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Stability Bracing Requirements for Lean-on Systems

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Abstract

Lateral torsional buckling often controls the design of steel bridges during construction. The buckling resistance is affected by a number of factors, including the geometry of the girders and the spacing between braces. Steel bridges are commonly braced by cross-frames, which commonly consist of single-angle members. In bridges with complex geometry such as significant support skew, the installation and long-term maintenance of cross-frames are complicated due to the skewed geometry. Lean-on bracing concepts, which selectively remove cross-frames in place of utilizing only top and bottom struts are becoming an attractive design option. Lean-on bracing significantly improves the ease of installation during erection and can also reduce the fatigue demands on the braces. However, there are several questions regarding the stability bracing behavior of these systems. Recent research has shown that system behavior is greatly influenced by the implementation of lean-on bracing, but has not specifically addressed the impact of different lean-on bracing configurations on the stiffness and strength of these systems. This paper is focused on a methodology for quantifying the impact of configuration on lean-on bracing systems through the use of linear eigenvalue buckling and nonlinear imperfection analyses.

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1. Introduction

There are a number of stages in the life of a bridge that need to be considered by designers. The critical stage for lateral-torsional buckling generally occurs during early stages of erection when not all bracing is present or during construction of the concrete deck when the steel girder alone supports the entire construction load. Stability not generally a problem in the completed bridge since the girders are laterally and torsionally restrained by the concrete bridge deck. The elastic lateral torsional buckling capacity, M_{cr}, of a doubly symmetric girder is given by the following expression derived by Timoshenko (Timoshenko and Gere, 1961):

$$M_{cr} = C_b \frac{\pi}{L_b} \sqrt{E I_y G J + \left(\frac{E\pi}{L_b}\right)^2 I_y C_w} \tag{1}$$

Where E is the elastic modulus; G is the shear modulus; J is the torsional constant; I_y is the moment of inertia about the weak axis; C_w is the torsional warping constant, and L_b is the unbraced length defined by the spacing between braced points.

Effective stability bracing of beams can be achieved by either preventing lateral movement of the compression flange (lateral bracing) or controlling twist of the girders (torsional bracing). The most common form of bracing is steel bridges are cross-frames that fit into the category of torsional braces since the braces restrain twist of the girders. Common cross-frame geometries are shown in Fig. 1a and 1b, which consist of X-frames and K-frames.

The conventional layout of cross-frame systems in steel bridges generally have braces located between each adjacent girder as depicted in Fig. 2a. Historically, cross-frames were restricted to a maximum spacing of 25 ft. (7.62 m.); however, in 1994, AASHTO removed the spacing limit in-place of a requirement for the spacing dictated by a rational analysis. Considering the demand during construction, a common spacing between cross frame lines in steel bridges are 25 ft. (7.62 m.) to 40 ft (12.19 m.).

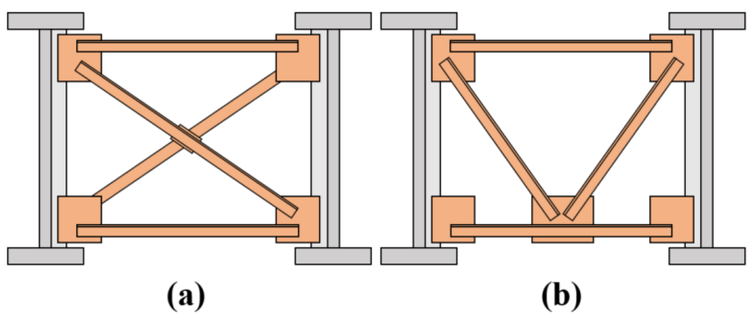

Figure 1: Examples of lateral torsional braces: (a) X-frame and (b) K-frame.

Cross-frames and diaphragms often represent the most expensive component on the bridge per unit weight due to the high fabrication costs. In addition, installation of cross-frames during erection can be complicated by a number of factors. The effects of support skew can often result in difficulty installing cross-frames. In addition, skewed girder applications also often result in larger live-load induced forces compared to bridges with normal supports. The use of lean-on bracing concepts provide an effective means of reducing fabrication costs, improving cross-frame installation during erection, and also minimizing long-term fatigue damage. Lean-on concepts consist of replacing full cross-frames with top and bottom struts as depicted in Fig. 2b. The cross-frame can be selectively positioned along the width of the bridge to minimize the live-load induced force and to best-facilitate installation.

