Report No. K-TRAN: KU-01-2
FINAL REPORT

CROSS-FRAME DIAPHRAGM BRACING OF STEEL BRIDGE GIRDERS

W. M. Kim Roddis, Ph.D., P.E.
Soudeh Baghernejad
George Washington University
Washington, District of Columbia

Erika L. Winters
The University of Kansas
Lawrence, Kansas

January 2008

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:

KANSAS DEPARTMENT OF TRANSPORTATION
KANSAS STATE UNIVERSITY
UNIVERSITY OF KANSAS

1 Report No. K-TRAN: KU-01-2	**2 Government Accession No.**	**3 Recipient Catalog No.**
4 Title and Subtitle Cross-Frame Diaphragm Bracing of Steel Bridge Girders		**5 Report Date** January 2008 **6 Performing Organization Code**
7 Author(s) W. M. Kim Roddis, Ph.D., P.E., Erika L. Winters, Soudeh Baghernejad		**8 Performing Organization Report No.**
9 Performing Organization Name and Address Kansas University Transportation Center The University of Kansas 1530 W 15th Street, 2150 Learned Hall Lawrence, KS 66045-7609		**10 Work Unit No. (TRAIS)** **11 Contract or Grant No.** C1268
12 Sponsoring Agency Name and Address Kansas Department of Transportation Bureau of Materials and Research 700 SW Harrison Street Topeka, Kansas 66603-3745		**13 Type of Report and Period Covered** Final Report Spring 2001 - Summer 2004 **14 Sponsoring Agency Code** RE-0242-01
15 Supplementary Notes For more information write to address in block 9.		

16 Abstract

Steel plate girder bridges make use of traditional cross-frame diaphragms to stabilize the compression flange of girders. These braces are required during construction, especially during deck placement, to prevent lateral torsional buckling of bridge girders. Girder buckling capacity is a function of cross-frame diaphragm spacing as well as strength and stiffness. Recent developments in bridge design may cause the governing girder limit state to shift from one of strength to one of stability. These developments include the elimination of in-plan bracing, composite girders, High Performance Steels, and phased deck replacements. In addition, the American Association of State Highway and Transportation Officials (AASHTO) has changed its code requirement for cross-frame diaphragm spacing in the 1998 AASHTO LRFD Bridge Design Specifications. The requirement for 25-foot maximum brace spacing has been removed. The current requirement is for a “rational analysis” to determine cross-frame diaphragm spacing. Explanations of the problems these changes cause in design are discussed. A case study is presented of a bridge that suffered construction difficulties during deck placement. This investigation found that the cross-frame diaphragms were not stiff enough to brace the plate girders during the deck placement. Suggestions are given as to an efficient, economical design and spacing for cross-frame diaphragms on plate girder bridges.

17 Key Words Steel Girder, diaphragm bracing, bridge design, plate girder bridges		**18 Distribution Statement** No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19 Security Classification (of this report) Unclassified	**20 Security Classification (of this page)** Unclassified	**21 No. of pages** 59	**22 Price**

Form DOT F 1700.7 (8-72)

CROSS-FRAME DIAPHRAGM BRACING OF STEEL BRIDGE GIRDERS

Final Report

Prepared by

W. M. Kim Roddis, Ph.D., P.E.
Soudeh Baghernejad
George Washington University
801 22nd Street NW
Washington, DC 20052

and

Erika L. Winters
Kansas University Transportation Center
The University of Kansas
1530 W. 15th Street, 2150 Learned Hall
Lawrence, KS 66045-7609

A Report on Research Sponsored By

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

January 2008

PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

NOTICE

The authors and the state of Kansas do not endorse products or manufacturers. Trade and manufacturers' names appear herein solely because they are considered essential to the object of this report.

This information is available in alternative accessible formats. To obtain an alternative format, contact the Office of Transportation Information, Kansas Department of Transportation, 700 SW Harrison, Topeka, Kansas 66603-3745 or phone (785) 296-3585 (Voice) (TDD).

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the views or the policies of the state of Kansas. This report does not constitute a standard, specification or regulation.

ABSTRACT

Steel plate girder bridges make use of traditional cross-frame diaphragms to stabilize the compression flange of girders. These braces are required during construction, especially during deck placement, to prevent lateral torsional buckling of bridge girders. Girder buckling capacity is a function of cross-frame diaphragm spacing as well as strength and stiffness. Recent developments in bridge design may cause the governing girder limit state to shift from one of strength to one of stability. These developments include the elimination of in-plan bracing, composite girders, High Performance Steels, and phased deck replacements. In addition, the American Association of State Highway and Transportation Officials (AASHTO) has changed its code requirement for cross-frame diaphragm spacing in the 1998 AASHTO LRFD Bridge Design Specifications. The requirement for 25-foot maximum brace spacing has been removed. The current requirement is for a “rational analysis” to determine cross-frame diaphragm spacing. Explanations of the problems these changes cause in design are discussed. A case study is presented of a bridge that suffered construction difficulties during deck placement. This investigation found that the cross-frame diaphragms were not stiff enough to brace the plate girders during the deck placement. Suggestions are given as to an efficient, economical design and spacing for cross-frame diaphragms on plate girder bridges.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER ONE - PROBLEM STATEMENT

1.1 Research Problem Statement

Steel girder bridges require adequate lateral support of compression members to provide stability under all construction phases. The critical condition for lateral stability is usually during concrete deck placement. For bridge girders, the girder top flanges are primarily in compression, so provisions must be made for adequate stability of the top flange. Care must be taken to determine the actual limit states governing behavior of girders. Changes from past experience can be expected when implementing advances such as composite design and high performance steels. Implementing these design advances can lead to use of less steel with a possible accompanying change in governing limit state from a strength driven to a deflection driven behavior. Particular care needs to be taken in evaluating the effect of such changes in design approach on the construction phase of girder bridges. The construction load phase controls the design of the composite girder's top flange in the positive moment region. There are few specifications available regarding the design of temporary shoring and bracing. Failures in these areas occur often and can be very costly [Duntemann].

1.2 Background

Beam buckling involves both flexure and torsion, a behavior referred to as lateral torsional buckling [Galambos]. Beam bracing may be generally categorized as either lateral bracing or torsional bracing. Lateral bracing restrains lateral displacement. Torsional bracing restrains twist. For completed bridge structures, the in-place concrete deck provides continuous structural lateral bracing for bridge girders. Under construction, when the deck is not present or is present as a load but not yet as a load-resisting element, alternate means of bracing must be provided. In-plan bracing, sometimes referred to as “wind bracing”, may be used to provide lateral bracing. However, the elimination of this in-plan steel bracing, a common trend in bridge superstructure design, places an increased importance on the bracing role of cross-frame diaphragms during deck placement. Cross-frame diaphragms act as torsional braces during concrete placement. Although cross-frame diaphragms are able to displace laterally, they still function as brace points because twist of the bridge cross-section is prevented. To act as effective braces, the cross-frame diaphragms must be both strong enough and stiff enough. Design for strength requires correct determination of the required bracing force. Design for stiffness requires correct determination of the restraint required to prevent lateral torsional buckling in beams (Figure 1.1). The stiffness of the torsional bracing system is dependant on the vertical stiffness of the girders. Increasing the strength of the cross-frame diaphragms alone will not solve a stability problem due to insufficient system stiffness. Methods are available to determine both strength and stiffness for cross-frame diaphragms of composite girder bridges.

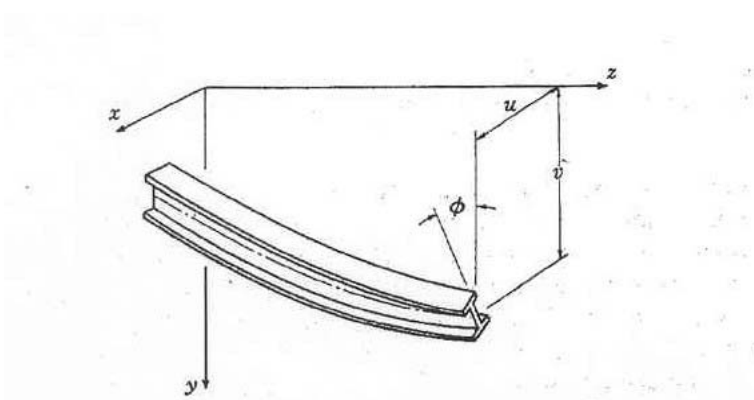


FIGURE 1.1: Lateral Torsional Buckling [Nethercot]

The Kansas Department of Transportation (KDOT) has several concerns regarding lateral bracing design [Jones]. The three concerns have to do with 1) consequences of the AASHTO code elimination of bracing requirements on new bridge design, 2) deck replacement, and 3) skewed bridges.

KDOT's primary concern stems from the AASHTO code eliminating bracing requirements first for in-plan bracing and more recently for a maximum 25 foot brace spacing. In the 1985 AASHTO code [AASHTO], in-plan wind bracing was eliminated as a requirement. This reflected the change in standard assumptions concerning how wind load was carried in a girder bridge. Previously, horizontal wind load acting on the vertical face of the girders was assumed to be carried by in-plan wind bracing spanning horizontally between bearings. Current practice is to assume wind on the upper half of the girders is resisted directly by the concrete deck and wind on the lower half of the girders is resisted by flexure of the bottom flange, either between brace points or bearings [Mertz].

In the most recent version of the code, the 1998 AASHTO LRFD, the stipulation that cross-frame diaphragms be placed at intervals not greater than 25 feet was also removed and replaced with a requirement to space braces using a "rational analysis." KDOT experience has been that the 25-foot stipulation was adequate and that performing a "rational analysis" including an influence surface analysis [AASHTO LRFD C4.6.3.3] for every job is impractical. KDOT has concerns that without these requirements, the known design envelope is being pushed.

Currently, KDOT continues to use a maximum brace spacing of 25 foot. A check is performed limiting the slenderness ratio, kl/r, of brace members to less than 120.

KDOT has consultants design about 70 % of their bridges, so it is of importance that the consultants are up to date on their methods of designing braces.

KDOT's second concern is the different problems arising with a deck replacement. The most common situation involves the replacement of an old non-composite deck with a new composite one. The deck is removed, shear studs are added to the girders, and then a new deck is placed. In this situation, KDOT would prefer to use temporary braces during deck placement instead providing new permanent braces. However, the code stipulates that in some situations new permanent bracing is necessary [A10.20.1 AASHTO LRFD]. In addition, bridges designed with the old rule of 25 foot maximum brace spacing may be inadequate if analyzed according to the new code.

KDOT's third concern is the issue of skewed bridge supports. When a bridge is on a skew of greater than 20 degrees, AASHTO requires that the cross-frame diaphragms be placed perpendicular to the web, not on the skew. KDOT regards this as impractical; if cross-frame diaphragms are placed on the skew it makes for easier erection and also avoids fatigue-prone situations. KDOT would like this issue explored and a solution proposed.

Ultimately, a straightforward, easy way to design and place the braces that conforms to the AASHTO code is desired. KDOT's eventual target is development of: a) a conservative computer design aid for spacing cross-frame diaphragms, b) prototype details that meet the requirements of the current code, perhaps using MC channels instead of double angles for brace members, incorporating bolted connections to avoid fatigue problems, and c) a typical set of specifications [Jones].

1.3 Organization of Report

This paper first presents a literature review addressing KDOT's current concerns in designing lateral bracing. This review looks at the function, design, and current practices with regard to cross-frame diaphragms. A survey of the Steel Bridge Collaboration (SBC) determines current methods used around the country to provide a safe and economic brace design. The consequences of the use of High Performance Steel (HPS) and the effects of skewed supports with regard to bracing are discussed. The MASTAN2 [McGuire] structural analysis program is introduced as a computational modeling tool is used to model several classic column and beam buckling examples. A case study of girder bracing design and performance is then investigated using both this computational analysis tool and an analytic method. This case study provides a specific example where the cross-frame diaphragms were not stiff enough to adequately brace the plate girders during the deck placement.

