1
Design Wind Pressure Values

Height Above Ground (ft)	Wind Pressure (psf) Typical Falsework	Wind Pressure (psf) Falsework over or Adjacent to Traffic Openings
0–25	30	35
25–50	35	40
50–75	40	45

TABLE 2.2
Design Wind Pressure Reduction Factor

Construction Period	Wind Pressure Reduction Factor
Less than 6 weeks	0.57
6 weeks to 1 year	0.64
1 to 2 years	0.73
2 to 5 years	0.81

> *Wind Load Procedure for Falsework Categorized as Exposure B or C:*
>
> The minimum horizontal load to be allowed for wind on all types of falsework shall be the sum of the products of the wind impact area, the applicable wind pressure value for each height zone, and the wind pressure reduction factor.
>
> *Wind Impact Area:*
>
> For unenclosed falsework frames and shoring towers, the wind impact area on each applicable face is the total projected area of all the elements in the falsework normal to the direction of the applied wind. Wind impact shall be considered on all faces normal to the direction of the applied wind.
>
> For enclosed frames or towers, the wind impact area shall be the gross projected area of the falsework and any unrestrained portion of the permanent structure, excluding the areas between falsework posts or towers where diagonal bracing is not used. Wind impact shall be considered on the first face normal to the direction of the applied wind. Wind impact shall also be considered to act on the sides of enclosed towers, applied perpendicular to the direction of the wind load and outward from the tower, multiplied by a factor of 0.60.
>
> *Wind Pressure Values:*
>
> Wind pressure values shall be determined from [Table 2.1].
>
> *Wind Pressure Reduction Factor:*
>
> The wind pressure reduction factor, as shown in [Table 2.2], shall be applied based on the length of the construction period. The construction period shall be taken as the time interval between the first and last use of falsework on the project."

Appendix B contains an explanation of the 0.60 factor and a figure clarifying its application.

3. TEMPORARY BRACING RECOMMENDATIONS

3.1 Introduction

As a result of committee meetings, it was decided that temporary bracing regulations should be developed for INDOT, since several contractors expressed a desire for more guidance in their use of temporary lateral bracing on girders.

Information from two different states in particular was used in developing new requirements for Indiana: Texas, which has a set of drawings for its *Minimum Erection and Bracing Requirements* (*29*), and Florida, which recently updated its *Structures Design Guidelines* (*30*) with some temporary bracing requirements.

According to FDOT's guidelines, there are three different phases in a bridge's construction that are critical to the stability of its girders (*30*):

1. Girder Placement – Girder is placed on its bearing pads and sits in place unbraced; loads include self-weight and wind load.
2. Braced Girder – Braced beam sits on bearing pads; loads include self-weight and wind load.
3. Deck Placement – Deck is cast, but not yet hardened, over braced girders; loads include beam self-weight, wind load, and construction loads including the weight of the deck.

These three cases were carefully considered in the development of new bracing requirements. The deck placement phase is often the most critical stage, since the beams must carry the weight of the deck, but the deck has not yet gained sufficient stiffness to provide lateral stability to the beams.

The objective of this portion of the study was to produce bracing drawings containing minimum bracing requirements that will provide more guidance to contractors on how to use temporary bracing. One drawing was developed for prestressed concrete girders, and one for steel girders. The drawings were based partly on Texas' *Minimum Erection and Bracing Requirements* (*29*), partly on analysis verifying the numbers found in those drawings, and largely on discussions with the committee about the best and most practical bracing methods. The resulting drawings can be found in Appendix C (prestressed girders) and Appendix D (steel girders).

The proposed requirements are only intended for guidance, and are not intended to relieve the contractor of responsibility for the adequacy of the bracing systems.

3.2 Temporary Bracing of Prestressed Concrete Girders

In order to ensure the stability of girders at placement, it was decided that each beam must be braced before it is released from the crane used to place it, eliminating the possibility of an unbraced beam falling after it is placed. The first girder placed in a span should be braced to the bent with "anchor" bracing, which may use various combinations of tension and compression members to secure the girder. Figure C.2 (in Appendix C) was created, based on a similar drawing in FDOT's *Structures Design Bulletin 10-01* (*31*), to show some acceptable forms of anchor bracing. The second girder in the span must be braced to the anchored beam in at least one location before it is released from its crane, using either cross bracing or horizontal bracing, whichever is required. Each subsequent girder must be braced to another secure girder before it is released from the crane. Ideally, each girder would be secured with all braces before it is released, but due to lane closure restrictions on the under-passing roadway, such practice is often not feasible. Instead, at least one brace – the one closest to midspan – is required to be in place, and the rest of the braces should be installed as soon as practical to fully secure the girder.

The drawings show some acceptable forms of cross bracing and horizontal bracing (again based on FDOT *Structures Design Bulletin 10-01* (*31*)). In TxDOT's *Minimum Erection and Bracing Requirements* (*30*), cross bracing is required in exterior bays, in every fourth bay, and between the first two girders erected, and horizontal bracing is considered adequate in other locations. Because cross bracing provides more stiffness than horizontal bracing, it was agreed that this requirement from TxDOT was appropriate, and is recommended for INDOT. It was also agreed that it would be acceptable for the contractor to simply install permanent diaphragms in place of the temporary braces.

In TxDOT's *Minimum Erection and Bracing Requirements* (*30*), the anchor bracing, cross bracing, and horizontal bracing are all considered part of the erection bracing system that stabilizes the span between girder placement and the deck placement. Before the casting occurs, though, the deck placement bracing system should be satisfied. Since the deck placement construction phase is often the most critical to the stability of the beams, deck placement bracing requires more bracing than the erection bracing system. After discussions with the committee, however, it was decided that such bracing is sometimes unnecessary, and should be used if required by design but not required for all cases. When extra bracing is necessary for deck placement, it is suggested that permanent diaphragms should be installed before deck placement, and additional cross bracing should be used in exterior bays of the span. Additional cross bracing, spaced halfway between the permanent diaphragms, would help resist rotation in the exterior beams; thus it is suggested as part of the *Minimum Bracing Requirement* drawings.

3.3 Analysis of Unbraced Lengths

TxDOT's *Minimum Erection and Bracing Requirements* (*30*) specifies a 60′ maximum spacing between temporary braces along the length of the beam. It was decided that it would be beneficial for INDOT to prescribe a similar spacing limit. An unbraced length of 50′ was selected and analyzed to ensure its validity.

The analysis was performed using a Mathcad 14 program released by FDOT titled "Concrete I-Girder Beam Stability Program" (*32*) that checks the stability of a girder at each of the three critical phases – girder placement, braced girder, and deck placement. At each phase, the program checks the stresses at midspan and at the ends of the beam due to the construction loads and prestressing forces. It then checks the roll stability of the beam according to the method detailed in Robert Mast's paper *Lateral Stability of Long Prestressed Concrete Beams* (1989 and 1993) (*33,34*). As mentioned in the Literature Survey, Mast argues that prestressed beams are torsionally rigid, and their stability is controlled by the ability of the bearing pad to rotationally resist the beam's lateral deflection. The program follows Mast's equations to calculate the beam's rotation and check it against cracking and fracture limits. The program was slightly modified in this study to incorporate the new proposed wind load for Indiana, which is different from that in Florida. The program is attached in Appendix E.

An unbraced length of 50 feet was tested for a wide range of AASHTO standard prestressed girders. The girders were designed in a program titled "PSBeam," which allows the user to design a prestressed girder, including the strand patterns, as part of a bridge span. Each AASHTO beam was designed in PSBeam in order to obtain a working prestress force and strand eccentricity to input into the FDOT Stability program. Appendix F shows the output for the 72″ AASHTO VI girder.

Five different girder sizes were tested, each with a 50′ unbraced length, and each passed the design checks in the stability program. Table 3.1 summarizes the results.

Based on the positive results of the analysis, it is concluded that 50 feet is a reasonable limit for maximum temporary brace spacing. The limit does not mean that 50 feet is an adequate spacing on every bridge – each project must have its bracing system designed, and the bracing must satisfy strength and stability conditions. The proposed requirement is that temporary bracing be spaced as required by design, but not to exceed 50' in any case. It would provide a conservative limit to ensure stability.

3.4 Temporary Bracing of Steel Girders

The development of steel bracing regulations was approached in much the same way as the prestressed regulations. The same provision was agreed upon that requires each girder to be braced before it is released from the crane, and for the first beam to be connected to the bent using anchor bracing. The proposed steel requirements, like prestressed, require an erection bracing system when the girders are placed, and if required by design, specifies a slab placement bracing system that should be in place before the slab is cast. The erection bracing system for steel girders, though, requires the permanent diaphragms to be installed rather than temporary braces. For slab placement bracing, it is recommended that additional intermediate braces be placed in exterior bays to help prevent rotation. Similar to the proposed prestressed requirements, it is suggested that if needed, these intermediate braces should be spaced halfway between permanent diaphragms.

TxDOT requires that all curved girders be secured at both ends with anchor bracing. Since curved girders are significantly less stable than straight girders, it was agreed that this provision should be adopted in Indiana. The committee also suggested that cross bracing should be placed next to all field splices in curved girders. An exception can be made for any curved girder spans that are connected on the ground before being lifted onto the piers, as such practice prevents the initial instability the bracing also addresses.

The proposed bracing requirements would be an important step in improving the safety of bridge construction projects across the state. If implemented,

TABLE 3.1
Results of Unbraced Length Analysis

Beam Type	AASHTO II	AASHTO III	AASHTO IV	AASHTO V	AASHTO VI
Height (in)	36	45	54	63	72
# PS Strands	22	32	54	60	64
PS force (kips)	740	990	1670	1860	1980
PS cg (in)	13.2	13.6	16.0	15.7	19.0
yb (in)	15.8	20.3	24.7	32.0	36.4
Eccentricity (in)	2.6	6.7	8.7	16.3	17.4
Span (ft)	50	50	100	100	100
# Braces	0	0	1	1	1
Unbraced Length (ft)	50	50	50	50	50
Stress - Placement	OK	OK	OK	OK	OK
Stress - Braced	OK	OK	OK	OK	OK
Stress - Deck	OK	OK	OK	OK	OK
Stability - Braced	OK	OK	OK	OK	OK
Stability - Deck	OK	OK	OK	OK	OK

they would help prevent accidents that could be caused by unstable girders under construction loads.