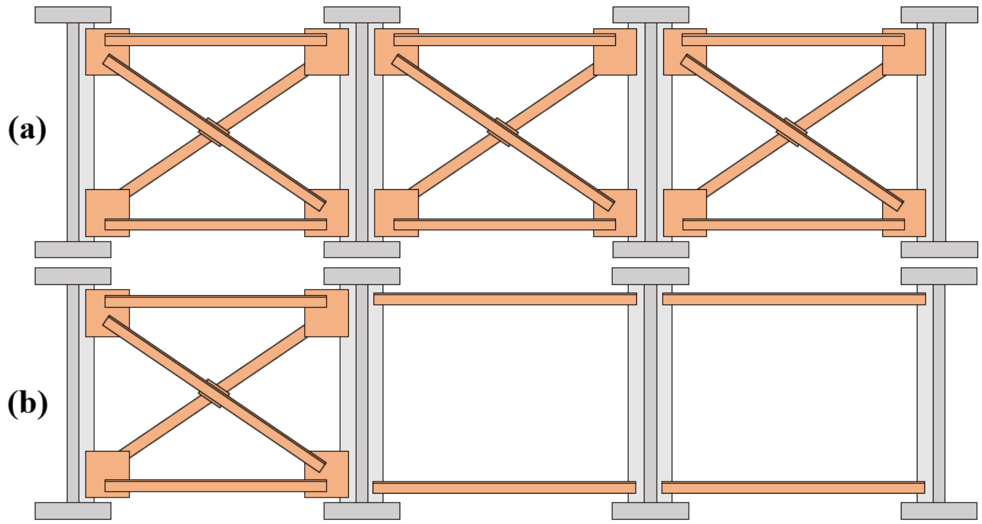

Figure 2: Example of typical bracing system with full cross-frames (a) and using lean-on bracing (b)

The concepts of lean-on bracing in steel bridges were first developed in TxDOT Study 0-1772 (Helwig and Wang 2003) as a means to reduce live-load induced forces. The recommendations in the 0-1772 study have been implemented in multiple bridges including systems with both skewed and normal supports. However, through the process of design, additional questions have arisen on the design requirements for lean-on systems. These questions include optimum layouts for cross-frames both within a given bracing line and the distribution along the length of the bridge. The work in project 0-1772 focused on bridges with skewed supports and therefore focused on the distribution of braces to minimize live-load induced forces. Recently, bridges with normal supports have been designed and the optimal layout of the bracing is not clear for these cases.

This research paper is focused on a study related to optimal layouts for cross-frame systems in straight and skewed bridge systems. The research study has included the instrumentation of two constructed bridges that have made use of lean-on concepts. One of the bridges had normal supports, while the other had skewed supports. The bridges were instrumented and monitored with a variety of loading conditions using four loaded dump trucks. Additionally, the researchers had data obtained from monitoring a skewed bridge with lean-on bracing during construction. The data form these bridges were used to validate a finite element model of the bridges with lean-on bracing. The model is currently being used to conduct a parametric analysis of the bridge to focus on a number of areas on the behavior of lean-on bracing systems. One of the aspects in the design is optimizing the bracing layout on the bridges, which is the focus of this paper.

Following this introductory section, background information on stability bracing and lean-on bracing is provided. This information includes the necessary stiffness and strength requirements for effective bracing. The next section of the paper outlines the finite element model including information on the boundary conditions, loading, and girder sizes. The final two sections provide an outline of the parametric studies that are being conducted using linear eigenvalue buckling and nonlinear geometric analyses on imperfect systems. Finally, the results of the paper are summarized along with future work to be conducted.

2. Background

There have been a number of investigations related to bracing over the years. Two of the most significant efforts were conducted by Winter (1960), and Yura (2001). Winter's work demonstrated the dual criteria that stability bracing systems must satisfy, consisting of both stiffness and strength requirements. Winter's work also demonstrated the impact of initial imperfections on the brace strength requirements. Yura (1992) extended Winter's fundamental work and developed comprehensive formulations for column and beam systems. The behavior of beam bracing systems is summarized in Yura (2001).