Finally, results and recommendations are provided in a form suitable for implementation by KDOT. These include straightforward, simple methods of determining brace spacing and design for girder bridges.

CHAPTER TWO - LITERATURE REVIEW

2.1 Function of cross-frame diaphragms

Cross-frame diaphragms perform three main functions: transfer of lateral wind loads from the deck to the bearings, distribution of vertical dead load and live load to the longitudinal beams or girders, and a stable brace point for beam or girder flanges during erection and placement of the deck [A6.7.4.1 AASHTO LRFD].

Current practice is to assume wind on the upper half of the girders is resisted directly by the concrete deck and wind on the lower half of the girders is resisted by flexure of the bottom flange, either between brace points or bearings [Mertz].

Cross-frame diaphragms aid in the redistribution of live loads [Nethercot]. It has been shown that after construction the slab alone is adequate to distribute live loads, thus some researchers recommend cross-frame diaphragms should be designed for easy removal after construction [Azizinamini]. However, cross-frame diaphragms provide redundancy of load path and for this reason some engineers are reluctant to get rid of them altogether. Different sources deal differently with the role of cross-frame diaphragms in distributing live loads among the girders. The National Cooperative Highway Research Program (NCHRP) recently released Guide Specifications for Distribution of Loads for Highway Bridges with equations having no provision for the presence of cross frames [Mertz]. The AASHTO LRFD Specifications have similar equations, but with a provision giving a benefit to those bridges with cross-frame diaphragms.

After construction cross-frame diaphragms have little use and are even harmful in some cases. Cross-frame diaphragms are sometimes thought to have a benefit when dealing with collision loading, however this is doubtful due to the high loading rate [Mertz]. Out-of-plane distortion fatigue cracking may develop in plate girder webs at cross-frame diaphragm connection plates [Zhao]. Stiff torsional braces develop large localized forces during truck loading, particularly for skewed supports. These large brace forces combined with complex connection details to the girder webs usually lead to large stress concentrations that result in fatigue problems. A major reason for the removal of the spacing limit in the AASHTO LRFD is due to the fatigue problems that are often found around the brace locations. [Wang & Helwig]. For this reason, in some cases temporary bracing that is present during and deck placement may be preferable to permanent bracing.

The scope of this report examines the third function, cross-frame diaphragms as brace points for the girder compression flanges to reduce the buckling length of bridge plate girders. Bracing of any horizontal member must be designed to satisfy two limits, strength and stiffness. AASHTO requires that all limit states must be taken into account in order to achieve an acceptable design, but it is silent on the specific methods of analysis for determining brace strength and stiffness [AASHTO LRFD].

Brace strength is often quantified in terms of the load that the brace must carry in order to make it an adequate support point for the member. In the American Institute for Steel Construction's (AISC) Load and Resistance Factor Design Specifications for Buildings, a brace point is required to carry 0.4% of the factored load, P_u, in the member to be braced. This force is usually very small and can be achieved in a member with minimal material [AISC].

Brace stiffness, is defined in units of force per length, for a lateral brace, or in units of force per rotation, for a torsional brace. For a pin-ended, concentrically loaded column of length L with a lateral brace at stiffness β at its top, when the member reaches its critical, or Euler, buckling load P_{cr}, the brace force F is β times L (Figure 2.1).

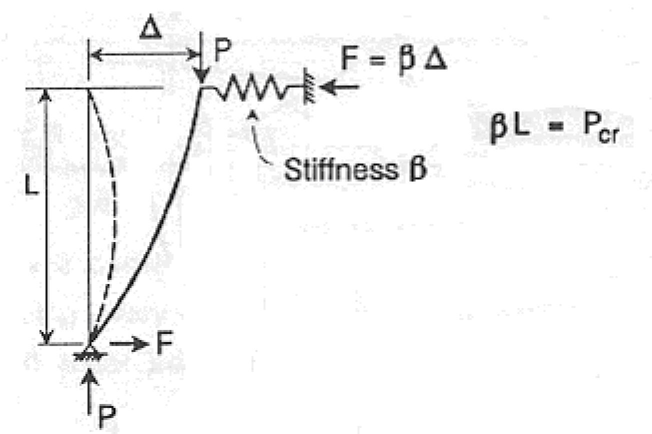


FIGURE 2.1: Column with top brace [Yura]

If a member is braced only with this ideal stiffness, the strength requirement of the brace will tend toward infinity, as shown in Figures 3 and 4. In Figure 3, the bracing force is plotted as a function of the brace stiffness proved relative to the ideal brace stiffness. A practical limit is to establish a minimum brace stiffness of twice β, which corresponds to required brace strength of 0.4% P_{cr} (Figure 4). The graph in Figure 5 shows that by reducing beam design load levels, one can significantly reduce the brace force required. The factored load that the beam was designed for, P_u, used 81% of the material capacity of the brace. By reducing the design load level to 90%, one can reduce the stress in the brace to 29% of the yield capacity.

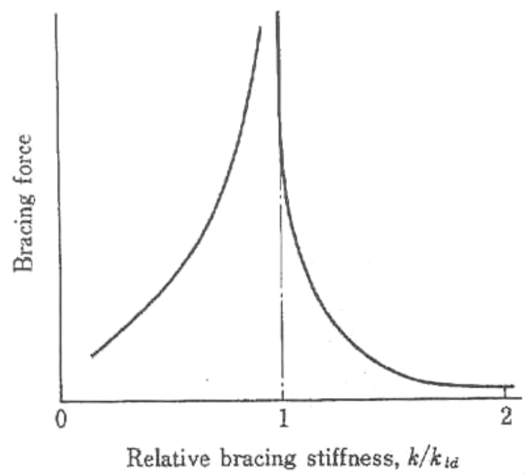


FIGURE 2.2: Relationship between brace force and brace stiffness [Nethercot]

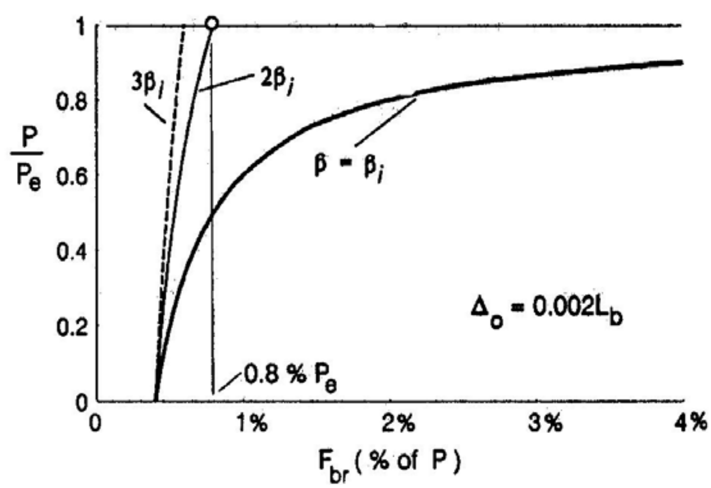


FIGURE 2.3: Brace force as a function of critical load [Yura]

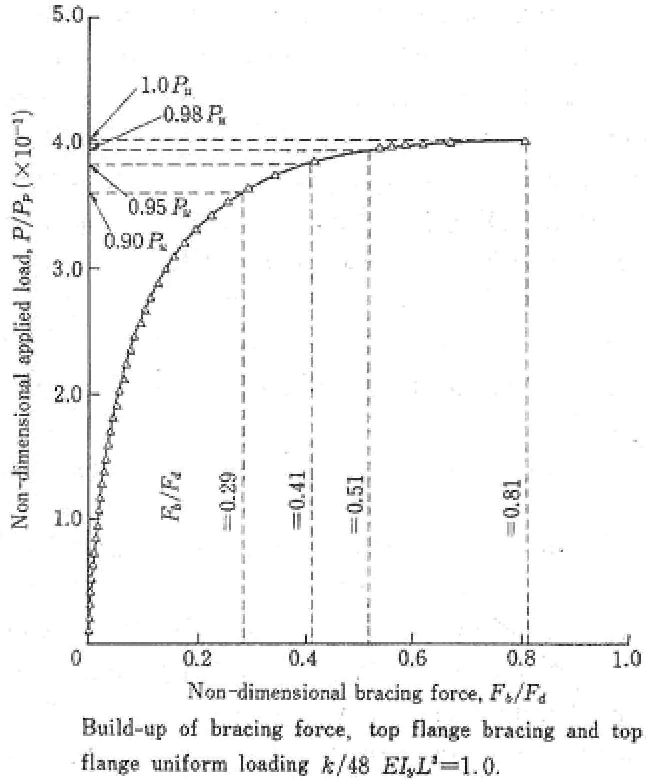


FIGURE 2.4: Relationship between percentage of Euler Buckling load and brace force required [Nethercot]

If a beam brace has enough stiffness, the strength requirement may be 1% of the axial flange force if there is a single brace. For a frame with many braces, 2% is a valid number [Wang].

2.2 Design of cross-frame diaphragms

The basic function of cross-frame diaphragms is to reduce the slenderness ratio of bridge girders. Cross-frame diaphragms provide the torsional brace points needed to stabilize the compression flange during construction. They provide stability in negative moment regions (above supports) at all times [Nethercot]. Properly designed braces, are necessary to ensure steel girder sections perform adequately.

Cross-frame diaphragms must be both strong and stiff enough to provide a sufficient brace point. Strength resists forces by limiting deformations. Stiffness modifies the buckled shape of the member, increasing its load carrying capacity. At the ideal stiffness limit, the force required in the brace is infinity; therefore a brace must have twice the ideal stiffness as a practical limit. For brace stiffness satisfying this practical limit, if beam design loads are reduced, lower brace strength is needed. The force that the brace can carry is also affected by the influence of imperfections, residual stresses, and initial out-of-straightness [Nethercot].

Stiffness, β, depends on the bracing type, beam geometry, beam load type, bracing elevation, and load elevation. Of particular concern for torsional bracing is the bracing elevation. In lateral torsional buckling, the compression flange moves more than the tension flange. An effective torsional brace is placed at the mid-depth of the web, not at the top or bottom, so as to achieve maximum torsional resistance.

Dennis Mertz has published a paper on designing cross-frame diaphragms for bridges which is becoming widely used for simple, non-skewed bridges [Mertz]. He states that designing cross-frame diaphragms takes many different factors into account and often involves many compromises of economy. When there are more closely spaced braces, girder flange size decreases. Designers often want to utilize a design that produces the least weight; however least weight does not lead to least cost. He suggests that standardization of a few common flange sizes could be very economical. Stronger compression flanges could reduce or even eliminate the need for cross-frames.

The methods Mertz presents for designing cross-frame diaphragms are covered in more detail in section 5 of this paper. Mertz addresses cross-frame diaphragm spacing, configuration, and sizing but does not address the critical issue of brace stiffness. Designers who rely solely on his paper thus have an incomplete approach to design of cross-frame diaphragms. The Steel Bridge Collaboration [SBC], a joint effort of AASHTO and AISC's National Steel Bridge Association (NSBA) was surveyed through its e-mail distribution list to get input from around the country on how bridge designers are dealing with the new requirement of a "rational analysis" to design and place bracing. It was found that Minnesota, as a state, still uses by the 25-foot limit. New Jersey has adopted the NSBA details designed by the Steel Committee for Economical Fabrication, as these details conform to the new code. Other responses said that for straight bridges, the 25-foot limit was still used, while for skewed bridges the design engineer might utilize a design program, such as MDX [MDX] or other software that allows the input of skewed supports in the geometric layout. AISC suggested Dennis Mertz' paper for brace design [NSBA]. As can be seen from these responses, there is not a consensus on the appropriate changes to make in design practice in response to the changes in AASHTO requirements brace spacing. Of more concern is the apparently lack of perception of the need to consider both brace strength and stiffness for girder stability, and that these needs are not and were not addressed by the 25-foot requirement.