4. SUMMARY AND CONCLUSIONS

The goal of this study was to properly identify the state's temporary support and loading requirements during construction. In implementing these requirements, INDOT can proactively reduce its risk of construction accidents similar to those that have occurred elsewhere in the country. Despite potentially raising the cost of construction, the proposed upgrades would prove invaluable by helping to eliminate the possibility of an expensive and disastrous accident. The following provisions are recommended for inclusion in INDOT's requirements:

- A dead load of 150 pcf for concrete and reinforcing steel, plus 15 psf for formwork, consistent with the current design memo.
- A live load consisting of known construction loads, a 20 psf uniform load, and a 75 plf load at overhangs, also consistent with the current design memo.
- Minimum vertical load of 100 psf and minimum horizontal load, equal to 2% of the dead load, to ensure conservative design.
- An impact load requiring conservative design of any falsework potentially affected by placement or lifting operations, and of any falsework over or adjacent to traffic.
- A new wind load provision, including simple tables that would provide a wind pressure table and a reduction factor. Analysis was performed using ASCE 7-10 method with a 115-mph design wind speed. A SAP model was used to confirm proper use of the gust effect factor.
- Minimum bracing requirements for prestressed and steel girders. Proposed standard drawings would require girders to be adequately braced before they are released from the cranes during erection and before slab placement. Buckling analysis verified potential spacing limits for lateral bracing.

Implementing the new design loads will ensure that structures are more equipped to handle actual construction loads, and the lateral bracing provisions will help improve safety in the field by reducing the possibility of dangerous collapses. By implementing these proposed requirements, INDOT can take a significant step toward preventing bridge construction accidents.

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APPENDIX A. EXAMPLE LIVE LOAD CALCULATION

An analysis was performed in order to determine how much more conservative the proposed new live load requirement would be compared to the old requirement of 50 psf for both falsework and formwork. A design example found in Chapter 5 of the 2009 Kansas Department of Transportation (KDOT) *Design Manual* (*1*) shows a thorough design of a typical falsework and formwork system for a steel girder bridge. The design example uses 50 psf for both falsework and formwork design. For this study, the falsework design was redone using the new proposed live load requirement for the design of the overhang brackets, which are categorized as falsework, with all other given information held constant.

Figure A.1 shows a cross section of the bridge in the design problem. The overhang bracket analyzed in this study is shown on the left exterior girder. The information given about the bracket is shown in Table A.1 and Figures A.2 and Figures A.3.

In the example, four design checks were performed related to the overhang bracket: the total vertical load, the compression force in the strut, the shear in the bolt, and the force in the hanger rod. Each of these components is shown in Figure A.2. The four calculations were performed in the Design Manual, using a live load of 50 psf, as shown in Figure A.4. They have been reformatted for this report, but the values are the same.

To observe the effect of using the proposed new live load, the four design checks were then performed in this study using a 20 psf uniform live load and a 75 plf linear live load at the overhangs. All other values (dead load, screed weight, etc.) were held constant to isolate the effect of changing the live load. The results of these calculations are shown in Figure A.5.

Finally, Table A.2 shows a comparison of the design checks calculated with the two different live loads. The results show that the proposed new live load is about 8–10% less conservative than the old method – a significant but not extreme value. This validates the proposed values as appropriate for adoption into INDOT's specifications.

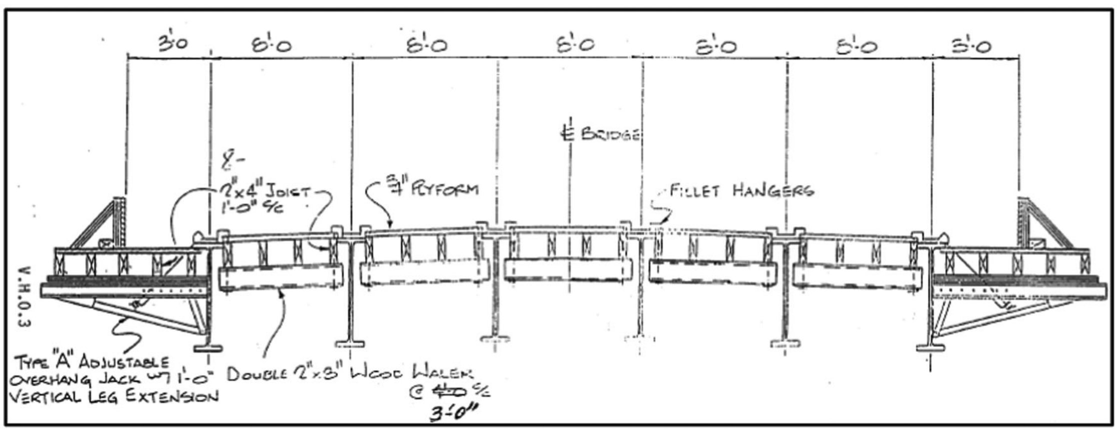

Figure A.1 Falsework and formwork design example (*1*).

TABLE A.1
Given Overhang Bracket Information

Overhang Bracket Spacing		**3**	**Feet**
Bracket Vertical Load Rating		3600	lbs
Screed Load		1087	lbs (per bracket)
Dead Load	Concrete Weight	1080	lbs (per bracket)
	Bracket Weight	50	lbs (per bracket)
	Formwork Weight	88	lbs (per bracket)
	Total Dead Load	1218	lbs (per bracket)

Source: (*1*).

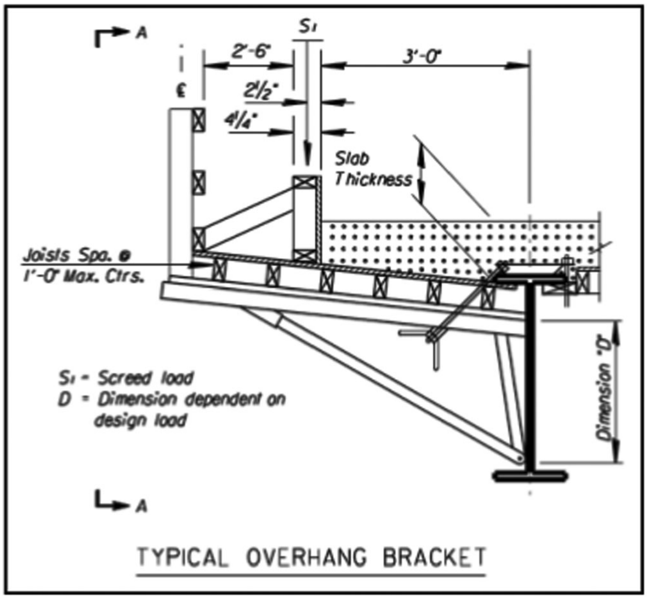

Figure A.2 Given overhang bracket setup (*1*).

Overhang Bracket Design Checks, LL = 50 psf

Compute Live Load to Bracket:

On concrete: (50 psf)(3 ft)(3 ft)	=	450	lbs
On walkway: (50 psf)(2.5 ft)(3 ft)	=	375	lbs
Total Live Load	=	825	lbs

Total Vertical Load to Bracket:

Dead Load	=	1218	lbs
Live Load	=	825	lbs
Screed Load	=	1087	lbs
Total Vertical Load	=	3130	lbs

-- OK compared to 3600-lb rating per manufacturer

Force in Strut, Calculated Using Moment Arms:

$$(3.0')(F) = (3.21')(1087 \text{ lbs}) + (2.93')(825 + 88 \text{ lbs}) + (1.5')(1080 \text{ lbs}) + (1.95')(50 \text{ lbs})$$

Solve for F: F = 2627 lbs

Force in Strut / Angle: $\dfrac{2627}{\cos(29.7°)}$ = 3025 lbs

Check Bolt Double Shear: (Assume A307, 1/2")

Force in Strut / Bolt Area: $\dfrac{3025}{2(0.20 \text{ in}^2)}$ = 7562 psi

-- OK compared to allowable 11000 psi

Check Hanger Rod:

Total Vertical Load / Angle: $\dfrac{3130}{\cos(45°)}$ = 4426 lbs

-- OK compared to 6000-lb rating per manufacturer

Figure A.4 Given design checks with 50 psf live load (*1*).

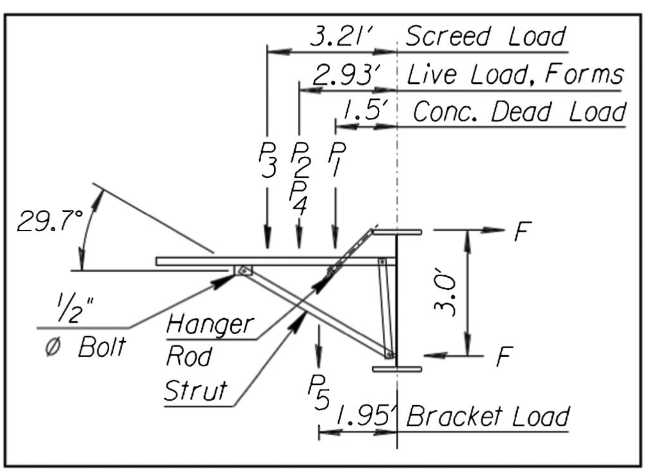

Figure A.3 Given overhang bracket dimensions (*1*).

Overhang Bracket Design Checks, LL = 20 psf & 75 plf

Compute Live Load to Bracket:

On concrete: (20 psf)(3 ft)(3 ft)	=	180	lbs
On walkway: (20 psf)(2.5 ft)(3 ft)			
+(75 plf)(3 ft)	=	375	lbs
Total Live Load	=	555	lbs

Total Vertical Load to Bracket:

Dead Load	=	1218	lbs
Live Load	=	555	lbs
Screed Load	=	1087	lbs
Total Vertical Load	=	2860	lbs

-- OK compared to 3600-lb rating per manufacturer

Force in Strut, Calculated Using Moment Arms:

(3.0')(F) = (3.21')(1087 lbs) + (2.93')(825 + 88 lbs)
+(1.5')(1080 lbs) + (1.95')(50 lbs)

Solve for F: F = 2364 lbs

Force in Strut: / Angle: $\dfrac{2364}{\cos(29.7°)} = 2721$ lbs

Check Bolt Double Shear: (Assume A307, 1/2")

Force in Strut: / Bolt Area: $\dfrac{2721}{2(0.20\ in^2)} = 6803$ psi

-- OK compared to allowable 11000 psi

Check Hanger Rod:

Total Vertical Load: / Angle: $\dfrac{2860}{\cos(45°)} = 4045$ lbs

-- OK compared to 6000-lb rating per manufacturer

Figure A.5 Design checks calculated with proposed live load.