As noted above, effective stability bracing requires the sufficient strength and stiffness as outlined in the American Institute of Steel Construction (AISC - 2016) specifications. Recent work documented in Reichenbach et al. (2021) resulted in ballots related to stability bracing requirements that have been approved for inclusion into the AASHTO Bridge Design Specifications (BDS). Many stability bracing systems follow the behavior of springs in series as outlined in Yura et al. (1992), and presented in Eq. 1. The torsional brace stiffness of the system, β_T, of the system is a function of three components: the in-plane girder stiffness, β_g, the brace stiffness, β_b, and the cross-section stiffness, β_{sec}.

$$\frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g} \quad (2)$$

The system torsional brace stiffness will be smaller than the smallest of the three components. AISC (2017) provides the following expression for the required system brace stiffness, $\beta_{T,req}$:

$$\beta_{T,req} = \frac{2.4LM_u^2}{\varphi nEI_{eff}C_b^2} \quad (3)$$

Where L is the span length; M_u is the ultimate design moment; φ is the LRFD resistance factor equal to 0.75; E is the Young's Modulus; n is the number of intermediate braces; C_b is the moment gradient factor; I_{eff} is the effective moment of inertia about the weak axis given by $I_{eff} = I_{yc} + t/c \cdot I_{yt}$. I_{yt} is the lateral moment of inertia of the tension flange, t is the distance from the centroid of the tension flange to the neutral bending axis, and c is the distance from the centroid of the compression flange to the neutral bending axis.

The stiffness given by Eq. 2 is essentially twice the ideal stiffness. Providing twice the ideal stiffness is assumed to result in a twist at the brace location that is approximately equal to the initial imperfection, θ_o, when the girder is subjected to the maximum design moment, M_u. Based upon this assumption, the stability brace moment is given by the following expression:

$$M_{br} = \beta_{T,req}\theta_o = \frac{2.4LM_u^2}{\varphi nEI_{eff}C_b^2}\frac{L_b}{500h_o} \quad (4)$$

Where h_o is the distance between flange centroids and the other terms are as defined previously.

The initial imperfection, θ_o, in Eq. 4 comes from work documented in Wang and Helwig (2005) that demonstrated the critical shape imperfection involves a lateral translation of the compression flange at the brace location (assumed value $L_b/500$) while the bottom flange remains straight. In addition to the critical shape imperfection, Wang and Helwig (2005) also demonstrated the critical location of the imperfections. The maximum twist should occur in the cross frame located near the point of maximum moment. Fig. 3 shows a typical imperfection that was used in the studies. The imperfection was modified slightly to provide a slight asymmetry to the shape, similar to work recommended by Prado and White (2015) and also used by Liu and Helwig (2019) in a study related to torsional brace strength requirements.

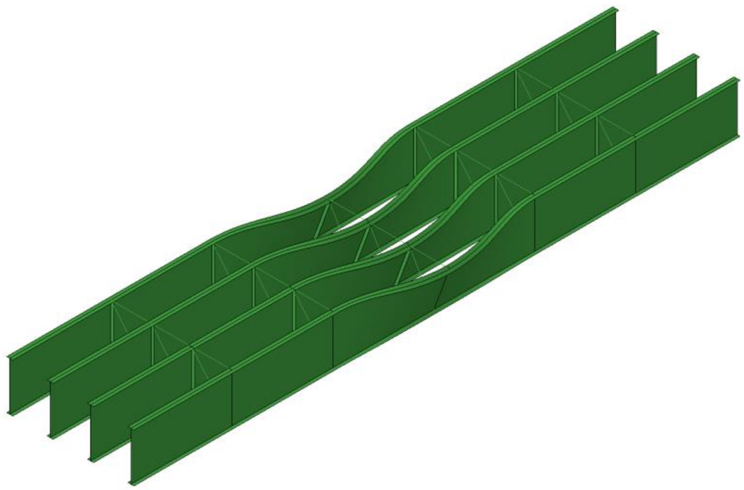

Figure 3: Exaggerated shape of critical imperfection located in maximum moment region

While the goal of effective stability bracing system is for the girder to buckle between the brace points, narrow girder systems are susceptible to a system buckling mode discussed in Yura et al. (2008) and incorporated into the AASHTO BDS in 2015. The system mode of buckling is not sensitive to the spacing between cross-frames since the girders tend to buckle in a mode consisting of a half-sine curve. Sanchez and White (2012) demonstrated that narrow girder systems are often susceptible to significant second order amplification, which led to limiting the maximum moment during construction to 50% of the elastic critical buckling load. Additional work was conducted by Han and Helwig (2019) related to the performance of system buckling on girders with an initial imperfection that resulted in raising the AASHTO limit from 50% to 70% of the critical system buckling capacity. The imperfections utilized in Han and Helwig provided guidance on the current study.