2.3 High Performance Steel

Use of High Performance Steels (HPS) can lead to a change in governing limit state from one that is strength driven to one that is stiffness driven. Since all constructional steels have essentially the same elastic modulus, use of higher strength steels would be expected to lead to larger deflections. Higher strengths cannot be fully utilized if a deflection or stability limits state controls. If this change in limit state is unforeseen, problems will arise. For example, since the amount of restraint provided by torsional braces in the form of cross-frame diaphragms is dependant on the vertical stiffness of the girders, an increase in girder steel strength which reduces girder size and vertical stiffness, can lead to reduced resistant to lateral torsional buckling during construction.

Current AASHTO specifications restrict the use of HPSs due to lack of sufficient test data revealing its capabilities. Several researchers have explored HPS with regard to standard bridge design practice and code provisions. HPS 70 girders have been tested to see if AASHTO strength ranges can be expanded. Lower carbon content improves the weldability of this steel. It also has a high toughness and resistance to brittle fracture. This paper finds that the strength limitations should be lifted for 70-ksi steel. Tests have been conducted to compare the ultimate moment capacity and inelastic rotation behavior of 50 and 70-ksi steels. Specimen girders, braced at the elastic limit, were tested as simply supported with a central point load. Compact and non-compact specimens were tested. Analysis shows that larger than anticipated forces were developed at brace points. Both 50 and 70-ksi steels were able to meet and exceed AASHTO strength standards. Compact 70 ksi girders were able to reach full plastic capacity. Non-compact 70-ksi girders were able to reach elastic yield capacity. Compact 70 ksi girders are unable to provide adequate inelastic rotational capacity as required by AASHTO. Therefore, the 10% moment redistribution should not be allowed for non-compact 70-ksi plate girders [Azizamini].

The applicability of AASHTO compact section criteria for HPS has also been investigated. For girders of grade 36 and 50 steel, local (flange or web) and global (lateral-torsional) buckling of I-beams are treated separately. It has been observed that with sections of HPS grade 70 and 80 steel, these two buckling behaviors occur simultaneously. This report investigates the slenderness ratios in the AASHTO manual as applied to grade 70 steel. It finds that the ratios are unconservative. It is assumed that if a section meets compact criteria it will be fully able to develop a plastic hinge rotation of three and collapse. The AASHTO slenderness requirements are $b_f/2t_f$ of 7.75 for flanges and $2D_{cp}/t_w$ of 76.5 for webs. Several girders that meet these criteria were tested and none met the rotational criteria of three. The vast majority only achieved half of this rotation before failure. This presents an apparent lack of correlation between slenderness ratios and rotation capacity. Two methods were taken to correct this behavior. HPS 70 girders with a flange ratio of 4.5 and a web ratio of 90 were able to meet the rotation criteria with an altered brace placement. Stiffeners are moved from the regular spacing of about 7 meters to 0.375 meters around the load point. The flange criteria is stricter while the web criteria is looser. Based on these results, Earls suggests changing the compact section criteria and brace spacing for HPS 70 steel [Earls].

2.4 Bridges with skewed supports

AASHTO specifies that if a bridge is skewed less than 20 degrees, braces may be placed on the skew (Figure 2.5). If the skew is greater than 20 degrees, braces must be placed perpendicular to the girders. When this happens, the two ends of the braces will develop unequal forces and displacements. This can worsen the fatigue issue.

A second consequence of skew is the method of determining adequate brace stiffness. As previously mentioned, all braces must have adequate stiffness, with a practical limit of twice the ideal value. For bridges with non-skewed supports, brace stiffnesses may be evaluated using the equations provided by Yura [Yura]. For braces in even highly skewed bridges that are erected perpendicular to the girders, the original equations can be applied without modification. However, for braces placed on the skew, the equations [Yura] need to be adjusted as follows [Wang& Helwig]. For bridges with braces placed on the skew, the brace stiffness is reduced to β cos^2ϕ where ϕ is the angle of skew. The moment developed in the brace is magnified to Mbr/cosϕ.

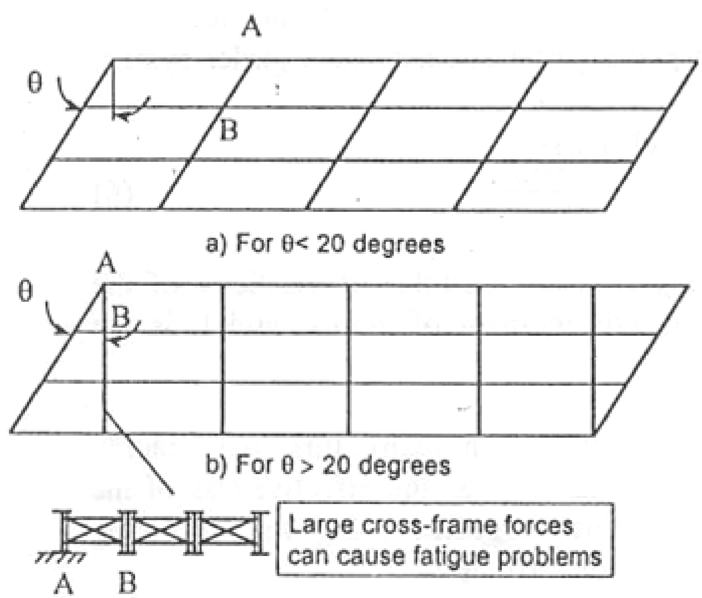


FIGURE 2.5: Braces on skew (top) and perpendicular to girders (bottom) [Wang& Helwig]

CHAPTER THREE - METHODS OF ANALYSIS

3.1 Background

The MASTAN2 structural analysis software is used to perform elastic critical buckling load analysis for this report. [McGuire] This software analyzes two and three-dimensional frames of various materials using a traditional stick modeling approach. It has the capability to analyze a structure for elastic and inelastic critical buckling loads. The software will also accept Matlab programs for more complicated analyses.

3.2 Design Cases

Several models of structural buckling problems with known solutions [Yura] are built and tested in MASTAN2 in order to gain facility with this software as well as verify its appropriateness for use on this project. The first model deals with a column braced at its top. The second model deals with a column braced at mid-height. These models are chosen because they directly demonstrate basic brace theory and show the software's ability to calculate the critical buckling modes of structures while taking into account brace stiffness.

The first column modeled is a 20-foot tall column with a pinned base and a flexible brace at the top (Fig. 3.1). The column is modeled with the properties of a W8x31 rolled section for major axis buckling, with a yield stress of 50 ksi. A concentric axial load of 1 kip is applied to the top of the column, since MASTAN2 will then compute the load ratio for column buckling. MASTAN2 does not have specific built-in boundary conditions for spring supports. Therefore, the flexible top brace is modeled as grade 50 steel with properties of E, A, and L set to achieve the stiffness desired. As the column is loaded axially, the brace shortens as a linear spring support.

In the first example brace stiffness values β are compared with the ideal brace stiffness β_i. When β is less than β_i, the buckling load increases as brace stiffness increases. However, when β is greater than β_i the buckling load remains constant as brace stiffness increases. This behavior is illustrated in Figure 3.2. The relationship is presented mathematically as follows:

For $\beta < \beta_i$, $P_{cr} = \beta L$ and,

For $\beta \geq \beta_i$, $P_{cr} = \dfrac{\pi^2 EI}{L^2}$

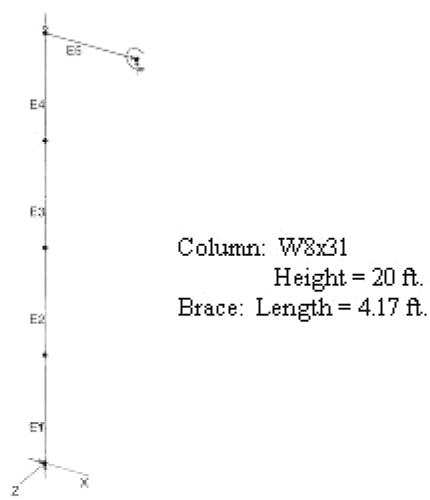


FIGURE 3.1: MASTAN2Column model #1

For this case, the ideal brace stiffness is equal to the Euler critical load divided by the length of the column, $\beta_i = 2.277$ k/in. For this example, the analytic solution is taken to be that given using the formulas in [Yura]. The Euler load is 546.6 kips. The practical difficulty with this situation is that when a member is braced with the ideal stiffness, the force in the brace is infinite and the full axial capacity of the member can never be realized. When the brace stiffness is twice the ideal value, the required brace force is less than 1% of P_{cr}, a realistic value.

To model the different modes, the area of the brace was varied while the brace length and modulus of elasticity were held constant. This is to say $\beta = \frac{EA}{L}$. See Table 1 for parameter values. Results are graphed in Figure 3.3. These results match very well with the theoretical solutions shown in Figure 3.2.

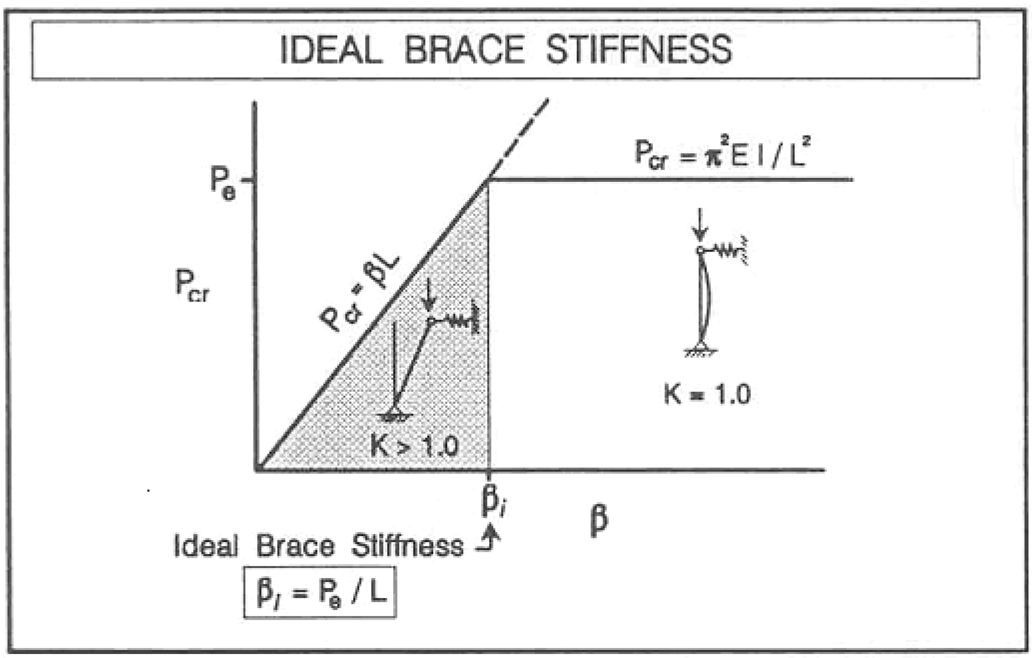


FIGURE 3.2: Column with top brace ideal stiffness, β_{ideal} [Yura]

Using MASTAN2, the brace is modeled first with a stiffness of 2 k/in., resulting in a computed buckling load of 480 kips (Figure 3.4). This is consistent with the analytic formulas. The buckling load of the second mode is 546.6 kips, shown in Table 3.1. This corresponds to the case of a brace with only the ideal stiffness, β_i.

For the second run, the brace area was varied to change β to 4.55 k/in, twice the ideal value, resulting in a buckling load 546.8 kips (Figure 3.5), and closely corresponding to the analytic result of 546.6 kips.