APPENDIX A REFERENCE

1. Kansas Department of Transportation (KDOT). *Design Manual*. Topeka, Kansas, 2009.

TABLE A.2
Comparison of Design Checks with Current and Proposed Live Loads

Calculation	Using Live Load = 50 psf	Using Live Load = 20 psf + 75 plf	Difference (%)
Total Vertical Load to Bracket (lbs)	3130	2860	−8.6
Compression Force in Strut (lbs)	3025	2721	−10.0
Bolt Shear (psi)	7562	6803	−10.0
Hanger Rod Force (lbs)	4426	4045	−8.6

APPENDIX B. WIND LOAD CALCULATION

The wind load analysis was performed assuming the Exposure C case, because Exposure D should rarely be necessary in Indiana. The resulting wind load requirements are valid for both Exposure B and Exposure C, and if a project requires Exposure D, it should be designed according to *ASCE 7-10* (*1*).

Wind Pressure Values:

The design wind pressure in *ASCE 7-10* is calculated in Chapters 26–28 of the document (*1*). The equation for wind pressure is

$$p = qGC_p - q_i(GC_{pi}) \qquad (Eq.1)$$

where q = q_z, the velocity pressure, G is the gust effect factor, and C_p is an external pressure coefficient determined from a table. C_{pi} is an internal pressure coefficient, which is zero for open structures such as falsework. The velocity pressure q_z is shown in Equation 3.2, where K_z is a velocity pressure exposure coefficient dependent on the height and the exposure category, K_{zt} is a topographic factor, K_d is a wind directionality factor (all three determined from tables), and V is the basic wind speed in miles per hour.

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \qquad (Eq.2)$$

Table B.1 shows the value selected for each of the constants, along with the corresponding sections in *ASCE 7-10*. K_{zt} and K_d can be conservatively estimated as 1.0. C_p, the external pressure coefficient, was determined from *ASCE 7-10*, Table 27.4-1. For this study, C_p was considered to include both the windward and leeward walls, with the wind pressure to be applied only on the windward face, to simplify design. The C_p value was estimated at 1.15, the average value between the maximum (1.30) and minimum (1.00) possible sums of the windward and leeward pressures.

The gust effect factor, G, is dependent on whether the structure under consideration is rigid. A rigid structure is defined as having a natural frequency no less than 1 cycle per second. For rigid structures, G can be taken as 0.85. For non-rigid structures, it must be calculated with a series of formulas provided in *ASCE 7-10*. An analysis was performed with a SAP2000 model in order to confirm that typical falsework structures are considered rigid.

Dimensions of a falsework system set up on campus were measured, and a model using similar elements was created. A steel pipe with an outer diameter of 2 inches and a thickness of 1/8″ was used as a typical falsework member. The vertical members were considered continuous, with the horizontal and diagonal members pinned at all joints. The falsework was modeled as a series of bays measuring 6 feet wide and 6 feet 8 inches high, with diagonal members bracing each bay. Three models were constructed; each measured 12 feet wide by 42 feet long and topped with an 8-inch concrete slab, and the height was varied among the models from 20 feet to 40 feet to 60 feet. Figure B.1 shows side and end elevation views of the 60-foot model.

The models were analyzed for their natural frequencies, and were confirmed to be rigid structures. The 20-foot model had a natural frequency of 4.20 Hz, the 40-foot model 2.15 Hz, and the 60-foot model 2.02 Hz. It was concluded that typical falsework structures are rigid and can be assigned a gust effect factor of 0.85.

The exposure coefficient K_z varies with height. Table B.2 shows the value for each height zone from Table 27.3-1 of *ASCE 7-10*, the velocity pressure calculated using Equation 2, and finally the wind pressure calculated with Equation 1.

To simplify the wind pressure values for falsework and formwork design, the values were then approximated into three different height zones at intervals of 25 feet: 30 psf for falsework between 0 and 25 off the ground, 35 psf from 25–50 feet, and 40 psf from 50–75 feet. Table B.3 compares the calculated wind pressure with the approximated value.

It is suggested that, like in California's and Washington's specifications (*2–4*) and *AASHTO Bridge Temporary Works* (*5,6*), the wind load should be increased by 5 psf for falsework over or adjacent to traffic openings to account for the importance of such falsework. Table B.4 is the wind pressure requirement recommended for Indiana's specifications.

Wind Pressure Reduction Factor:

As mentioned in the Literature Survey, *ASCE 37-02* (*7*) contains a provision for reducing the wind speed based on the length of the construction period. The reduction is encouraged because the basic wind speed prescribed in *ASCE 7-10* (*1*) is the 50-year design wind speed; i.e., the highest speed which is expected in a 50-year time frame. For building design, a 50-year design wind is appropriate because the expected life span of a structure is often more than 50 years, but for temporary structures, it is highly unlikely that speed will be reached during short-term construction periods. The design wind speed can be significantly reduced while still maintaining a low probability of reaching the 50-year speed. In this study, it was desired to produce a factor to reduce the wind pressure, rather than the wind speed, in order to shorten the wind load design process. Because Equations 1 and 2 show that wind pressure varies with the square of wind speed, the wind speed reduction factors in Table B.5 can simply be squared and applied to the wind pressure instead. Table B.5 shows the results of this calculation. The construction period is defined as the time between the first and last use of falsework on the project.

Wind Impact Area:

After the design wind pressure values and an appropriate reduction factor were determined, the area over which the load should be applied needed to be specified. For unenclosed falsework, the wind load simply acts on the projected area of the falsework perpendicular to the wind. For enclosed (wrapped) falsework, the wind acts on the gross area of the falsework on the face normal to the wind, and there is also an external pressure effect that acts outward on the side faces of enclosed structures. Table 27.4-1 of *ASCE 7-10* lists a pressure coefficient C_p of 0.7 outward from side walls. However, since a C_p of 1.15 was already factored into the pressure values for walls normal to the wind, 0.7 must be divided by 1.15 to produce a coefficient of 0.60 that should be multiplied by the wind load to find the pressure acting outward from the side walls. Figure B.2 illustrates where the wind load, represented as "W," shall be applied.

TABLE B.1
Wind Load Variables for *ASCE 7-10* Calculation

Variable		Value	Units	ASCE 7–10 Section
Exposure Category		C	—	26.7.3
Wind Speed	V	115	mph	26.5.1
Wind Directionality	K_d	1.0	—	26.6
Topographic Factor	K_{zt}	1.0	—	26.8.2
Internal Pressure	GC_{pi}	0	—	26.11.1
External Pressure	C_p	1.15	—	27.4.1
Gust Effect	G	0.85	—	26.9

TABLE B.2
Calculation of Wind Pressure Values

Height (ft)	Exposure Coefficient, K_z Section 27.3.1	Velocity Pressure, q_z (psf) Section 27.3.2	Wind Pressure, p (psf) Section 27.4.1
0–15	0.85	28.8	28.1
15–20	0.90	30.5	29.8
20–25	0.94	31.8	31.1
25–30	0.98	33.2	32.4
30–40	1.04	35.2	34.4
40–50	1.09	36.9	36.1
50–60	1.13	38.3	37.4
60–70	1.17	39.6	38.7
70–80	1.21	41.0	40.0
80–90	1.24	42.0	41.0
90–100	1.26	42.7	41.7
100–120	1.31	44.4	43.4

TABLE B.3
Approximation of Wind Pressure Values

Height (ft)	ASCE 7-10 Wind Pressure (psf)	Approximate Wind Pressure (psf)
0–15	28.1	30
15–20	29.8	30
20–25	31.1	30
25–30	32.4	35
30–40	34.4	35
40–50	36.1	35
50–60	37.4	40
60–70	38.7	40
70–80	40.0	40
80–90	41.0	N/A
90–100	41.7	N/A
100–120	43.4	N/A

TABLE B.4
Wind Pressure Values for Design of Falsework and Formwork

	Wind Pressure (psf)	
Height Above Ground (ft)	Typical Falsework	Falsework over or Adjacent to Traffic Openings
0–25	30	35
25–50	35	40
50–75	40	45

TABLE B.5
Wind Pressure Reduction Factor

Construction Period	Wind Pressure Reduction Factor
Less than 6 weeks	0.57
6 weeks to 1 year	0.64
1 to 2 years	0.73
2 to 5 years	0.81

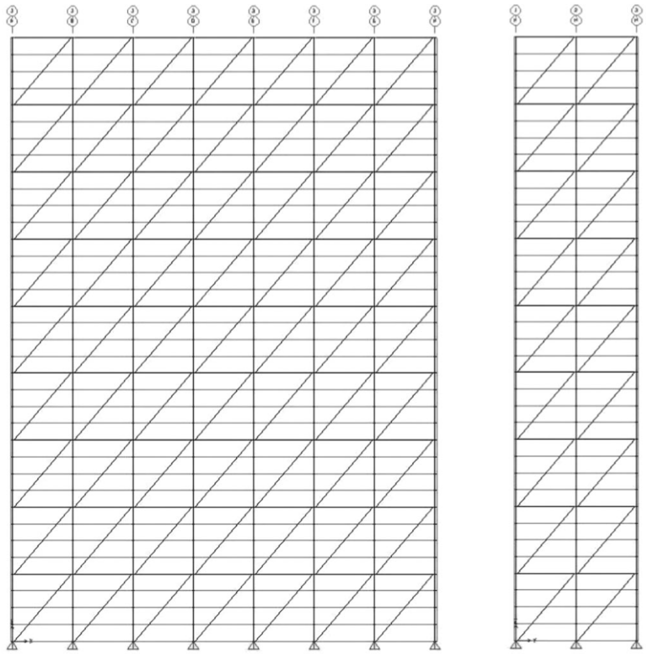

Figure B.1 60-foot falsework SAP2000 model.