3. Finite Element Model and Analysis Types

The general-purpose finite-element analysis program ABAQUS (2022) was used in the finite element studies conducted on representative bridge geometries. The focus of the analysis was to categorize the effectiveness of different cross-frame configurations with respect to the strength and stiffness requirements discussed in the previous section.

The girders were modeled using S4 linear shell elements. The flanges of the girders were modeled with one element on either side of the web. Efforts were made for an element aspect ratio as close to unity as possible. Braces were modeled using T3D2 linear truss elements. The corners of the cross-frame braces shared the node at the web-flange junctures so there was no cross-sectional distortion from the web. Although cross-sectional distortion was not a concern, web stiffeners were included for completeness. The stiffeners were modeled using S4 linear shell elements located along the web of each brace location.

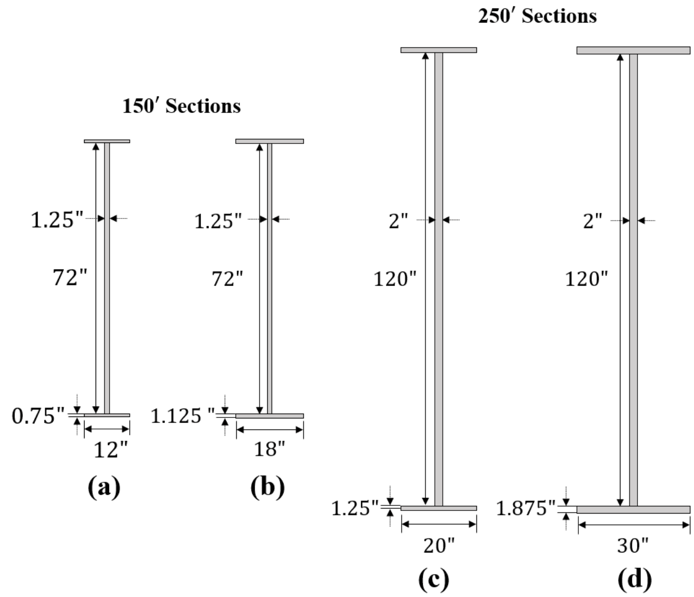

Figure 4: Illustrations of sections used for parametric study that includes a 150 ft (45.72 m) section (a/b) and 250 ft (76.20 m) sections (c/d). The left of each pair is based upon a width to depth ratio of 1/4 whereas the right of each pair is based upon a ratio of 1/6.

The typical girder sections used for the parametric studies are displayed in Fig. 4. These sections were proportioned in respect to typical design ratios. The girder span to depth ratio was maintained at a value of 25, which is consistent with values often targeted in design for girders with simple supports. The web thickness was chosen to be at a depth over thickness ratio of 60 so as to avoid issues with web shear on the LTB behavior as outlined in Liang et al. (2022). AASHTO (2017) limits the flange width to a minimum value of the girder depth over 6 (D/6), however values of D/4 are often targeted in design to avoid excessively slender compression flanges. Therefore, the flanges widths were based upon a width to depth ratio of 1/4 or 1/6. The thicknesses of the flanges were based upon a total flange width to thickness ratio of 16, producing a compact flange to control local buckling.

A uniform distributed load was applied to the top flanges of each bridge girder. The boundary conditions for the model were selected based upon a simply supported girder that was free to warp at the supports. A pinned support that restrained vertical, lateral, and longitudinal movement was positioned at the bottom flange-web juncture on one end of the girder. At the other end a roller support preventing vertical and lateral movement was utilized at the juncture between the bottom flange and the web. Lateral movement of the web nodes were restricted at the support regions. With lateral movement restrained along the web, girder twist was therefore restrained at each support.