TABLE 3.1: Buckling load calculations

Model	Length of the column	Brace length	Stiffness, β (k/in)	Area (in^2)	Buckling Load (kips)
	20 ft	4.17 ft ,or 50 in	0	0	0
	20 ft	4.17 ft ,or 50 in	1	0.00172	239.424
	20 ft	4.17 ft ,or 50 in	2	0.00344	480
Mode 1	20 ft	4.17 ft ,or 50 in	2.277	0.003926	546.48
	20 ft	4.17 ft ,or 50 in	3	0.00516	546.877
	20 ft	4.17 ft ,or 50 in	4	0.00688	546.8778
Mode 2	20 ft	4.17 ft ,or 50 in	4.555	0.007844	546.877

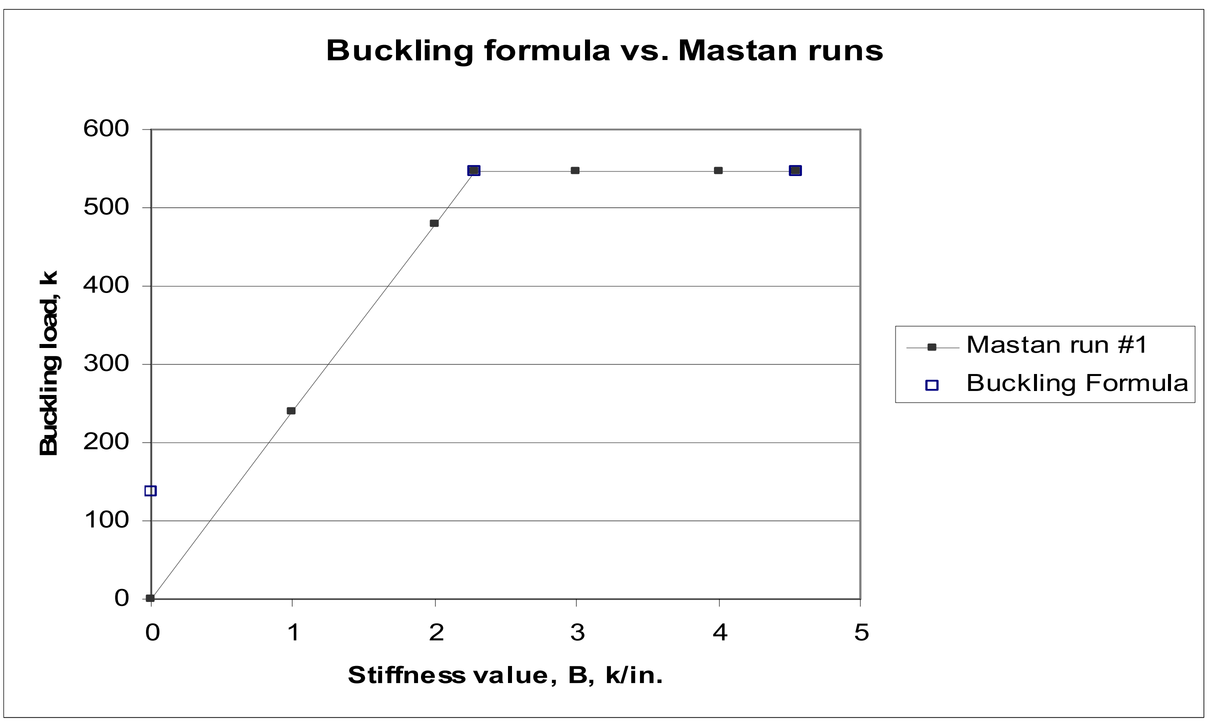


FIGURE 3.3: MASTAN2 column #1, various runs relating buckling load (k) to stiffness (k/in.)

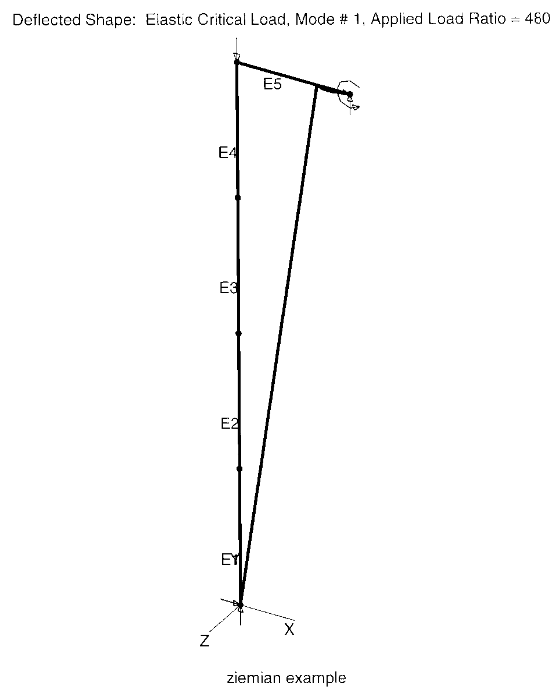


FIGURE 3.4: MASTAN2 column #1, with stiffness of 2 k/in., and buckling load of 480 k.

FIGURE 3.5: MASTAN2 column model #1, with critical buckling load of 546.8 k.

The next column example is similar to the first except the column is braced at mid-height as shown in Figure 3.6. The column is modeled with the properties of a W16x26 rolled section for major axis buckling, with an overall length of 20 ft. and a yield stress of 50 ksi. A concentric axial load of 1 kip is applied to the top of the column, since MASTAN2 will then compute the load ratio for column buckling. In this case, the unbraced length, L_b, of the column is only half the height of the column, that is to say 10 ft. The length of the brace is 60 in. Figure 3.7 shows the behavior as the brace stiffness is varied. The Euler buckling load for this column is found using the formula in Figure 3.7 to be 5983 k, with a corresponding β_i of 99.7k/in.

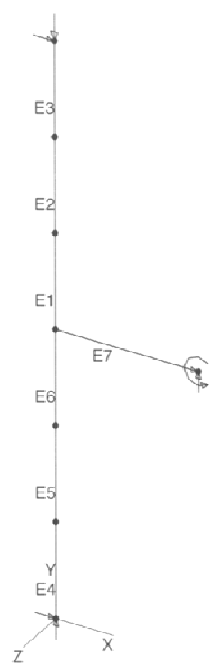


FIGURE 3.6: MASTAN2 column model #2.

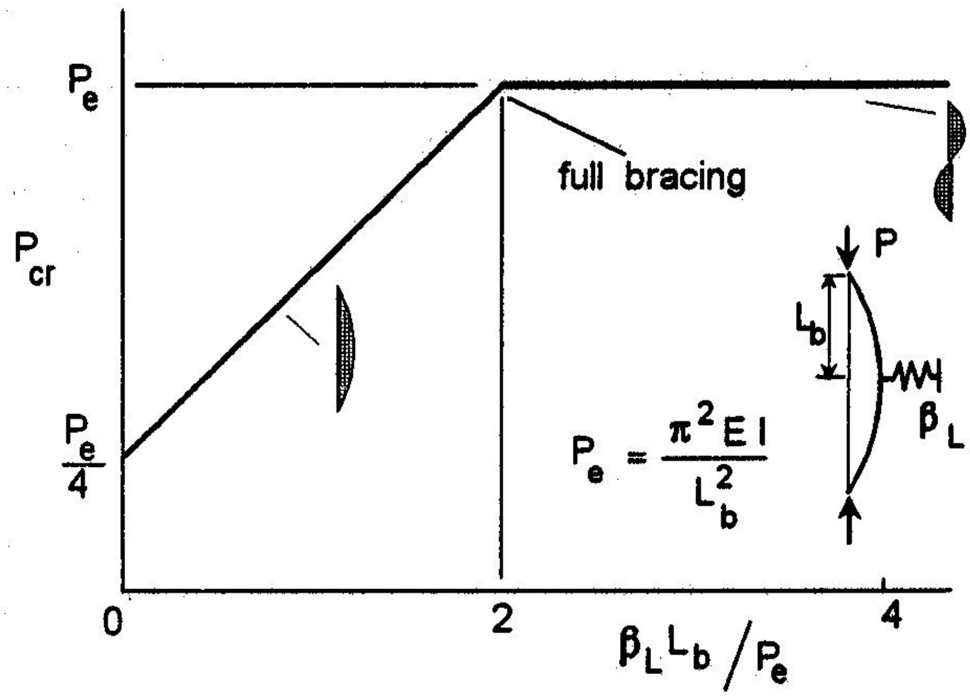


FIGURE 3.7: Critical buckling load vs. stiffness for column braced at mid-height [Yura]

The support is modeled first with a β of 50 k/in., about half the ideal value. The column buckles (in the second mode, since the first mode corresponds physically to the brace not holding and the column buckling over its entire length) at a load of 3843k (Figure 3.8). The buckling load is found with the equation shown in Figure 3.7, with $P_e/4$ of 1496 k. The buckling load found with the equation is 1496 + 0.375*βL_b, a value of 3746 k. The MASTAN2 value of 3843k and the formula value of 3746 are within 2.5% of one another. When the brace is modeled with the ideal stiffness of 99.7 k/in, the column-buckling load is 5992 k. (Figure 3.9). Figure 3.10 shows the results of a sequence of models run with braces of varying stiffness. Parameter values are listed in Table 3.2. These results match very well with the known equations (Figure 3.8).

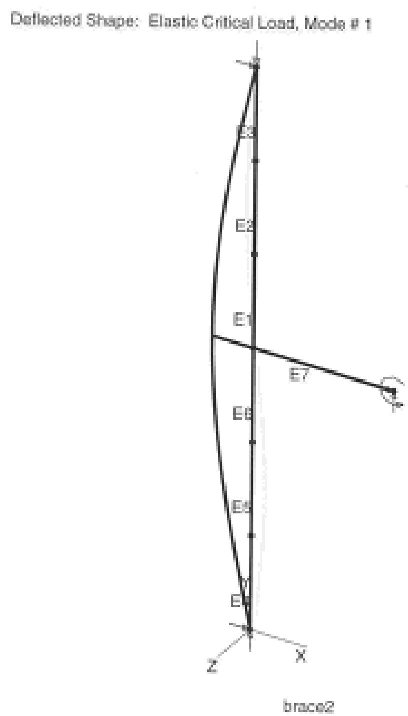


FIGURE 3.8: MASTAN2 column model #2, buckling load of 3843 k with a   of 50k/in.

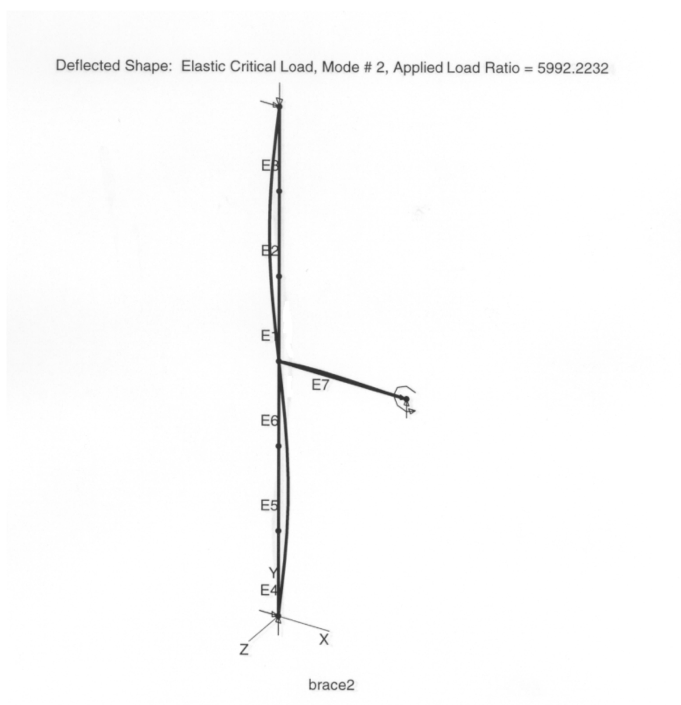


FIGURE 3.9: MASTAN2 column model #2, critical buckling load of 5992 k

TABLE 3.2: Buckling load calculations

Model	Length of the column	Brace length	Stiffness, β (k/in)	Area (in^2)	Buckling Load (kips)
	20 ft	5 ft ,or 60 in	0	0	1496
	20 ft	5 ft ,or 60 in	50	0.00172	3843
	20 ft	5 ft ,or 60 in	80	0.00344	5096
Mode 1	20 ft	5 ft ,or 60 in	99.7	0.003926	5992.22
	20 ft	5 ft ,or 60 in	100	0.00516	5992.22
	20 ft	5 ft ,or 60 in	150	0.00688	5992.22
Mode 2	20 ft	5 ft ,or 60 in	200	0.007844	5992.22