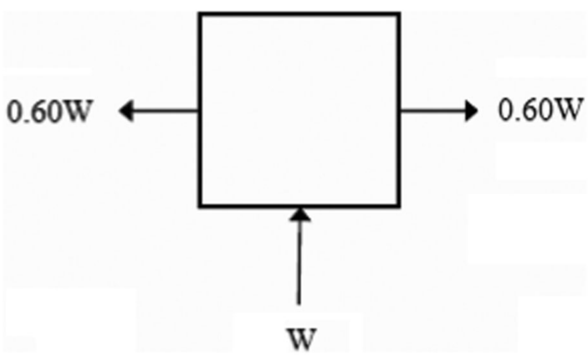

Figure B.2 Application of wind load on enclosed falsework.

APPENDIX B REFERENCES

1. American Society of Civil Engineers (ASCE). *ASCE 7-10: Minimum Design Loads for Buildings and Other Structures*. Reston, Virginia, 2010.
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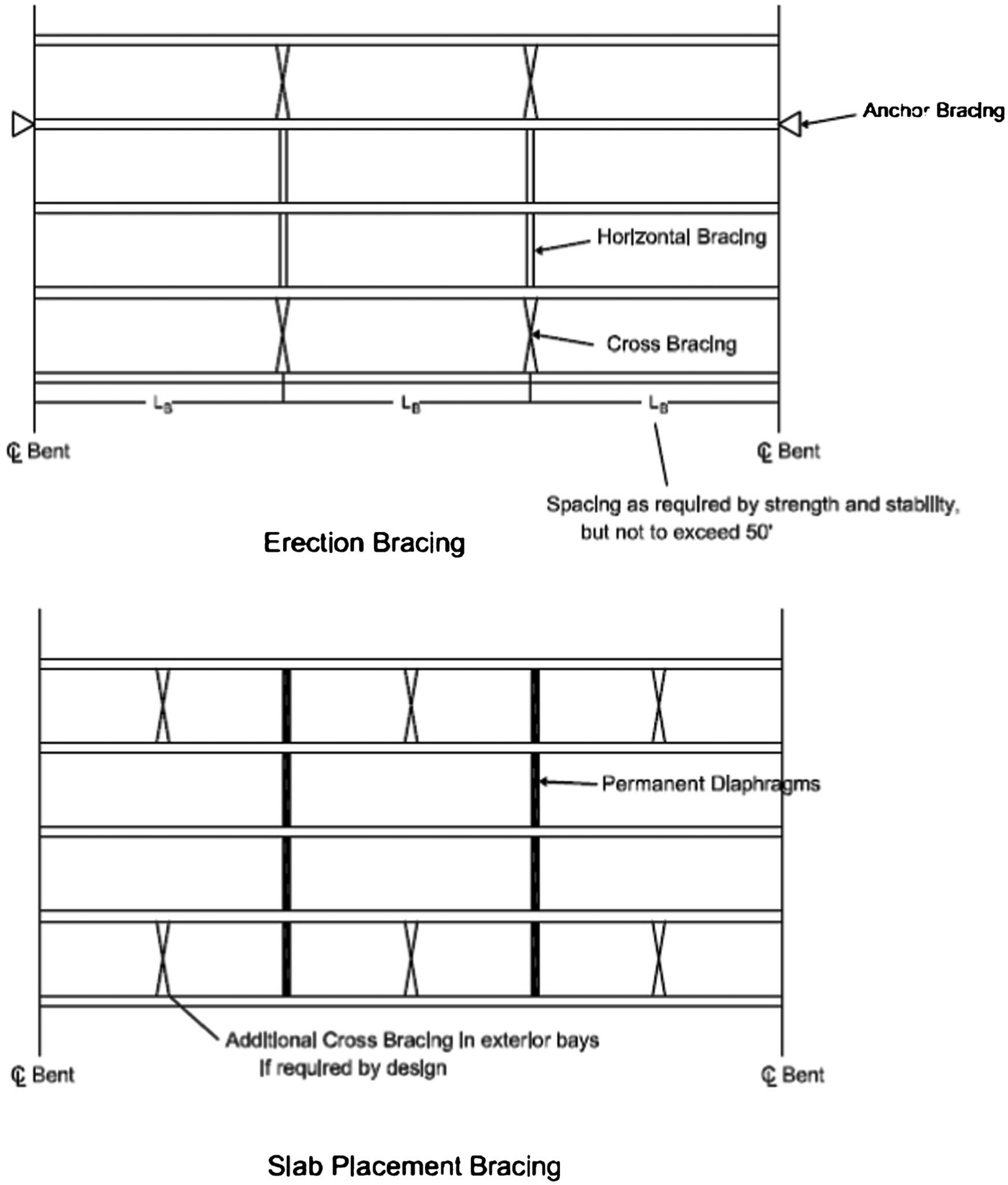


Figure C.1 Erection and slab placement bracing.

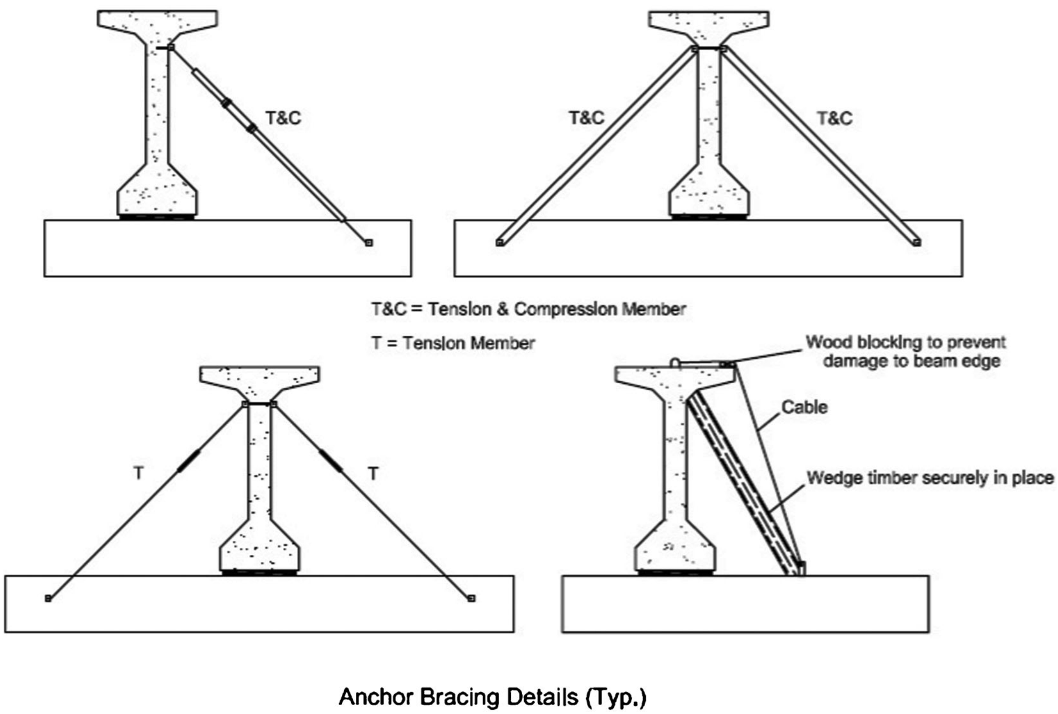


Figure C.2 Typical anchor bracing details.

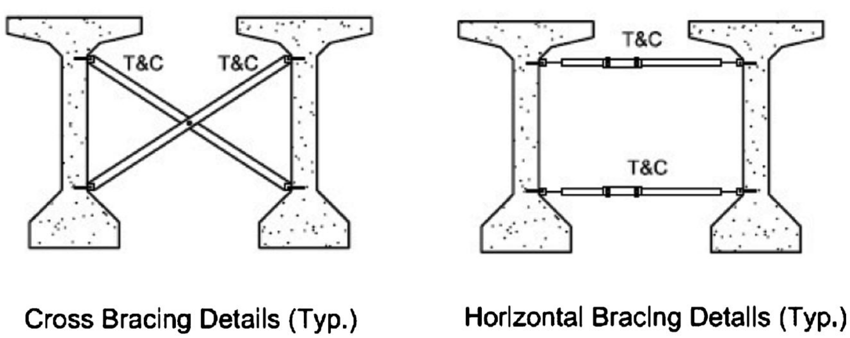


Figure C.3 Typical cross bracing and horizontal bracing details.

MINIMUM BRACING REQUIREMENTS FOR PRESTRESSED BEAMS AND GIRDERS

GENERAL NOTES

- Use of systems shown in these details does not relieve the Contractor of the responsibility for the adequacy of the bracing and the safety of the structure.
- The bracing systems shown in these details are schematic only. and are meant only to show geometry in which bracing should be placed. Bracing members and connections shall be designed and detailed by the Contractor.
- Proposed bracing details are to be designed by an Engineer and submitted to INDOT for review prior to erection.
- All connections must be capable of developing the full strength of the brace member, e.g. cable or brace.
- Bracing details are for spans less than 150'. Bracing details for spans greater than 150' are not provided.
- Bracing details are not provided for beams or girders with lateral spacing greater than or less than typical values.

ANCHOR BRACING

- Anchor Bracing must be used at both ends of the first beam or girder erected in the span, known as the Anchor Beam. The location of the Anchor Beam may vary. All subsequent beams are to be braced off the Anchor Beam sequentially. The Anchor Bracing may be located at an exterior girder provided that all required bolt clear distances are met and overhang bracing is not affected.
- Anchor Bracing must be installed before the Anchor Beam is released from the cranes used to lift it.
- If the obtuse angle between the beam or girder and the pier exceeds 120 degrees, Anchor Bracing shall be placed perpendicular to the beam or girder.
- Anchor Bracing shall not be removed until all girders and all Horizontal and Cross Braces in the span are in place.

ERECTION BRACING

- Required erection bracing must be installed immediately after erection of each beam and remain in place until slab concrete has attained a compressive strength of 3000 psi.
- Use brace spacing required by strength and stability, but spacing is not to exceed 50'.
- Cross Bracing or Horizontal Bracing at both beam ends is recommended.
- Spans no longer than 100' may have one brace placed at midspan in each bay, rather than the two braces between beam ends shown in the details, if such spacing is adequate according to design.