For longer span girders, buckling was sometimes governed by the system mode of buckling. However, because the goal of the study was to investigate conventional and lean-on bracing, the decision was made to try to improve the system mode of buckling relative to conventional LTB, which is generally governed by unbraced segments near midspan. The system mode of buckling will often be enhanced by providing a few lateral truss panels near the support regions. To simulate the lateral truss bracing, warping restraints were provided at the support regions of the girders. The warping restraints were achieved by defining a multi-point constraint. For the cases with multi-point constraints, the center node of each flange was identified as the master node (control point) and the other flange nodes at that location were treated as slave nodes (secondary nodes). This restriction causes the degree-of-freedoms of each secondary node to match the control point. Adding warping restraint increases the system buckling capacity of the model while preserving the LTB capacity. Furthermore, as system buckling modes do not provide adequate insight into the brace behavior, these modes need to be minimized for the parametric studies.

Validation of these modelling methods were conducted by applying live load testing data of three bridges that made use of lean-on bracing. Two of the bridges were instrumented to gather validation data using loaded dump trucks. One of the instrumented bridges had normal supports while the other had skewed supports. Additionally, data from an implementation study from TxDOT study 0-1772 (2003) was also utilized that included data during construction and subsequent truck loading on the finished bridge. The validation data provided cross-frame forces and girder displacements at measured locations along the length of each bridge. The modelling methodologies were applied to construct these bridges in ABAQUS and adjustments were made to the modelling methodologies to minimize discrepancies between the experimental and model results.

The analyses that are being conducted in the parametric study include linear eigenvalue buckling analyses and also nonlinear geometrical analyses on systems with an initial imperfection. The linear eigenvalue buckling is utilized to identify the ideal brace stiffness required for the bridges. The focus of the study is on the behavior of lean-on bracing systems relative to conventional bracing. The research is considering both the stiffness performance and the strength behavior of the bracing.

After obtaining the stiffness requirements from the eigenvalue buckling analyses, nonlinear imperfections analysis, utilizing the Riks Arclength method, is being used to simulate a critical imperfection on a system and mimic the desired mode of buckling. The imperfection is modelled by selecting nodes within a bounding box and translating those nodes based upon the imperfection equation shown in Eq. 5.

$$\frac{sin\left([x-(x_{crit}+L_b)]\frac{\pi}{L}-\frac{\pi}{2}\right)}{2}\cdot\frac{L_b}{500} \text{ where } x_{crit}-L_b \leq x \leq x_{crit}+L_b \tag{5}$$

Where x_{crit} is the location of the critical cross-frame line and $L_b/500$ is the imperfection magnitude. Additionally, the imperfection needs to be adjusted to be slightly asymmetric to better allow for the system to develop the buckling shape. This can be done by augmenting the equation illustrated in Fig. 5 by a skew modification. The skew modification is a normalized weight that decreases the magnitude of half of the critical shape imperfection causing an asymmetry. The equation for the skew modification’s second half is the same as the critical shape imperfection equation.

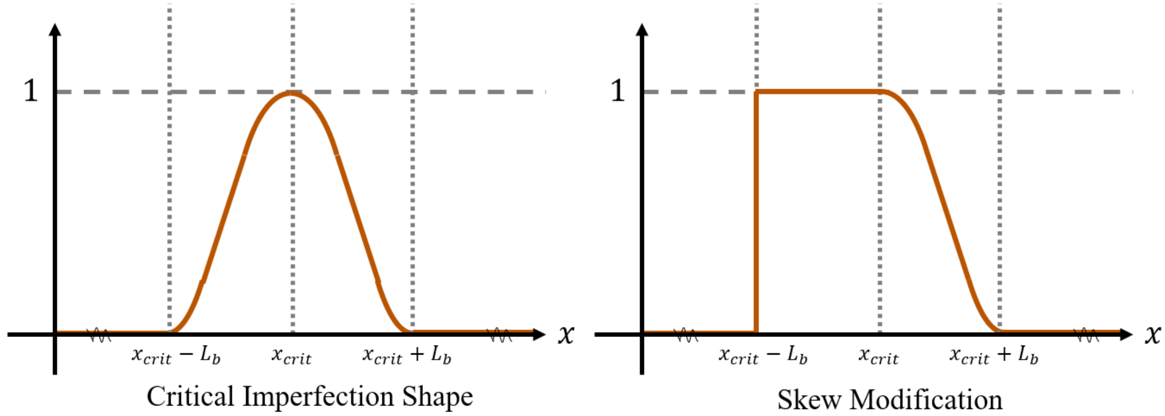

Figure 5: Illustration of critical imperfection shape produced by Eq. 5. and skew modification.