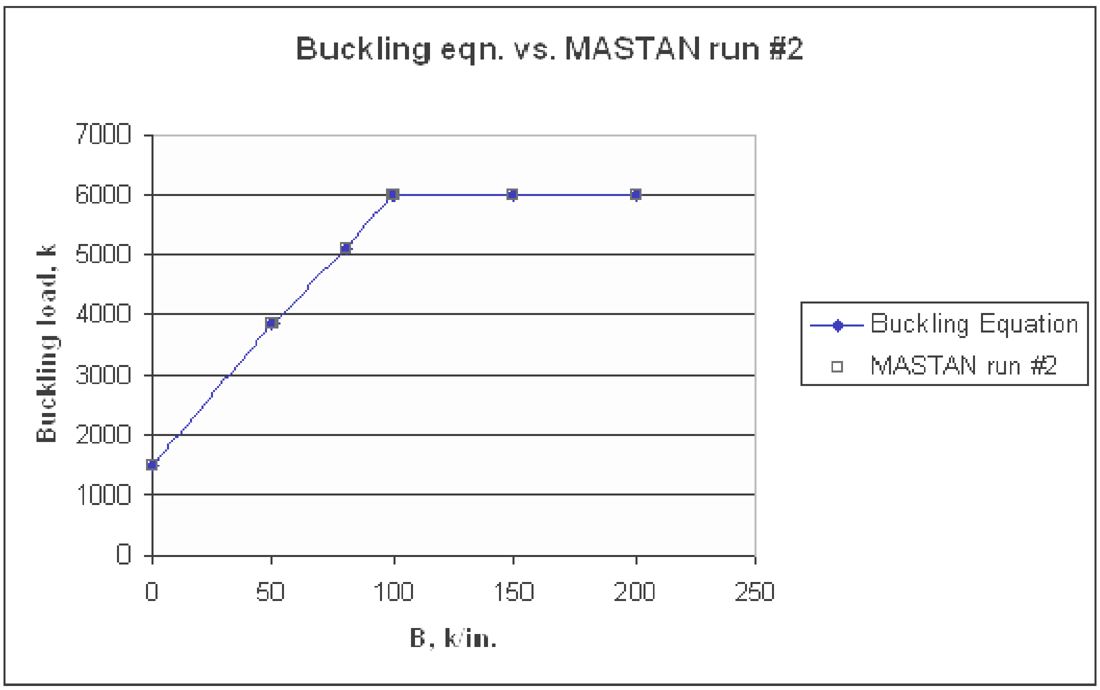


FIGURE 3.10: MASTAN2 column model #2, buckling load vs. stiffness.

To further verify the correct usage of the software, a typical wide-flange beam was also modeled. The purpose of this model is to compare the MASTAN2 critical buckling length and load to those based on capacity equations specified in the LRFD manual [AISC]. For a W16x40 rolled section with a yield stress of 50 ksi, the LRFD manual lists the limiting inelastic laterally unbraced length of 5.6 ft, and the elastic limiting laterally unbraced length as 14.7 ft. For an unbraced length of 14.7, the maximum moment capacity, ϕM_r, is 194k-ft, or 2328k-in. Dividing out the resistance factor, the limiting moment is 2328/0.9, or 2586k-in.

The beam modeled is a W16x40 rolled section torsionally restrained at each end to prevent it rotating about its own axis. The beam's span is set at 14.7 feet in MASTAN2 (Figure 3.11). With concentrated moments of 1 k-ft applied to each end the results were (Figure 3.12), within 1% (2563.9 k-in.) of the LRFD equation specified capacity.

With a distributed load of 1 k-in., the simple span moment is 3889.6 k-in. The elastic buckling load ratio determined by MASTAN2 for this uniform loading is 0.7617. Using this ratio to compute the elastic buckling moment gives 0.7617 times 3889.6 k-in., or 2913.5k-in., within 13% of the code-specified value. When one divides this value by the C_b value for a uniformly loaded beam of 1.136, the results are within 1.6% of the LRFD equation specified capacity.

Again, MASTAN2 results are verified. Modeling with actual bridge case study can now be done.

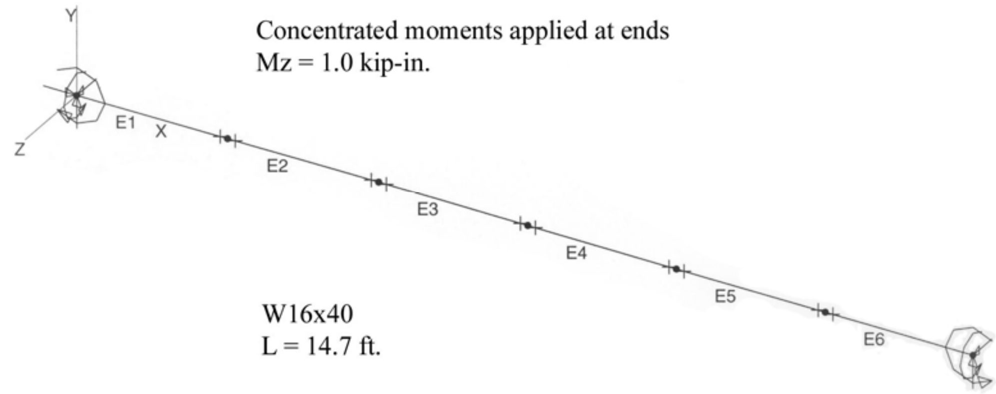


FIGURE 3.11: MASTAN2 beam model

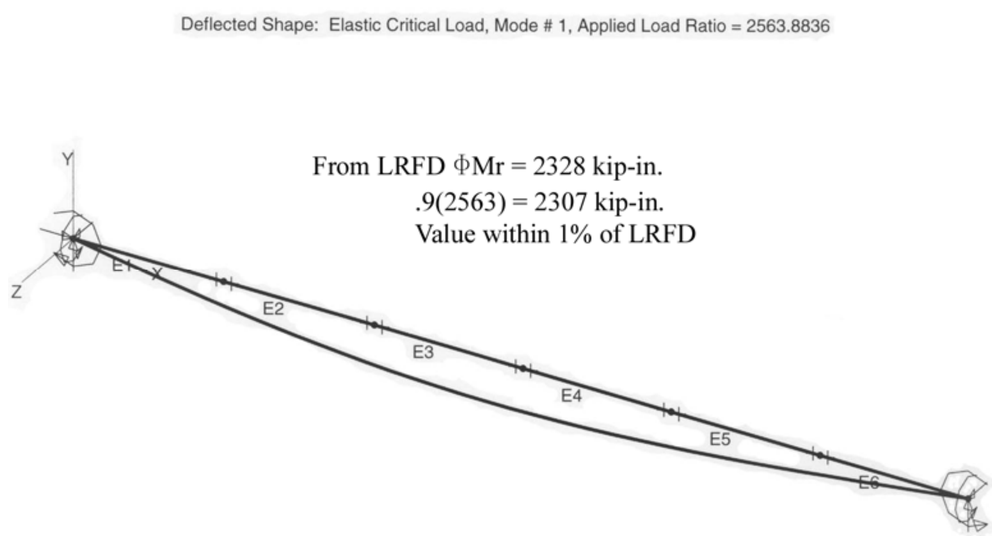


FIGURE 3.12: MASTAN2 beam model buckling load, 2563k-in.

CHAPTER FOUR - CASE STUDY

4.1 Background

The South Snyder River Bridge in Nebraska is a specific case where the steel plate girders experienced excessive displacement during the concrete deck placement. The bridge consists of a single, 150-foot simple span. In this report, several models are run in order to determine whether the diaphragm cross-frames used on this bridge were adequate to brace the girders during deck placement.

The Snyder Bridge was a phased construction project. The original bridge was kept open to traffic while Phase I of the new bridge was constructed. Figure 4.1 shows the construction of Phase I while traffic was maintained on a portion of the original bridge. When Phase I was complete, the original bridge was demolished and the new bridge construction was finished as Phase II.

The complete bridge consisting of both Phases I and II has 5 girders, as shown in Figure 4.2. However, the first stage of construction, Phase I, included only two new girders. The deck overhung the girders by a significant amount, causing many to believe that the deflection the bridge experienced was solely due to the fascia girder torsion attributable to this overhang. The supposition was that the two-girder configuration twisted under the off-center pour load due to this torsion. Stability of the girder system was not assumed to contribute.

However, two-girder systems can be susceptible to stability problems due to lateral torsional buckling during construction. This becomes of particular concern for vertically flexible two-girder systems with closely spaced girders having a low moment of inertia and a long span. Note that this two-girder system problem can arise during construction even if the final bridge has more than two-girders, if construction is phased so that the load carrying system during some part of deck placement is a two-girder system.

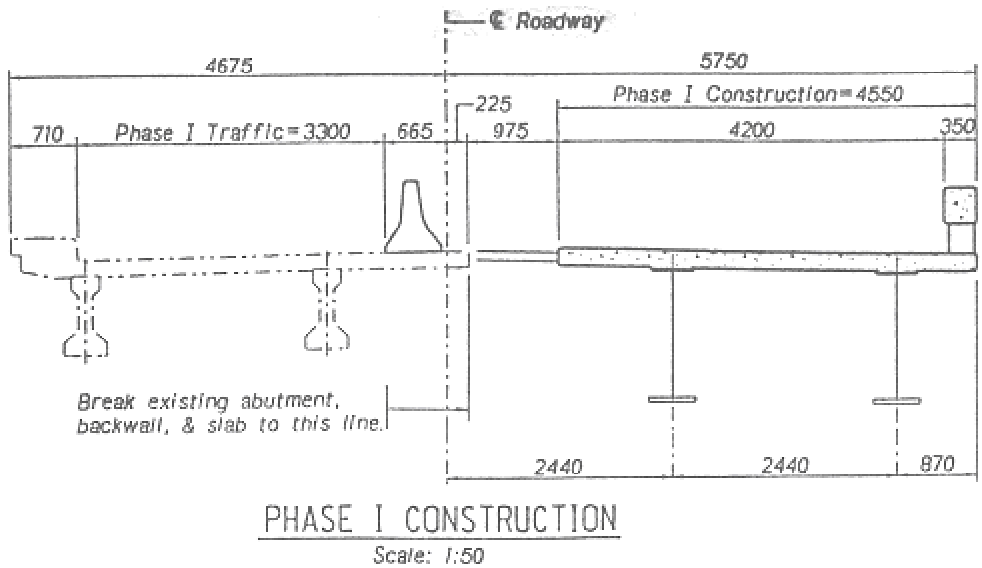


FIGURE 4.1: South Snyder River Bridge, Phase I Construction with a Two-Girder System.

The Snyder Bridge was one of the earliest to use HPS-70W, new high-strength steel with yield strength of 70 ksi. Deflection and stability limit states are of greater concern when one uses HPS. Since all constructional steels have essentially the same elastic modulus, use of HPS would be expected to lead to stability governing in some cases where lower strength steel designs would be governed by strength. If this change in governing limit state is unforeseen, problems will arise. For example, since the amount of restraint provided by torsional braces in the form of cross-frame diaphragms is dependant on the vertical stiffness of the girder, an increase in girder steel strength, reducing girder size and vertical stiffness, can lead to reduced resistance to lateral torsional buckling during construction.

There are thus several reasons to investigate the stability of the two-girder HPS system under the first phase deck placement, especially with regard to the stiffness of the torsional bracing provided by the cross-frame diaphragms.