CROSS BRACING

- Cross Bracing must be placed between the first two beams/girders erected, in exterior bays, and in every fourth bay. At the Contractor's option, remaining bays may have either cross bracing or horizontal bracing.
- Cross Bracing members must be capable of developing tension and compression, and positively attached to the beam web or flange.
- Each beam using Cross Bracing must have at least one brace installed to connect it to an adjacent beam before it is released from the cranes used to lift it. Remaining braces shall be installed as soon as practical. At minimum, the brace closest to midspan shall be in place before the beam is released.
- A permanent diaphragm is an acceptable substitute for Cross Bracing.

HORIZONTAL BRACING

- Horizontal Bracing systems are to use members which are capable of developing tension and compression, and are positively attached to the beam web or flange.
- Each beam using Horizontal Bracing must have at least one brace installed to connect it to an adjacent beam before it is released from the cranes used to lift it. Remaining braces shall be installed as soon as practical. At minimum, the brace closest to midspan shall be in place before the beam is released.
- A permanent diaphragm is an acceptable substitute for Horizontal Bracing.

SLAB PLACEMENT BRACING

- Permanent diaphragms, spaced as required by the design of the bridge, shall be installed prior to slab placement.
- Additional Cross Bracing in exterior bays shall be installed if required by design.
- Temporary bracing must remain in place until slab concrete has attained a compressive strength of 3000 psi.

HAULING & ERECTION

- The Contractor's attention is directed to the possible lateral instability of pre-stressed concrete beams and girders over 130' long, especially during hauling and erection. The use of the following methods to improve stability is encouraged:
    - Locate lifting devices at the maximum practical distance from beam ends.
    - Use lateral stiffening devices during hauling and erection.
    - Lift with vertical lines using two cranes.
    - Take care in handling to minimize inertial and impact forces.

Figure C.4 Notes on minimum bracing requirements for pre-stressed beams.

APPENDIX D. MINIMUM BRACING REQUIREMENTS FOR STEEL BEAMS

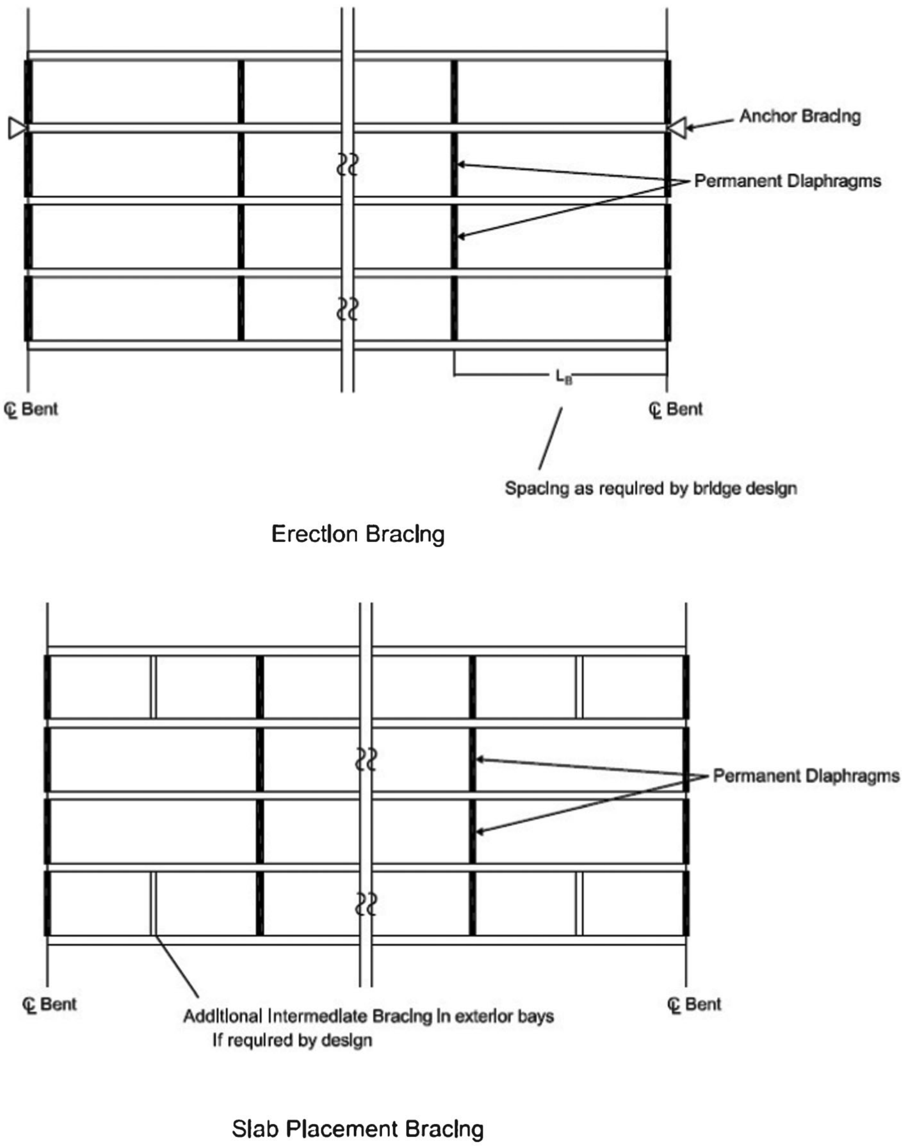


Figure D.1 Erection and slab placement bracing.

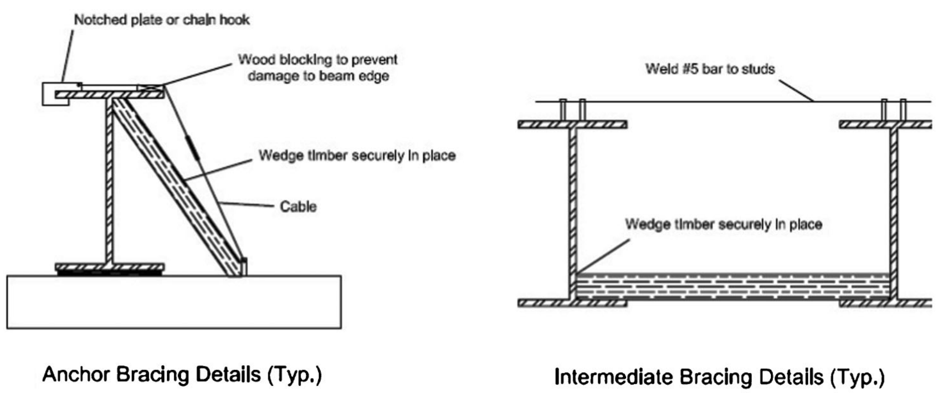


Figure D.2 Typical anchor bracing and intermediate bracing details.

MINIMUM BRACING REQUIREMENTS FOR STEEL BEAMS AND GIRDERS

GENERAL NOTES

- Use of systems in these details does not relieve the Contractor of the responsibility for the adequacy of the bracing and the safety of the structure.
- The bracing systems shown in these details are schematic only, and are meant only to show geometry in which bracing should be placed. Bracing members and connections shall be designed and detailed by the Contractor.
- Proposed bracing details are to be designed by an Engineer and submitted to INDOT for review prior to erection.
- All connections must be capable of developing the full strength of the brace member, e.g. cable or brace.
- Timber sections less than 4x4 nominal are not to be used as bracing members.

ANCHOR BRACING

- For straight beams, Anchor Bracing must be used at both ends of the first beam erected in the span, known as the Anchor Beam. The location of the Anchor Beam may vary. All subsequent beams are to be braced against the Anchor Beam sequentially. The Anchor Bracing may be located at an exterior girder provided that all required bolt clear distances are met and overhang bracing is not affected.
- Anchor Bracing must be used at both ends of all curved beams, unless beams are connected on the ground before being lifted onto piers.
- Anchor Bracing must be installed before the Anchor Beam is released from the cranes used to lift it.
- If the obtuse angle between the beam or girder and the pier exceeds 120 degrees, Anchor Bracing shall be placed perpendicular to the beam or girder.
- Anchor Bracing shall not be removed until all girders and all diaphragms in the span are in place.

ERECTION BRACING

- Required Erection Bracing includes Permanent Diaphragms spaced as required by the design of the bridge.
- Each beam must have at least one diaphragm connecting it to an adjacent beam before it is released from the cranes used to lift it. Remaining diaphragms shall be installed as soon as practical. At minimum, the diaphragm closest to midspan shall be in place before the beam is released.

CROSS BRACING

- On curved girders, temporary Cross Bracing must be installed next to all field splices, unless beams are connected on the ground before being lifted onto piers.

SLAB PLACEMENT BRACING

- Slab Placement Bracing includes additional Intermediate Bracing to be installed in exterior bays if required by design.
- Intermediate Bracing must remain in place until slab concrete has attained a compressive strength of 3000 psi.

Figure D.3 Notes on minimum bracing requirements for steel beams.

Modified by Danny McPheron, Purdue University

Concrete I-Girder
Beam Stability Program

Project =
DesignedBy =
CheckedBy =
BackCheckedBy =

These are calculations for the Lateral Stability of Precast Concrete Bridge Girders during construction. Instructions for use of this program are as follows:

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.