A set imperfection does not guarantee a failure mode such as buckling between brace points or system buckling. Imperfections must be synchronized with the controlling failure mode via the eigenvalue buckling analysis.

4. Linear Elastic Buckling Parametric Study

Conducting linear eigenvalue analysis serves three purposes: to provide a comparison of the stiffness of each cross-frame configuration, to determine the ideal stiffness for the nonlinear imperfection analysis, and to confirm the shape of the failure mode. For the purposes of this study, the ideal stiffness is measured as the stiffness required to buckle between the brace points and reach at least 90% of the calculated M_{cr} according to Eq. 1.

In order to characterize the behavior of lean-on versus conventional bracing systems a matrix of parameters that represent a reasonable range of typical bridge geometries were established, as provided in Table 1. The initial studies are focused on simply supported systems, which will be followed by continuous girders. The values in Table 1 are for the simply supported girders. Some minor adjustments have been made to some of the parameters to provide a reasonable geometry. For example, the spans considered range from 150 ft. (45.72 m.) to 250 ft (76.20 m.). However, to provide a uniform spacing between cross frame lines, spans of 160 ft. (48.77 m.) and 240 ft. (73.15 m.) were used for the cases with 40 ft. spacings between bracing lines. Additionally, the cross-frames configurations that are being considered are illustrated in Fig. 6 and are based upon preliminary investigations. Note that the number of configurations may change based upon the number of available cross-frame lines and bays, but each follow the same fundamental distribution. Overall, these parameters and configurations produce a total of 672 bridges that are being analyzed. Expansions and adjustments to the parametric ranges may occur based upon study results from the studies.

Table 1. Initial parametric study variables and ranges for bridges.

Parametric Variable	Range Imperial Units	Range SI Units
Girder Spacing	{10 ft, 12 ft}	{3.05 m, 3.66 m}
# of Girders	{4, 5}	{4, 5}
# of Spans	{1, 2}	{1, 2}
Span Length	{150 (160) ft, 250 (240) ft}	{45.72 (48.77) m, 76.20 (73.15) m}
Support Skew	{0°, 30°, 60°}	{0°, 30°, 60°}
Cross-frame Spacing	{25 ft, 40 ft}	{7.62 m, 12.19 m}