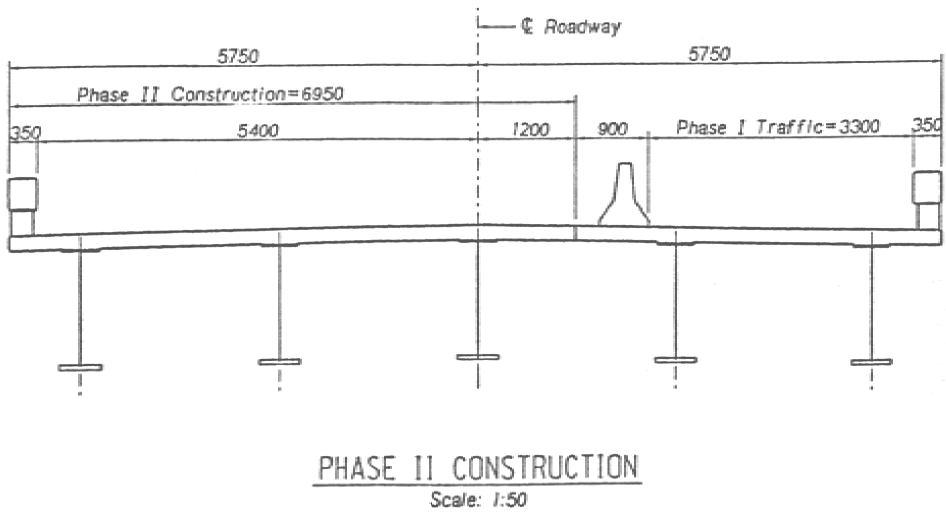
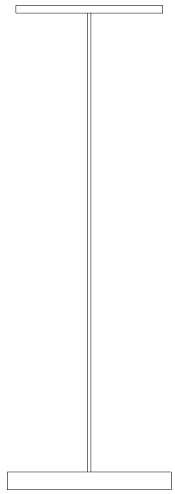


FIGURE 4.2: South Snyder River bridge, completed construction – [KDOT].

4.2 Analytic Approach

The analytic approach used here applies established bracing formulas [Yura] to determine whether the bridge braces were sufficiently stiff. The stiffness of the braces as designed is found compared to the ideal stiffness value for a brace.

The girder is fabricated as a typical I-shaped three plate welded girder, as shown in Figure 4.3. All three plates are HPS-70W high strength steel. The top flange is 17.91 in. wide by 0.945 in. thick. The bottom flange is 20.0 in. wide by 2.165 in. thick. The web is 56.1 in. deep by 0.394 in. thick. All units have been converted from the metric units used on the construction documents to U.S. Customary units. The properties of the Snyder Bridge Plate Girder were calculated to be:



I_y = 1896.3 in^4

I_x = 48975.6 in^4

J = 74.48 in^4

C_w = 1,518,600 in^6

Zx = 1668.8 in3

A = 82.32 in^2

N.A.: 20.76” from exterior of bottom flange

FIGURE 4.3: South Snyder River Bridge girder cross-section, to scale.

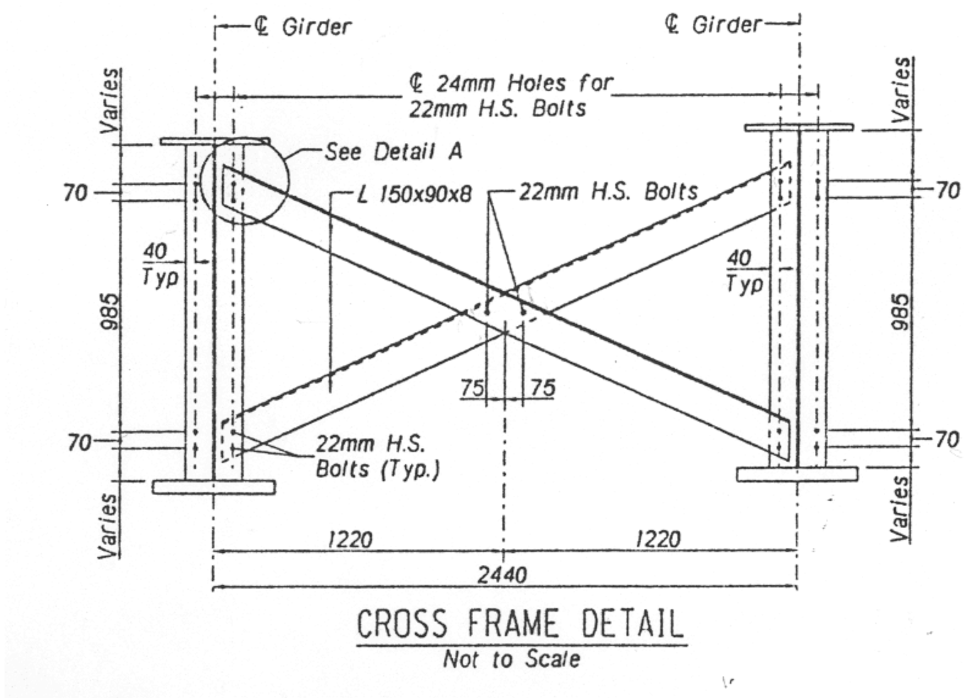


FIGURE 4.4: South Snyder River Bridge cross-frame design.

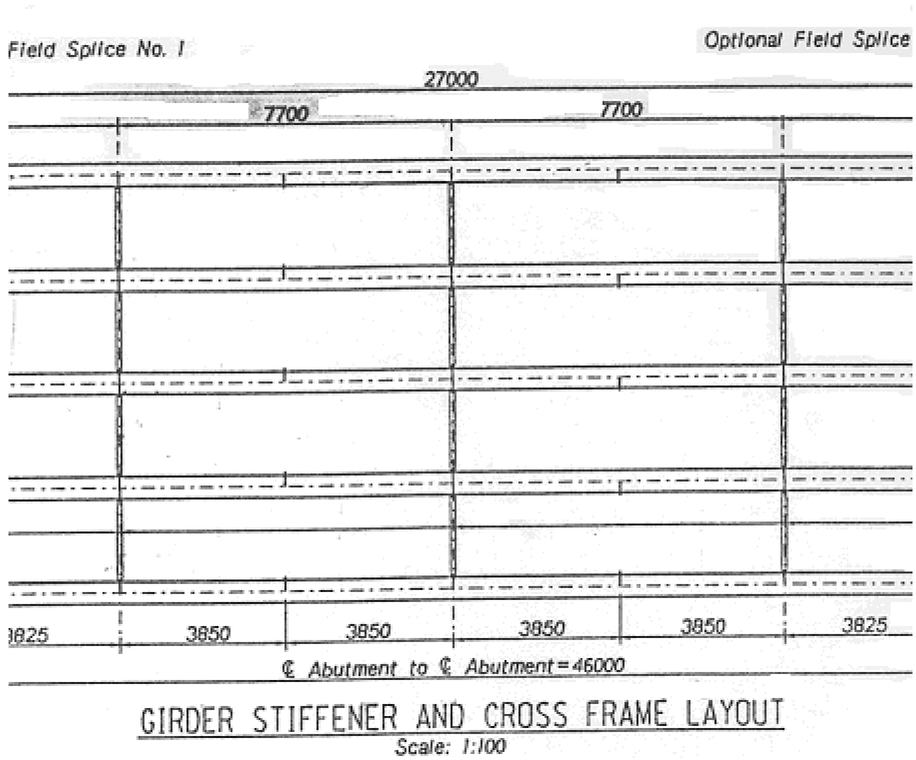


FIGURE 4.5: South Snyder River Bridge cross-frame spacing.

The Snyder River Bridge girders were braced by discrete torsional braces in the form of cross-frames in an X configuration as shown in Figure 4.3. Cross-frames were placed approximately every 7700mm, or 25.3 ft. Figure 4.5 shows the central portion of the bridge plan, including 3 of the 5 cross-frame locations.

The torsional stiffness of a brace point, $\beta_{T,}$ is determined from the brace stiffness, $\beta_{b,}$ the cross-sectional web stiffness, β_{sec}, and the girder system stiffness, $\beta_{g'}$, [Yura]:

$$\frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_{g'}} \quad ,$$

The brace stiffness is found from the formula $\beta_b = \frac{A_c E S^2 h_b^2}{L_c^3}$, where A_c is the cross-sectional area of the angle used in the X cross-brace, E is the modulus of elasticity, S is the girder spacing, h_b is the cross-frame height, and L_c is the diagonal length. The angle given in millimeters is an L150x90x8. This is an L 6x31/2x5/16 in U. S. Customary units. Its area is 2.87in^2. E is 29,000 ksi in all calculations. S is 8 ft. h_b is 41.5”. L_c is 105”. This formula is for a tension-compression system corresponding to the X configuration cross-frames without top or bottom struts. Using this information, β_b is 1,141,171 k-in./rad.

The girder system stiffness, β_g', is given by the formula $\frac{12S^2EI_x}{L^3}$, where Ix is the strong-axis moment of inertia for the girder cross-section, and L is the girder span. Using information in the prior paragraph, β_g' is 27,662 k-in./rad.

The cross-section web stiffness, β_{sec}, is found using the equation $3.3\frac{E}{h}\left[\frac{(N+1.5h)t_w^3}{12} + \frac{t_s b_s^3}{12}\right]$, where t_w is the web thickness 0.394”, h is the distance between flange centroids, 56”, b_s is the width of the stiffener 9.45”, t_s is the thickness of the stiffener 0.354” (8mm), and N is the contact length of the torsional brace 0.354”. This equation is for full-depth stiffening. Using this information, β_{sec} is 43,881 k-in./rad..

Substituting these values into the original equation, the total brace stiffness, β_T, is 16,718 k-in./rad. This is the stiffness of the brace in the field.

The critical moment corresponding to this stiffness can be found using the equation $M_{cr} = \sqrt{C_{bu}^2 M_o^2 + \frac{C_{bb}^2 B_T EI_{eff}}{C_T}}$ < My or Mbp [Yura]. M_y and M_{bp} are the yield moment and the buckling moment between brace points, respectively. The yield moment 92,820 k-in. is the yield stress, 70 ksi, multiplied by the section modulus, 1326 in^3. C_{bu} and C_{bb} are limiting C_b factors corresponding to an unbraced beam and an effectively braced beam, respectively. These can both be conservatively estimated as one. B_T is the equivalent effective continuous torsional brace stiffness found from summing discrete brace stiffnesses and dividing by the member length. M_o is the buckling moment for the beam with no braces. E and I_{eff} are defined in previous paragraphs. The unbraced buckling moment, $M_{o,}$ is found from AASHTO equation 6.10.4.2.5. This equation produces a stress, which is then multiplied by the section modulus to determine a moment. The critical stress found from this equation is 1.340 ksi, giving a moment, M_o, of 1775.9 k-in. The critical moment, M_{cr}, corresponding to β_T computed for the structure in the field is 41,107.7 k-in., or 3425.6 k-ft. The applied dead load moment computed at the time of deck placement is 3488 k-ft. Since the load applied in the field is approximately equal to the girder capacity in the field, the observed behavior of excessive girder lateral movement could be explained as incipient buckling.

Having found that the bracing stiffness required was insufficient, it is of interest to determine the stiffness, β_T^*, that would have been required to allow the cross-braces to provide a brace point. The formula used was $\beta_T^* = 2.4L\ M_f^2/(nEI_{eff}C_b^2)$ [Yura].

The span length, L, is 1811 in., or about 150 ft. There are 5 braces. $I_{eff} = I_{yc} + t/cI_{yt}$ [Yura]. I_{yc} is the moment of inertia for the compression flange (top). I_{yt} is the moment of inertia for the tension flange (bottom). The top flange is about 1" by 18", with a moment of inertia of 486 in^4. The tension flange is 20" by 2.16", with a moment of inertia of 1440in^4. t and c are distances to tension and compression flange centerlines from the neutral axis of the cross section and are 20.14" and 37.44", respectively. M_f is the moment from non-composite dead load, including the slab. This has been calculated as 3488 k-ft. Using this information, I_{eff} is 1260 in.^{4}. C_b is 1.0. The stiffness, B_T^* is 41,424 k-in./rad. This is almost three times the stiffness provided in the field. If one considers that 41,424 is 2.4 times the ideal stiffness, the ideal stiffness is 17,260 k-in. This would have been the stiffness necessary to provide a brace point, however when one uses the ideal stiffness, force in the braces will tend to infinity.