Girder Variables:

Girder Type	Girder := AASHTO Type VI
Unit weight of Concrete	$w_c := 150 \cdot pcf$
Concrete Strength	$f_{c'} := 8 \cdot ksi$
Effective Prestressing Force (may assume all losses have occurred)	$P_e := 1980 \cdot kip$
Eccentricity of Prestressing	$e := 17.4 \cdot in$

Geometry:

Beam Span Length (centerline to centerline bearing)	$L := 100 \cdot ft$
Number of Intermediate Bracing Points, (from 0 to 20)	$n_b := 1$
Sweep Tolerance	$tol_S := \dfrac{\frac{1}{8} \cdot in}{10 \cdot ft}$
Initial imperfection of bracing	$e_b := .25 \cdot in$
Eccentricity due to setting the beams off-center on the pads (recommend 0.25 in)	$e_{set} := .25 \cdot in$

Skew Angle (between 0 and 60)	$\phi := 0 \cdot deg$	
Beam Spacing	$S := 8 \cdot ft$	
Number of Beams in X-Section (from 2 to 12)	$n_{beam} := 5$	
Overhang Length (measured from centerline of exterior beam)	$OH := 4.542 \cdot ft$	
Deflection of Deck Limit at Edge of Cantilever (recommend .25 in)	$\delta_{max} := .25 \cdot in$	
Deck thickness (total, including IWS)	$t_d := 8 \cdot in$	

Bearing Pad Properties:

Bearing pad plan dimensions (a=width, b=length)	$a := 23 \cdot in$	$b := 12 \cdot in$
Thickness of internal elastomer layer	$t := .5 \cdot in$	*When the thickness of the exterior layer of elastomer is equal to or greater than one-half the thickness of an interior layer, the parameter, n, may be increased by one-half for each such exterior layer.*
Number of interior layers of elastomer	$n := 5$	
Elastomer Shear Modulus	$G_{bp} := 93.5 \cdot psi$	
Tilt Angle of Support in Radians (Bearing Pad Construction Tolerance)	$\alpha := .01$	*(recommend 0.01)*
Distance from Bottom of Beam to Roll Axis (half bearing pad thickness)	$h_r := \frac{\left(3\frac{11}{16}\right)}{2} \cdot in = 1.844 \cdot in$	

Loads:

Basic Wind Speed	$V_B := 115 \cdot mph$	
Wind Pressure Value	$WP := 35psf$	
Wind Pressure Reduction Factor	$RF := 0.64$	
Construction Wind Load Factor	$\gamma := 1.25$	
Bridge Height, measured to mid-height of beam (ft)	$Height := 20.5 \cdot ft$	
Construction Active Wind Load for single girder	$w_{wE} := WP \cdot RF = 22.4 \cdot psf$	
Construction Inactive Wind Load for single girder	$w_w := WP \cdot RF = 22.4 \cdot psf$	
Construction Active Wind Load for entire bridge section	$w_{wD} := WP \cdot RF = 22.4 \cdot psf$	

Weight of build-up $w_b := 50 \cdot plf$

Weight of forms (20 psf recommended) $w_f := 20 \cdot psf$

Live loads during deck pour (20 psf and 75 plf at edge of overhang per AASHTO Guide Design Specifications for Temporary Works) $w_l := 20 \cdot psf$ $P_l := 75 \cdot plf$

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)*

Location of (plf) live load in relation to edge of overhang, positive is to exterior of overhang edge, negative is to interior of overhang edge $d_{ll} := 1 \cdot ft$ *(+1 ft.. recommended, accounts for workers on platform)*

Girder Properties

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

Unbraced Length of Beam $L_b := \dfrac{L}{n_b + 1} = 50\,ft$

Height $h := Properties_{Girder,1} \cdot in = 72 \cdot in$

Top flange width $b_t := Properties_{Girder,2} \cdot in = 42 \cdot in$

Bottom flange width $b_b := Properties_{Girder,3} \cdot in = 28 \cdot in$

Modulus of Elasticity $E_c := 33000 \cdot .9 \cdot .145^{1.5} \cdot \left(\dfrac{f_{c'}}{ksi}\right)^{0.5} \cdot ksi = 4.638 \times 10^3 \cdot ksi$

Shear Modulus $G_{shear} := .416667 \cdot E_c = 1.933 \times 10^3 \cdot ksi$

Area of Concrete $A_c := Properties_{Girder,4} \cdot in^2 = 1.085 \times 10^3 \cdot in^2$

Moment of Inertia, about x-axis $I_x := Properties_{Girder,5} \cdot in^4 = 733320 \cdot in^4$

Moment of Inertia, about y-axis $I_y := Properties_{Girder,6} \cdot in^4 = 61621 \cdot in^4$

Distance from CG to top of beam $y_t := Properties_{Girder,7} \cdot in = 35.62 \cdot in$

Distance from CG to bottom of beam $y_b := Properties_{Girder,8} \cdot in = 36.38 \cdot in$

Torsional Constant $J := Properties_{Girder,9} \cdot in^4 = 32045 \cdot in^4$

Section Moduli $S_t := \dfrac{I_x}{y_t} = 20587 \cdot in^3$ $S_b := \dfrac{I_x}{y_b} = 20157 \cdot in^3$ *Section moduli about x-axis*

$S_{yt} := \dfrac{2 \cdot I_y}{b_t} = 2934 \cdot in^3$ $S_{yb} := \dfrac{2 \cdot I_y}{b_b} = 4402 \cdot in^3$ *Section moduli about y-axis*

Self-weight of beam and deck $w := A_c \cdot w_c = 1.13 \times 10^3 \cdot plf$ $w_d := t_d \cdot w_c = 100 \cdot psf$

Lateral Deflection and Eccentricity of Girder Center of Gravity:

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

Maximum Lateral Deflection of Uncracked Section

$$z_o := \frac{w \cdot L^4}{120 \cdot E_c \cdot I_y} = 5.694 \cdot in$$

Eccentricity due to Sweep

$$e_s := \min(1.5 \cdot in, L \cdot tol_S) \cdot \frac{2}{3} = 0.833 \cdot in$$

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

$$e_w := \frac{w_w \cdot h \cdot L^4}{120 \cdot E_c \cdot I_y} = 0.677 \cdot in$$

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

$$e_{wE} := \frac{w_{wE} \cdot h \cdot L^4}{120 \cdot E_c \cdot I_y} = 0.677 \cdot in$$

Lateral deflection due to wind, based on uncracked section

Eccentricity due to wind loading at construction active wind speed, entire bridge section

$$e_{wD} := \frac{w_{wD} \cdot h \cdot L^4}{120 \cdot E_c \cdot I_y} = 0.677 \cdot in$$

Bearing Pad Rotational Stiffness

$$b\_a := (.5 \;\; .6 \;\; .7 \;\; .75 \;\; .8 \;\; .9 \;\; 1 \;\; 1.2 \;\; 1.4 \;\; 2 \;\; 4 \;\; 10 \;\; 1000)$$

Range of possible length:width ratios of bearing pad

$$C := (136.7 \;\; 116.7 \;\; 104.4 \;\; 100 \;\; 96.2 \;\; 90.4 \;\; 86.2 \;\; 80.4 \;\; 76.7 \;\; 70.8 \;\; 64.9 \;\; 61.9 \;\; 60)$$

Range of possible coefficient based on length:width ratio

$$C' := \text{linterp}\left(b\_a^T, C^T, \frac{b}{a}\right) = 132.352$$

Coefficient based on length:width ratio

Effect of Skew on Stiffness (coefficient) $\quad Ang := (0 \;\; 15 \;\; 30 \;\; 45 \;\; 60)$

$$Stiffness := (.88 \;\; .59 \;\; .47 \;\; .39 \;\; .32)$$

Range of stiffness coefficients per skew per UF Structures Research Report 2007/52290

Bearing Pad Rotational Stiffness

$$K_\theta := 2\,\text{linterp}\left(Ang^T, Stiffness^T, \frac{\phi}{deg}\right) \cdot \frac{G_{bp} \cdot a^5 \cdot b}{C' \cdot n \cdot t^3} = 153650.369 \cdot \frac{kip \cdot in}{rad}$$

Coefficient for Reaction at Bracing based on number of brace points, i=intermediate, e=end

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

$$k_{ve} := \begin{pmatrix}
0.500 & 0.500 & 0.500 & 0.500 & 0.500 & 0.500 & 0.500 & 0.500 & 0.500 & 0.500 & 0.500 \\
0.684 & 0.581 & 0.529 & 0.498 & 0.478 & 0.464 & 0.454 & 0.443 & 0.436 & 0.433 & 0.426 \\
0.943 & 0.763 & 0.670 & 0.618 & 0.582 & 0.557 & 0.536 & 0.521 & 0.510 & 0.500 & 0.490 \\
1.189 & 0.921 & 0.790 & 0.708 & 0.660 & 0.618 & 0.591 & 0.570 & 0.557 & 0.543 & 0.529 \\
1.434 & 1.082 & 0.910 & 0.807 & 0.739 & 0.696 & 0.653 & 0.627 & 0.601 & 0.584 & 0.575 \\
1.680 & 1.247 & 1.031 & 0.907 & 0.825 & 0.763 & 0.722 & 0.680 & 0.660 & 0.629 & 0.618 \\
1.924 & 1.407 & 1.154 & 0.998 & 0.902 & 0.830 & 0.782 & 0.734 & 0.709 & 0.673 & 0.661 \\
2.158 & 1.567 & 1.264 & 1.099 & 0.976 & 0.893 & 0.838 & 0.797 & 0.756 & 0.728 & 0.701 \\
2.396 & 1.716 & 1.391 & 1.190 & 1.051 & 0.974 & 0.897 & 0.850 & 0.804 & 0.773 & 0.742 \\
2.628 & 1.872 & 1.495 & 1.288 & 1.134 & 1.031 & 0.962 & 0.893 & 0.859 & 0.807 & 0.790 \\
2.872 & 2.022 & 1.606 & 1.379 & 1.209 & 1.096 & 1.020 & 0.945 & 0.907 & 0.850 & 0.831 \\
3.092 & 2.185 & 1.732 & 1.464 & 1.278 & 1.154 & 1.072 & 1.010 & 0.948 & 0.907 & 0.866 \\
3.327 & 2.323 & 1.831 & 1.541 & 1.362 & 1.228 & 1.139 & 1.050 & 0.983 & 0.938 & 0.893 \\
3.535 & 2.477 & 1.948 & 1.635 & 1.443 & 1.299 & 1.178 & 1.106 & 1.034 & 0.986 & 0.938 \\
3.762 & 2.603 & 2.061 & 1.726 & 1.495 & 1.340 & 1.237 & 1.160 & 1.082 & 1.031 & 0.979 \\
3.985 & 2.749 & 2.144 & 1.814 & 1.567 & 1.402 & 1.292 & 1.209 & 1.127 & 1.072 & 1.017 \\
4.205 & 2.891 & 2.249 & 1.869 & 1.635 & 1.460 & 1.343 & 1.256 & 1.168 & 1.110 & 1.051 \\
4.422 & 3.030 & 2.350 & 1.948 & 1.701 & 1.515 & 1.391 & 1.299 & 1.206 & 1.144 & 1.082 \\
4.635 & 3.166 & 2.448 & 2.024 & 1.763 & 1.567 & 1.436 & 1.338 & 1.240 & 1.175 & 1.110 \\
4.810 & 3.298 & 2.542 & 2.096 & 2.130 & 1.615 & 1.477 & 1.374 & 1.271 & 1.203 & 1.134 \\
5.051 & 3.427 & 2.633 & 2.201 & 1.912 & 1.696 & 1.551 & 1.407 & 1.335 & 1.263 & 1.190
\end{pmatrix}$$

$$K_{vi} := k_{vi_{n_b, n_{beam}-2}} = 1.003$$

$$K_{ve} := k_{ve_{n_b, n_{beam}-2}} = 0.498$$

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

Coefficient for Bending Moment in Girder based on number of brace points

$$K_M := k_{m_{n_b,\, n_{beam}-2}} = 0.124$$

Calculation of Bending Moments:

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

$$w_{D.ext} := w + w_b + (w_d + w_l)\cdot(.5\cdot S + OH) + w_f\cdot(.5\cdot S + OH - b_t) = 2.31\cdot klf$$

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

$$w_{D.int} := w + w_b + (w_d + w_l)\cdot S + w_f\cdot(S - b_t) = 2.23\cdot klf$$

Includes all construction loads except finishing machine

Strength I Torsional Distributed Overhang Moment during deck placement

$$M_c := 1.25\cdot(w_d + w_f)\cdot(OH + .5\cdot S - b_t)\cdot[.5\cdot(.5\cdot S + OH) - .5\cdot S] \ldots \\ + 1.5\cdot w_l\cdot \mathrm{if}\left[(OH + d_{fm}) > 0, (OH - .5\cdot b_t)\cdot\left[.5\cdot b_t + .5\cdot(OH - .5\cdot b_t)\right], -(.5\cdot S - .5\cdot b_t)\cdot .5\cdot(.5\cdot S - .5\cdot b_t)\right] = 0.47\cdot\frac{kip\cdot ft}{ft}$$

Strength I Torsional Finishing Machine and 75 plf Live Load Moment

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

Finishing Machine Moment at each exterior girder

Lateral Moment Due to Construction Inactive wind speed

$$M_w := K_M\cdot w_w\cdot h\cdot L_b^2 = 500\cdot kip\cdot in$$

Vertical Moment due to girder self-weight

$$M_g := \frac{w\cdot L^2}{8} = 16953\cdot kip\cdot in$$

Lateral Moment Due to Construction Active Wind speed, braced condition

$$M_{wE} := K_M\cdot w_{wE}\cdot h\cdot L_b^2 = 500\cdot kip\cdot in$$

Lateral Moment Due to Construction Active Wind speed, unbraced condition

$$M_{wE.u} := .125\cdot w_{wE}\cdot h\cdot L^2 = 2016\cdot kip\cdot in$$

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

$$M_{gD} := \max\left[\frac{w_{D.ext}\cdot L^2}{8} + \frac{(.5 w_{fm} + P_l\cdot 20\cdot ft)\cdot L}{4}, \frac{w_{D.int}\cdot L^2}{8}\right] = 37441\cdot kip\cdot in$$

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

Camber (approx.)

$$\delta_c := \frac{L^2 \cdot \left(P_e \cdot e - \frac{5w \cdot L^2}{48}\right) \cdot 2}{8 \cdot E_c \cdot I_x} = 2.151 \cdot in$$

Assumes creep factor is 2.0

Distance from Center of Gravity to Roll Axis

$$y := y_b + h_r + \delta_c \cdot \frac{2}{3} = 39.658 \cdot in$$

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

Elastic Rotational Spring Constant (sum of 2 bearing pads)

$$K_\theta = 153650.369 \cdot \frac{kip \cdot in}{rad}$$

Radius of Stability

$$r := \frac{K_\theta}{w \cdot L} = 113.291\,ft$$

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 at Top of Beam, Tension

$$f_{ttE} := -\frac{P_e}{A_c} + \frac{P_e \cdot e}{S_t} - \frac{M_g}{S_t} + \frac{M_{wE.u}}{S_{yt}} = -0.288 \cdot ksi$$

Sign convention is tension=positive, compression=negative

Stress at Top of Beam, Compression

$$f_{tcE} := -\frac{P_e}{A_c} + \frac{P_e \cdot e}{S_t} - \frac{M_g}{S_t} - \frac{M_{wE.u}}{S_{yt}} = -1.662 \cdot ksi$$

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

$$f_{btE} := -\frac{P_e}{A_c} - \frac{P_e \cdot e}{S_b} + \frac{M_g}{S_b} + \frac{M_{wE.u}}{S_{yb}} = -2.235 \cdot ksi$$

Stress at Bottom of Beam, Compression

$$f_{bcE} := -\frac{P_e}{A_c} - \frac{P_e \cdot e}{S_b} + \frac{M_g}{S_b} - \frac{M_{wE.u}}{S_{yb}} = -3.151 \cdot ksi$$

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

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

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
$$f_{tt} := -\frac{P_e}{A_c} + \frac{P_e \cdot e}{S_t} - \frac{M_g}{S_t} + \frac{M_w}{S_{yt}} = -0.805 \cdot \text{ksi}$$

Stress at Top of Beam, Compression
$$f_{tc} := -\frac{P_e}{A_c} + \frac{P_e \cdot e}{S_t} - \frac{M_g}{S_t} - \frac{M_w}{S_{yt}} = -1.145 \cdot \text{ksi}$$

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
$$f_{bt} := -\frac{P_e}{A_c} - \frac{P_e \cdot e}{S_b} + \frac{M_g}{S_b} + \frac{M_w}{S_{yb}} = -2.579 \cdot \text{ksi}$$

Stress at Bottom of Beam, Compression
$$f_{bc} := -\frac{P_e}{A_c} - \frac{P_e \cdot e}{S_b} + \frac{M_g}{S_b} - \frac{M_w}{S_{yb}} = -2.807 \cdot \text{ksi}$$

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

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

Modulus of Rupture
$$f_r := 7.5 \cdot \sqrt{f_{c'}} \cdot \sqrt{psi} = 670.82 \cdot psi$$

Lateral Cracking Moment
$$M_{lat} := \min\left[\frac{(f_r - f_{ttE}) \cdot I_y}{\left(\frac{b_t}{2}\right)}, \frac{(f_r - f_{btE}) \cdot I_y}{\left(\frac{b_b}{2}\right)}\right] = 2813.096 \cdot \text{kip} \cdot \text{in}$$

Rotation Angle at Cracking
$$\theta_{cr} := \frac{M_{lat}}{M_g} = 0.166 \cdot \text{rad}$$

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

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
$$FS_{\theta w} := \frac{\theta_{w.max}}{\theta_w} = 7.682$$

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

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

Stress at Top of Beam, Tension $f_{ttD} := -\frac{P_e}{A_c} + \frac{P_e \cdot e}{S_t} - \frac{M_{gD}}{S_t} + \frac{M_{wE}}{S_{yt}} = -1.8 \cdot ksi$

Sign convention is tension=positive, compression=negative

Stress at Top of Beam, Compression $f_{tcD} := -\frac{P_e}{A_c} + \frac{P_e \cdot e}{S_t} - \frac{M_{gD}}{S_t} - \frac{M_{wE}}{S_{yt}} = -2.14 \cdot ksi$

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

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

Stress at Bottom of Beam, Tension $f_{btD} := -\frac{P_e}{A_c} - \frac{P_e \cdot e}{S_b} + \frac{M_{gD}}{S_b} + \frac{M_{wE}}{S_{yb}} = -1.563 \cdot ksi$

Stress at Bottom of Beam, Compression $f_{bcD} := -\frac{P_e}{A_c} - \frac{P_e \cdot e}{S_b} + \frac{M_{gD}}{S_b} - \frac{M_{wE}}{S_{yb}} = -1.79 \cdot ksi$

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

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

Check for stress at deck placement condition

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

Roll Stability Check during Deck Placement (Service I, Construction Active):

Lateral Cracking Moment $M_{latD} := \min\left[\frac{(f_r - f_{ttD}) \cdot I_y}{.5 \cdot b_t}, \frac{(f_r - f_{btD}) \cdot I_y}{.5 \cdot b_b}\right] = 7249.335 \cdot kip \cdot in$

Y-direction moment that causes cracking

Rotation Angle at Cracking $\theta_{crD} := \frac{M_{latD}}{M_{gD}} = 0.194 \cdot rad$

Initial Rotation $\theta_{i.D} := \frac{\alpha \cdot r + e_s + e_{set} + \min(e_b, e_{wD})}{r - y - z_o} = 0.011$

Torque due to construction live loads $T_D := \max\left(\left|M_{fm}\right|, \left|M_c \cdot L_b\right|, \left|M_{fm} + M_c \cdot L_b\right|\right) = 92.897 \cdot kip \cdot ft$

Twist due to construction live loads $\phi_D := \frac{T_D \cdot (.5 \cdot L_b)^2}{G_{shear} \cdot J \cdot L_b} = 0.0027$

Deflection at cantilever due to twist $\delta_D := OH \cdot \tan(\phi_D) = 0.147 \cdot in$

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

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

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

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 e_b. 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.

Bracing Requirements:

Factored Horizontal Force at Each Beam End and Anchor Brace, at midheight of beam — *Strength III, Construction Inactive*

$F_e := w_w \cdot \gamma \cdot h \cdot L_b \cdot K_{ve} = 4.18 \cdot kip$

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

$F_i := if(n_b = 0, \text{"N/A"}, w_w \cdot \gamma \cdot h \cdot L_b \cdot K_{vi}) = 8.425 \cdot kip$

Factored Overturning Force at Each Beam End and Anchor Brace, at top of beam — *Strength I with Strength III Construction Active Wind*

$$M_e := max(|.5 \cdot M_{fm}|, |M_c \cdot L_b \cdot K_{ve}|, |.5 \cdot M_{fm} + M_c \cdot L_b \cdot K_{ve}|) + w_{wD} \cdot \gamma \cdot h \cdot L_b \cdot K_{ve} \cdot .5 \cdot h \ldots = 64.37 \cdot kip \cdot ft$$
$$+ 1.25 \cdot w \cdot L_b \cdot (z_o \cdot \theta_{i.D} + e_s + e_{set} + min(e_b, e_{wD}) + y \cdot \theta_{i.D}) \cdot K_{ve}$$

Note: The assumed finishing machine load is distributed to 2 bracing lines. Revise as necessary.