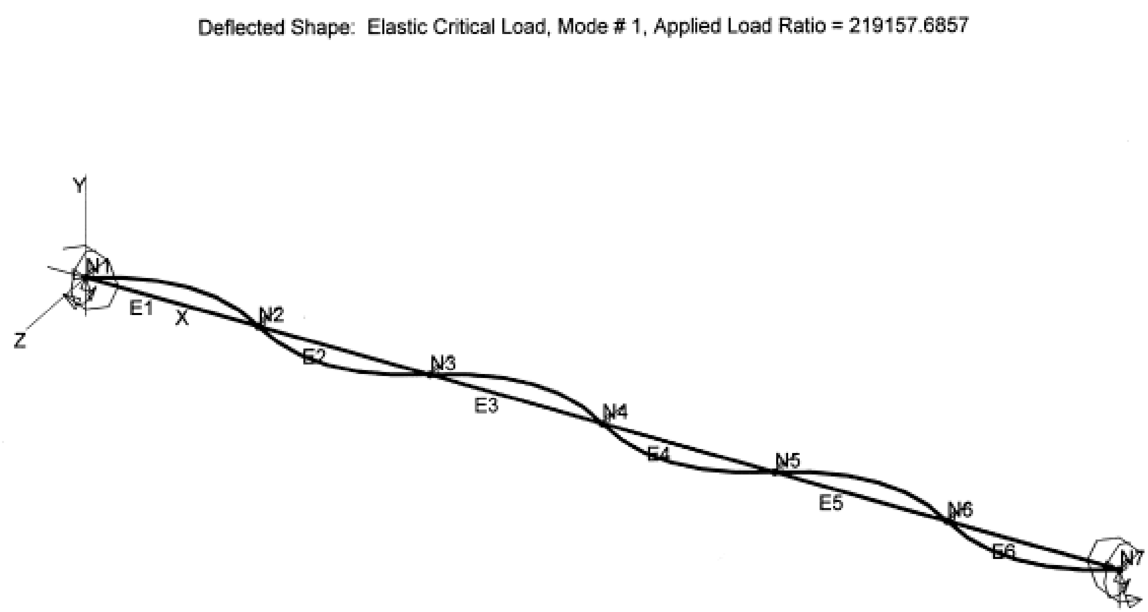
Therefore, a rational basis is established for explaining the excessive deflections observed during deck placement as due to instability caused by insufficient bracing stiffness.

4.3 Analysis Using MASTAN

The analytic approach taken in Section 4.2 gave results indicating that the bracing on the Snyder River Bridge was inadequate with regard to stiffness. The girder is also computationally modeled using the MASTAN2 software to confirm the analytic results and provide insight to a correct brace stiffness design.

The first model is a single girder with the same length and properties as a single girder of the Snyder River Bridge. This girder is rigidly supported laterally at the same points as the Snyder River Bridge brace points. The bending capacity for beams is standardly given for the case of uniform moment. Therefore, concentrated moments of 1 k-ft are applied at the end of the span. The model resulted in a moment capacity of the girder, using an unbraced length equal to cross-frame spacing, of 219,158 k-in. (Fig. 4.6). This establishes an upper limit on girder capacity, assuming the cross-frames act as adequately stiff braces.

FIGURE 4.6: Beam Model with 219158 k-in. capacity.



Second, the girder system stiffness is quantified at each cross-frame location as follows. The girder is modeled as simply supported at the abutments. A single vertical 1 kip load is applied at each brace point in turn. Deflections from the five load locations are tabulated (Table 4.1). Horizontal deflections are calculated as a component of vertical deflection by multiplying by the ratio of girder spacing to brace height. These numbers are used to determine the stiffness of the girder at various points. The stiffness is the inverse of the deflection if the applied force is 1.

TABLE 4.1: South Snyder River Bridge deflections at brace points.

Brace Point	Vertical Deflection, in.	Horizontal Deflection, in.
1	0.0268	0.06195
2	0.06869	0.15878
3	0.08712	0.20139
4	0.06869	0.15878
5	0.0268	0.20139

Third, the cross-frame is modeled. The brace angle is an L6x31/2x5/16. It is pinned at the bottom and allowed to deflect horizontally at the top. A one kip force was applied horizontally to the top and the deflection was 0.001493". The brace stiffness is the inverse of this number, or 669.8 k/in. Fourth, the stiffness for the system was found using the relationship:

$$\frac{1}{\beta_{total}} = \frac{1}{\beta_{brace}} + \frac{1}{\beta_{girder}}$$

These values are tabulated below (Table 4.2). The brace areas were calculated using the expression for axial stiffness, β = EA/L. L for this model was 60", or 5 feet.

TABLE 4.2: South Snyder River Bridge brace stiffness and area calculations.

Brace Point	Deflection, in.	Brace Stiffness k/in.	Girder Stiffness, k/in.	Total Stiffness, k/in.	Brace Area, in.^{2}
1	0.06195	669.8	16.14	15.76	0.03261
2	0.15878	669.8	6.298	6.24	0.01291
3	0.20139	669.8	4.695	4.93	0.0102
4	0.15878	669.8	6.298	6.24	0.01291
5	0.06195	669.8	16.14	15.76	0.03261

Finally, the Snyder River Bridge girder is modeled again with braces with these specific stiffnesses at the brace points in order to determine the buckling capacity. As shown in the upper portion of Table 4.3, when the girder is modeled with five braces with the stiffness of brace #3, the buckling moment is 42,641 k-in., or 3553 k-ft. This value matches well with the analytically determined value of 41,107.7 k-in., or 3425.6 k-ft. As shown in the lower portion of Table 4.3, when the girder is modeled more accurately with five braces with the stiffness of brace #1 – 5 in Table 4.2, the buckling moment is 50,944 k-in., or 4245.3 k-ft. This value is higher than the applied dead load moment of 3488 k-ft and confirms that the behavior expected would be excessive deflection due to softening of the structural system as its buckling capacity is approached rather than actual collapse, again corresponding to behavior observed in the field.

TABLE 4.3: Summary of the calculations

Considering equal stiffness for all braces (using values of brace # 3)		
buckling moment (k-in)	42,641	k-in
buckling moment (k-ft)	3553	k-ft
Considering stiffness of each brace individually		
buckling moment (k-in)	50,944	k-in
buckling moment (k-ft)	4245.3	k-ft

CHAPTER FIVE - DESIGNING EFFICIENT CROSS-FRAMES FOR PLATE GIRDER BRIDGES

5.1 Addressing KDOT Concerns

KDOT has three concerns regarding lateral bracing design. The three concerns have to do with 1) consequences of the AASHTO code elimination of bracing requirements on new bridge design, 2) deck replacement, and 3) skewed bridges. In order to addresses these concerns, Section 5 of this report provides specific direction on how to conduct a "rational analysis" to design and space cross-frame diaphragms on girder bridges applicable to new bridge design, deck replacement, and skewed bridges. Cross-frame spacing, strength and stiffness, and skew are addressed in turn.

5.2 Cross-frame diaphragm spacing

A straightforward approach for determining cross-frame diaphragm spacing is provided in the document "Designers' Guide to Cross-Frame Diaphragms" [Mertz]. These guidelines follow slenderness ratios outlined in AASHTO LRFD 6.10.4.2.6a Compression Flanges. These slenderness ratios will result in slender elements. This is to be expected since:

> "The girder, which is composite in its final condition, is non-composite prior to the hardening of the concrete deck. Trying to meet the compact or non-compact section requirements will result in brace point spacing even less than the traditional value of 25 feet" [Mertz].

This is an iterative process, so one needs to begin by selecting trial brace spacing. Convenient initial trial spacing might be the previous maximum of 25-feet, unless the design specifics make spacing convenient.

Determine the web slenderness ratio, $2D_c/t_w$, for the non-composite section. This ratio is then compared to the ratio $\lambda_b\sqrt{(E/F_{yc})}$. λ_b is 5.76 for members with a compression flange area equal to or greater than the tension flange area and 4.64 for members with a compression flange area less than the tension flange area. C_b is the moment gradient correction factor, and I_{yc} is the moment of inertia of the compression flange about the vertical axis in the plane of the web. If the web slenderness ratio is less than the λ_b ratio, proceed with AASHTO LRFD eqn. 6.10.4.2.6a-1. If the web slenderness ratio is greater than the λ_b ratio, another comparison must be made. If L_b is less than or equal to L_r, then proceed with AASHTO LRFD eqn. 6.10.4.2.6a-2. If L_b is greater than L_r, proceed with AASHTO LRFD eqn. 6.10.4.2.6a-3. These equations will produce the moment capacity of the non-composite section. This process is shown schematically in Figure 4.7. For the negative moment region (continuous girders near supports), AASHTO LRFD eqn. 6.10.4.1-1, where if L_b is less than L_p , applies.

If the section's moment capacity is less than the resistance required, the brace spacing must be decreased. If the section's moment capacity is much greater than the resistance required, the brace spacing may be increased. If the moment capacity is slightly greater than the moment capacity, the spacing is satisfactory [Mertz].

Cross frame spacing

Compare $2D_c/t_w$ ratio to $\lambda_b\sqrt{\frac{E}{F_{yc}}}$

If $2D_c/t_w \leq \lambda_b\sqrt{\frac{E}{F_{yc}}}$ LRFD Eqn. 6.10.4.2.6a-1

If $2D_c/t_w > \lambda_b\sqrt{\frac{E}{F_{yc}}}$ $\begin{cases} L_b \leq L_R & \text{LRFD Eqn. 6.10.4.2.6a-2} \\ L_b \rangle L_R & \text{LRFD Eqn. 6.10.4.2.6a-3} \end{cases}$

FIGURE 4.7: Cross-frame spacing flowchart.

5.3 Cross-frame diaphragm strength and stiffness

In order to perform a rational analysis that determines brace strength and stiffness, the principles of structural stability are followed [Galambos]. Specific guidance is available in the literature [Yura], as reviewed and used in Sections 2, 3, and 4. Application of these equations [Yura] provide a viable method for determining brace strength and stiffness for both design of new bridges and evaluation of existing bridges for deck replacement projects.

As a strength requirement, the brace must be able to carry 0.4% of the load in the compression flange of the member to be braced. For discrete torsional bracing, required brace strength is: $M_{br} = F_{br}h_b = 0.005\left(\frac{L_b}{h}\right)\frac{LM_f^2}{nEI_{eff}C_{bb}^2}$ [Yura]. This strength requirement is based on the presence of adequate bracing stiffness. Thus, according to Yura's equations if the brace stiffness is inadequate, the brace force tends to infinity.

As a stiffness requirement, the brace must provide 2 times the ideal stiffness. For discrete torsional bracing, required brace stiffness is: $B_T^* = \overline{B_T^*}\frac{L}{n} = \frac{2.4LM_f^2}{nEI_{eff}C_{bb}^2}$.

In these two formulas, M_f is the maximum beam moment, C_{bb} is the moment diagram modification factor for the fully effective bracing condition, L is the span length, L_b is the unbraced length, and n is the number of intermediate braces. $I_{eff} = I_{yc} + \left(\frac{t}{c}\right)I_{yt}$, where I_{yc} and I_{yt} are the compression and tension flange moments of inertia, respectively.

5.4 Cross-frame diaphragms on skewed bridges

A further concern of KDOT is how to treat cross-frame diaphragms on skewed bridges. The effect of skew on brace behavior is discussed in Section 2.4. For braces in even highly skewed bridges where the braces are erected perpendicular to the girders, the equations cited in Section 5.2 [Yura] can be applied without modification. However, for braces placed on the skew, the equations [Yura] need to be adjusted as follows [Wang& Helwig]. For bridges with braces placed on the skew, the brace stiffness is reduced to β cos^2ϕ.where ϕ is the angle of skew. The moment developed in the brace is magnified to Mbr/cosϕ.

This does not address KDOT's concerns, discussed in Section 2.4, about fatigue for braces placed perpendicular to the skew. However, it is possible to clarify when braces must be temporary or permanent as well as to present details available that will minimize fatigue-cracking risk for braces perpendicular to girders.