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

$$M_i := if(n_b = 0, \text{"N/A"}, max(|.5 \cdot M_{fm}|, |M_c \cdot L_b \cdot K_{vi}|, |.5 \cdot M_{fm} + M_c \cdot L_b \cdot K_{vi}|) + w_{wD} \cdot \gamma \cdot h \cdot L_b \cdot K_{vi} \cdot .5 \cdot h + 1.25 \cdot w \cdot L_b \cdot min(e_b, e_{wD}) \cdot K_{vi}) = 84.982 \cdot kip \cdot ft$$

Verification of Bracing Adequacy

Stress Checks

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

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

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

Stability Checks

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

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

Temporary Bracing Variables

Maximum Un-braced Length	$L_b = 50\,ft$
Factored Horizontal Force at Each Beam End and Anchor Brace, at mid-height of beam	$F_e = 4.18 \cdot kip$
Factored Horizontal Bracing Force at Each Intermediate SpanBrace (if present), at mid-height of beam	$F_i = 8.425 \cdot kip$
Factored Overturning Force at Each Beam End and Anchor Brace, at top of beam	$M_e = 64.37 \cdot kip \cdot ft$
Factored Overturning Force at Each Intermediate Span Brace (if present), at top of beam	$M_i = 84.982 \cdot kip \cdot ft$

Wind Load Variables

Basic Wind Speed	$V_B = 115 \cdot mph$
Wind Pressure	$V := WP = 35 \cdot psf$
Gust effect factor	$G = 1 \times 10^{-4}\,T$

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	$w_d = 100 \cdot psf$
Live load	$w_l = 20 \cdot psf$
Live Load at Extreme Deck Edge	$P_l = 75 \cdot plf$

APPENDIX F. EXAMPLE PSBEAM DESIGN

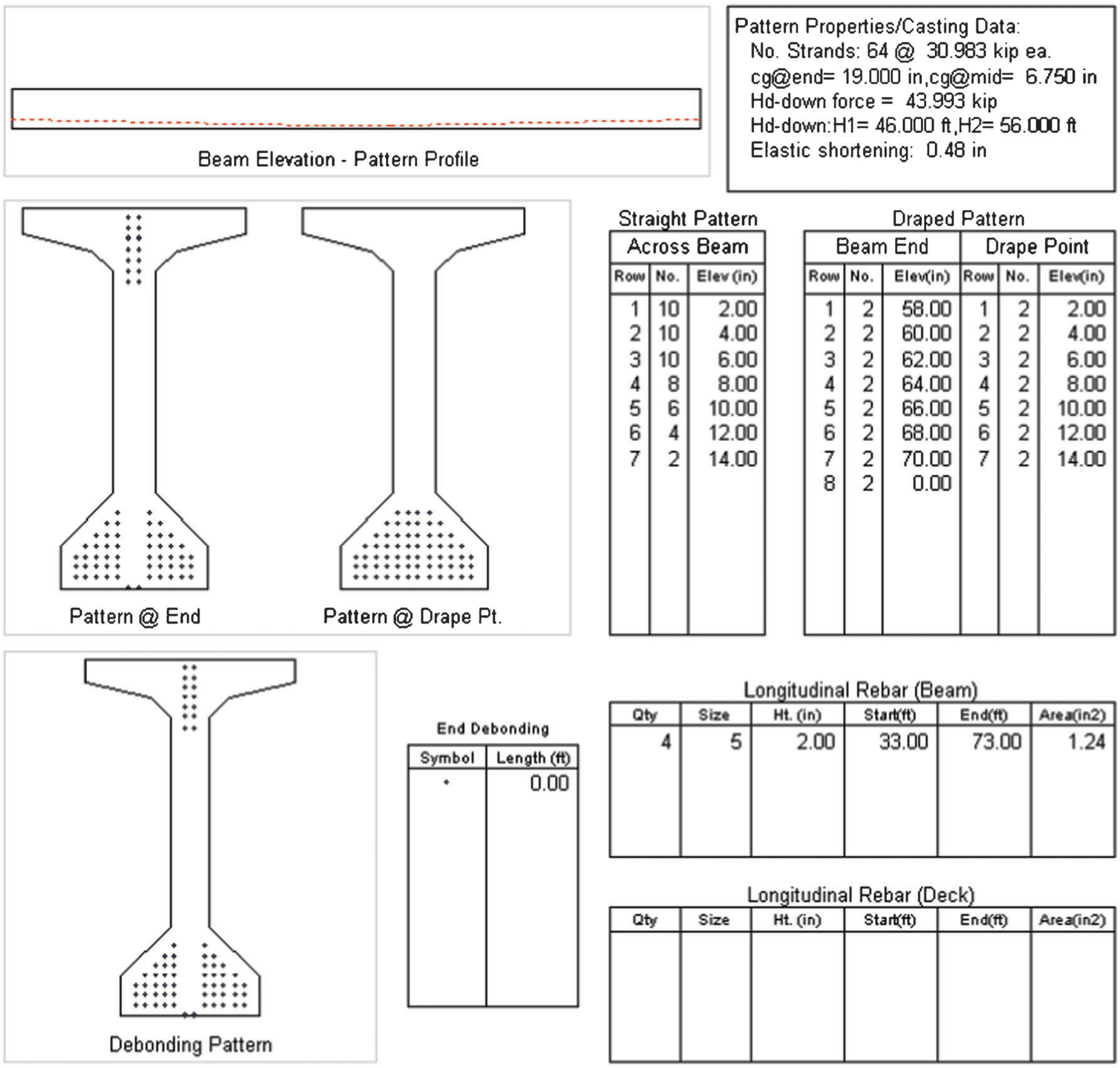
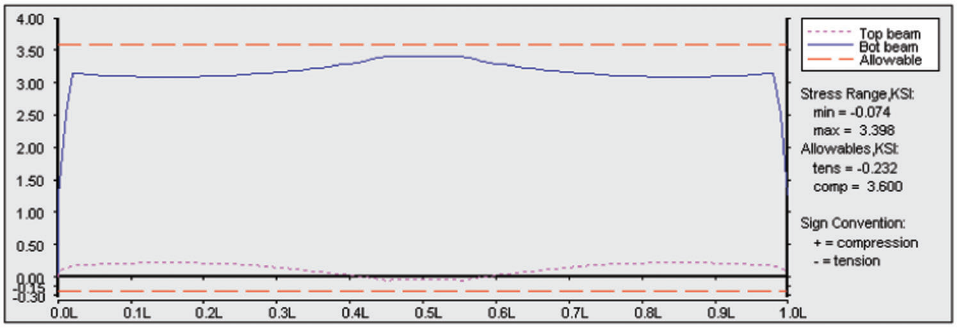


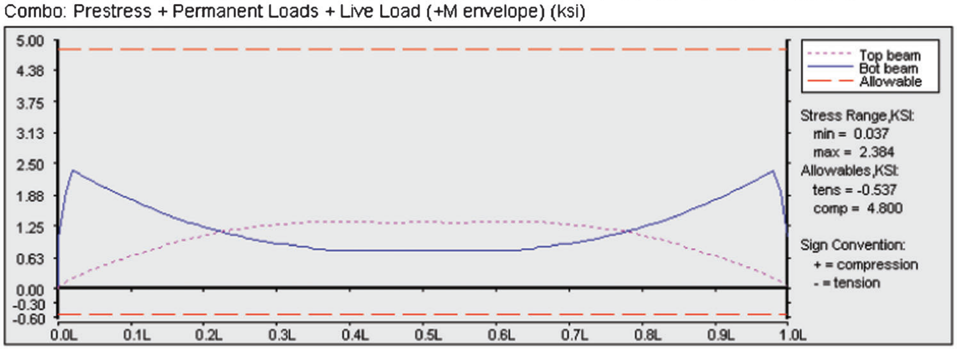
Figure F.1 PSBeam strand pattern design.



Final Stresses

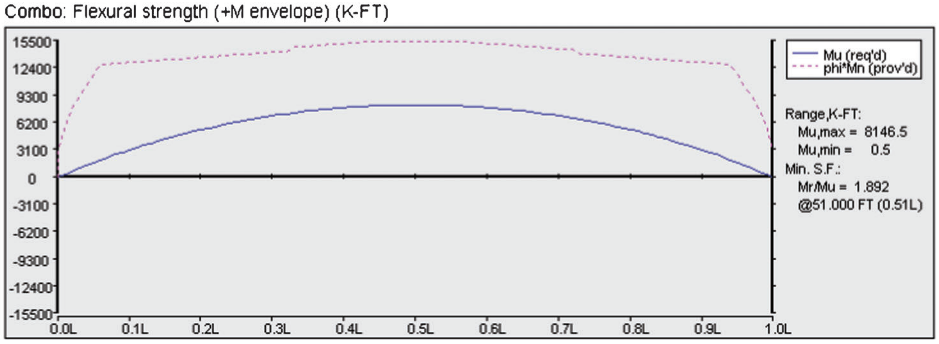
Final losses: ES= 18.604 ksi, RE= 1.544 ksi, SH= 5.750 ksi, CR= 31.315 ksi, Total= 57.213 ksi (28.3%)

Combo: Prestress + Permanent Loads + Live Load (+M envelope) (ksi)



Flexural Strength

Combo: Flexural strength (+M envelope) (K-FT)



Vertical Shear Strength

Required vs. provided shear steel(in2/ft)

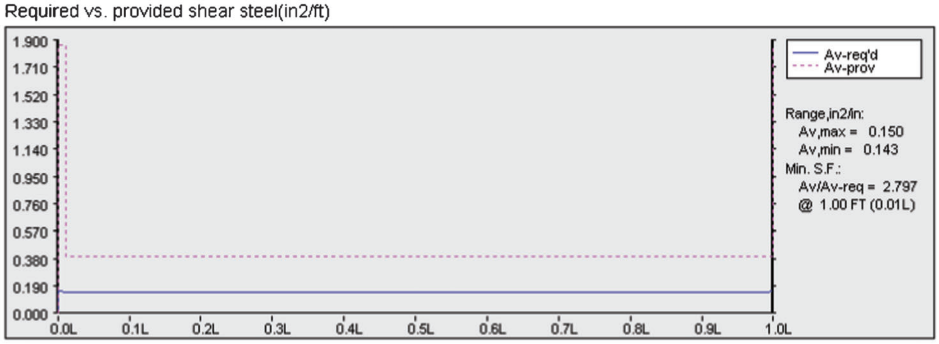


Figure F.2 PSBeam girder design output.