“Designers’ Guide to Cross-Frame Diaphragms” [Mertz] enumerates the cases where permanent and temporary cross-frame diaphragms are called for [He notes AASHTO LRFD (art. 6.7.4)]:

- *“Permanent* cross-frames are not required in simple span bridges or in positive moment regions of continuous bridges.
- *Temporary* bracing is required on compression flanges of simple span steel girders bridges and comp. Flange in positive moment regions of continuous bridges.
- *Permanent* bracing is not required in neg. moment regions of continuous steel girder bridges
- *Permanent* bracing is required in compression flanges in neg. moment regions of continuous bridges”

Mertz notes that if one uses only temporary braces, any future designers repairing the bridge must be aware of the need for bracing during any reconstruction of the deck. Mertz suggests the following options. An option that has been researched is to only have cross-frames in place during erection and then have them removed from the permanent structure. This would also improve any situations resulting from stress fractures in girders at the point of braces [Zhao]. This would require extra erection work, (new techniques that could be expensive initially due to new fabrication and erection techniques) but could prove beneficial in the long run.

Traditional cross frames are less expensive for fabrication [Mertz], with the simple X configuration being simplest to fabricate. An X configuration with a bottom strut is both stiffer and costlier. Top and bottom struts are necessary as the girder gets deeper. Top and bottom struts have the added advantage of resisting torsion applied to the fascia girders by deck form overhang brackets [Roddis et al.]. Tab plates are often welded or bolted to the web of the girder to accommodate these braces. However, fatigue cracking has been an issue with these connections, and this is an advantage of temporary bracing. Mertz argues that if the designer follows AASHTO LRFD article 6.6.1.3.1, which requires designing these plates to resist 20 kips, the fatigue cracking problem should not exist, however since this fatigue is distortion, not stress, driven, a such a strength design criteria is unlikely to solve the difficultly. A rational way to minimize fatigue is to make use of well-detailed connections such as those recommended by the Steel Committee for Economical Fabrication (SBC) and illustrated in Figures (Figures 4.8-4.9)

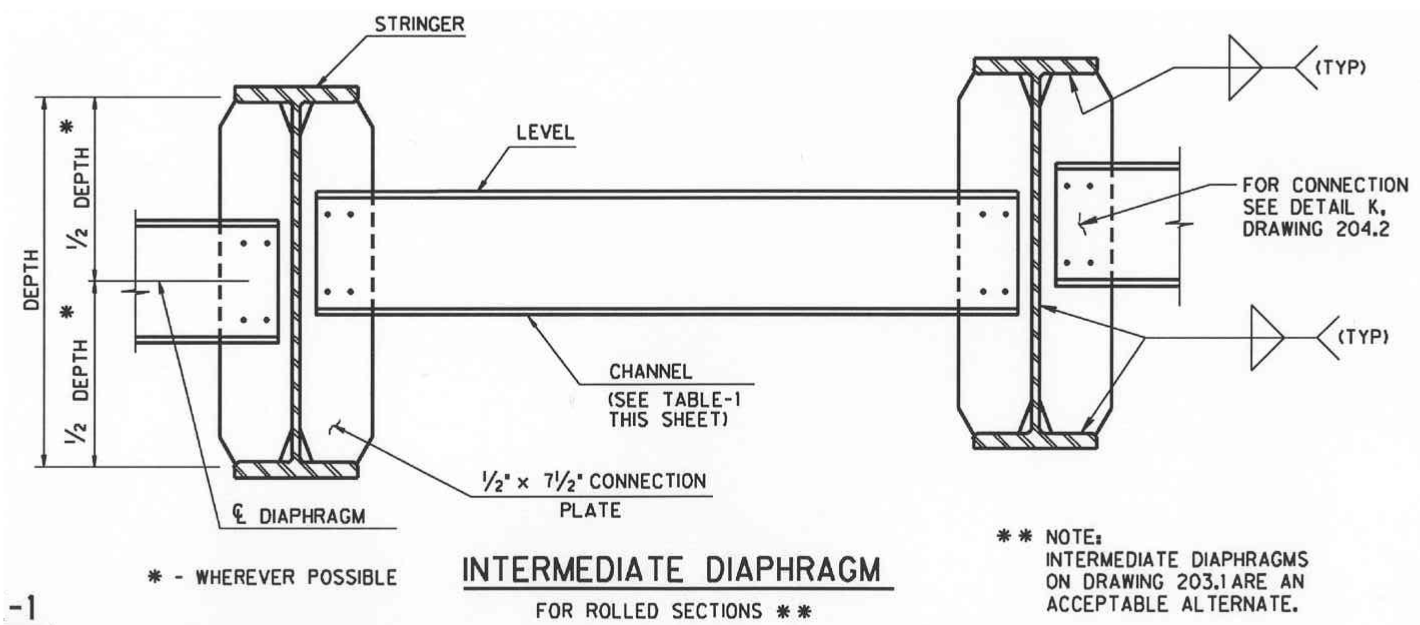


FIGURE 4.8: SCEF diaphragm detail.

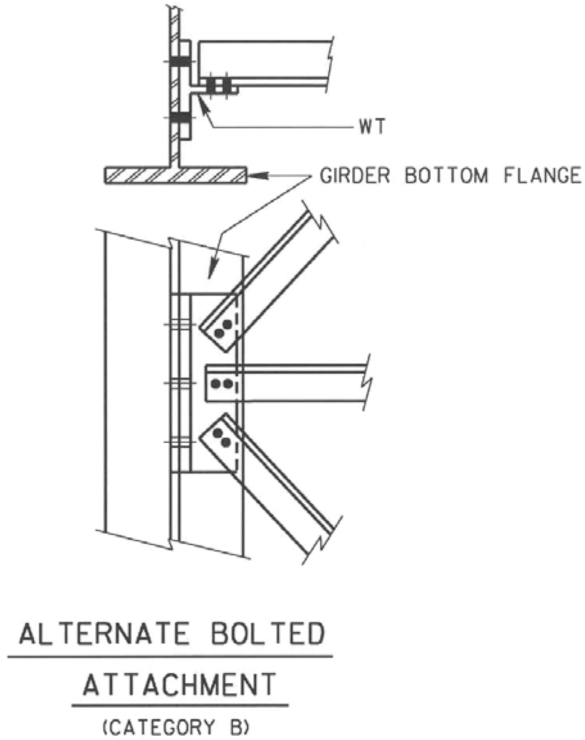


FIGURE 4.9: SCEF in-plan attachment detail.

CHAPTER SIX - IMPLEMENTATION PLAN

As previously stated in Chapter 5, KDOT has three concerns regarding lateral bracing design. The three concerns have to do with 1) consequences of the AASHTO code elimination of bracing requirements on new bridge design, 2) deck replacement, and 3) skewed bridges. This Implementation Plan organizes the results of the previous discussions to address KDOT concerns regarding lateral bracing design.

Given the possible contributing factors of elimination of in-plan bracing, decreased vertical stiffness reducing the restraint of torsional braces, and the occurrence of two-girder systems, it is understandable that some combination could lead to a situation where the stiffness of the bracing system is not adequate to provide stability of the compression flange during concrete deck placement. It would then be expected that the compression flanges of the girders would buckle under deck placement loads.

Chapter 5 of this report addresses KDOT's concerns by providing specific direction on how to conduct a "rational analysis" to design and space cross-frame diaphragms on girder bridges applicable to new bridge design, deck replacement, and skewed bridges. Cross-frame spacing [Mertz], strength and stiffness [Yura], and skew [Wang & Helwig] are addressed in turn. Direction is thus provided for conducting a "rational analysis" of bracing strength and stiffness. Appropriate standard details, developed by the Steel Committee for Economic Fabrication, are depicted.

The implementation plan recommended is that the specific direction provided in Chapter 5 be used by KDOT for design of cross-frame diaphragms for new bridges and evaluation of existing bridges for deck replacement projects. This information could be used by the KDOT Bridge Design Department immediately.

REFERENCES

[AASHTO LRFD] AASHTO LRFD Bridge Design Specifications, Second Edition, 1998, Washington, D.C.

[AASHTO] AASHTO Bridge Design Specifications, 1985, Washington, D.C.

[AISC] AISC Steel Design Manual (1993) Load and Resistance Factor Design Specification for Structural Steel Buildings, Chicago, IL.

[Azizinamini] Azizinamini, Atorod, Yakei, Aaron J., and Mans, Patrick. (2002). "Flexural Capacity and Ductility of HPS-70W Bridge Girders". *Engineering Journal*, First Quarter, 2002, 38-51.

[Duntemann] Duntemann, John F., P.E., S.E. (Wiss, Janney, Elstner Associates, Inc, Northbrook, USA), and Subrizi, Cris D., P.E. (2000). "Lessons Learned from Bridge Construction Failures." *Forensic Engineering, Proceedings of the Congress.* 2nd Forensic Engineering Congress, May 21-23, San Juan, Puerto Rico. ASCE. 374-385.

[Earls] Earls, C.J., and Shah, B.J. (2001). "High performance steel bridge girder compactness." *Journal of Constructional Steel Research*, 58 (5-8), 859-880.

[Galambos] Galambos, Theodore V. (1998). Guide to Stability Design Criteria for Metal Structures, Fifth Edition. John Wiley & Sons, Inc. New York, NY. Chapter 5, 192-217.

[Jones] Phone interviews with John Jones, Kansas Department Of Transportation, April 17th & 24th, 2003.

[MDX] http://www.mdxsoftware.com Design software that allows for skewed bridge placement

[McGuire] McGuire, William, Gallargher, Richard H., and Ziemian Ronald D. (2000). Matrix Structural Analysis, 2nd Edition, John Wiley & Sons Press, New York, NY. Attached software is MASTAN.

[Mertz] Mertz, Dennis R. (2001). "Designers' Guide to Cross-Frame Diaphragms". Prepared for the American Iron and Steel Institute. http://www.steel.org/infrastructure/pdfs/crossframe.pdf

[Nethercot] Nethercot, D.A. (1989) "Design of bracing systems for plate girder bridges". *Journal of Constructional Steel Research*, 6 (1) 23-34.

[SBC] E-mail survey of AASHTO/NSBA Steel Bridge Collaboration through e-mail distribution list. April 10, 2003. http://www.steelbridge.org/listserv.htm

[Roddis et al.] Roddis, W. M. Kim, Paul Kulseth, and Zhong Liu, "Torisional Analysis for Exterior Girders Version 2.0", *Report No. K-TRAN: KU-00-03*, Final Report, Kansas Department of Transportation, Topeka, KS, August 2002, 136 pages.

[Puckett] Puckett, Jay A., and Barker, Richard M. (1997). Design of Highway Bridges. John Wiley & Sons, Inc. New York, NY. 736-737.

[Wang] Wang, Y.C., and Nethercot, D.A. (1990). "Bracing Requirements for Laterally Unrestrained Beams." *Journal of Constructional Steel Research*, 17 (4), 305-315.

[Wang & Helwig] Wang, Liqun, and Helwig, Todd A. (2003). "Bracing of Steel Bridge Girders with Skewed Supports." *Proceedings, 2003 Annual Technical Session & Meeting, Structural Stability Research Council, April 2-5, 2003, Baltimore, MD*, 519-538.

[Yura] Yura, Joseph A., (2001) "Fundamentals of Beam Bracing". Structural Stability Research Council Conference, *Is Your Structure Suitably Braced?,* February, 2001.
Yura, Joseph A., (2001) "Fundamentals of Beam Bracing". *Engineering Journal*, First Quarter, 2001, 11-26.

[Zhao] Zhao, Yuan. (2003). "Fatigue-Prone Steel Bridge Details: Investigation and Recommended Repairs" PhD. Dissertation, University of Kansas.

K - TRAN

KANSAS TRANSPORTATION RESEARCH
AND
NEW - DEVELOPMENTS PROGRAM

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:

KANSAS DEPARTMENT OF TRANSPORTATION

THE UNIVERSITY OF KANSAS

KANSAS STATE UNIVERSITY