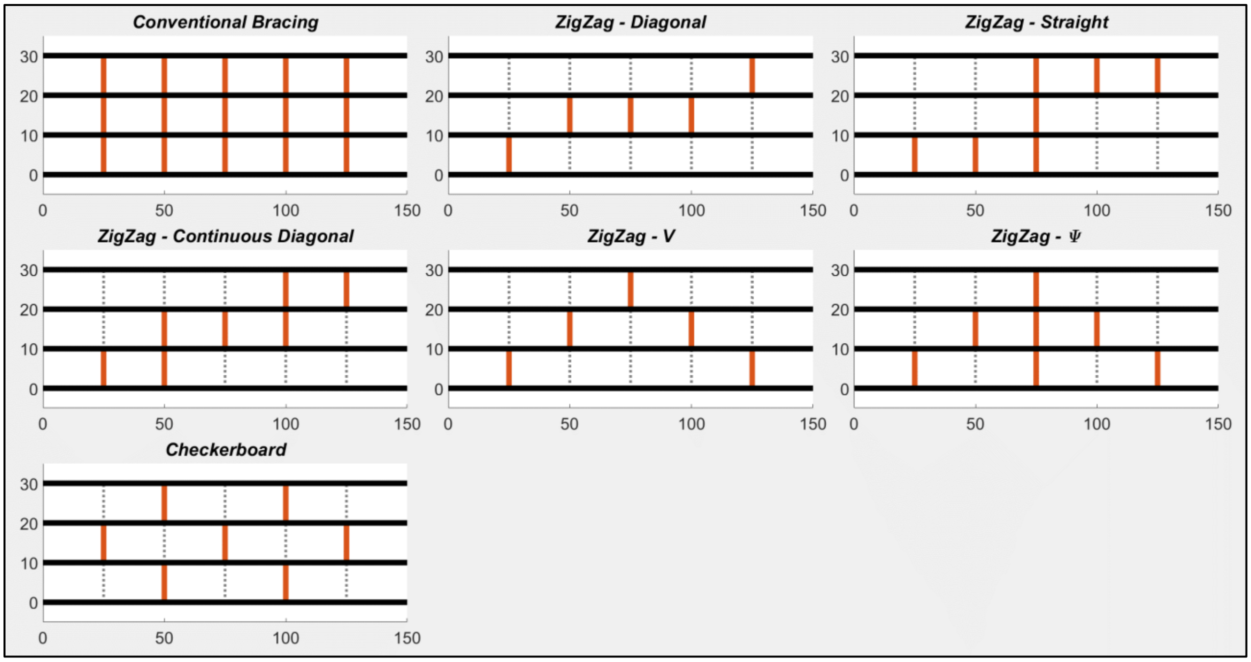

Figure 6. Cross-frame configurations utilizing lean-on bracing to be used in parametric study.

As noted earlier, the eigenvalue buckling analyses provide an indication of the ideal brace stiffness required. The ideal stiffness is determined from a load–stiffness curve. Due to moment gradient, unbraced segments adjacent to the critical segment often have moment levels below the critical moment. As a result, these adjacent segments provide warping restrain to the critical segment. However, similar to design specification that generally neglect warping restraint, the selected approach targets the stiffness necessary to reach M_{cr} based upon Eq. 1. The process is demonstrated by the curve shown in Fig 7. As the cross-frame stiffness is varied, the buckling capacity increases. The ideal stiffness is identified as the stiffness required to reach M_{cr} on the curve.

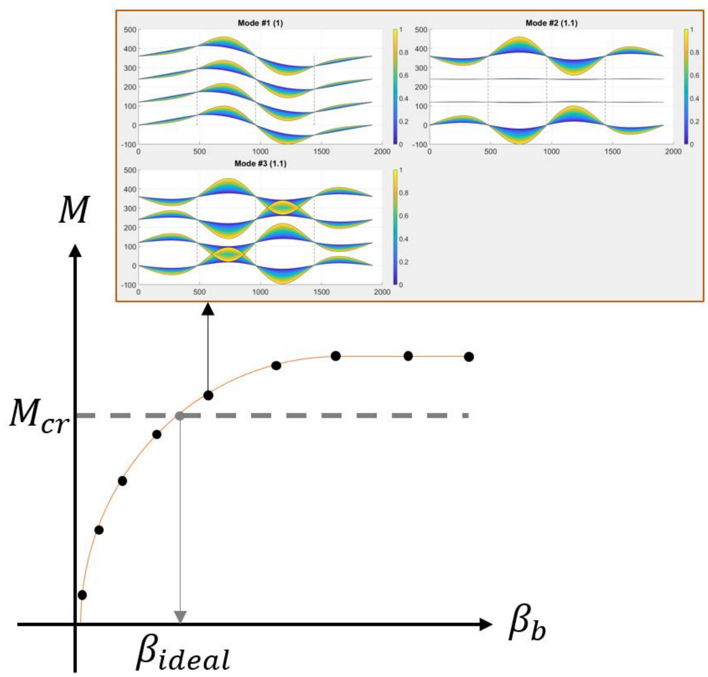

Figure 7. Example of a load-stiffness curve and the results of an included datapoint.

The results from various lean-on bracing configurations are compared with the behavior of the girder system with conventional bracing to evaluate the effectiveness of various cross-frame layouts.

5. Nonlinear Imperfection Parametric Study

A nonlinear geometrical parametric study is conducted in this investigation to evaluate brace forces that develop in the system. As noted in the last section, the eigenvalue buckling analyses are carried out to determine the ideal brace stiffness behavior. In practice, a stiffness larger than the ideal value is necessary to control brace forces. In general, the "target" increment on the ideal stiffness is one that results in a rotation at the brace location that is equal in magnitude to the initial imperfection, θ_o (total rotation is $2\theta_o$). Fig. 8 demonstrates the target girder moment-twist curve at the critical brace location. Therefore, the brace moment under these conditions is $M_{br} = \beta_T \theta_o$. In this study, the likely increment on the ideal stiffness will be $2\beta_{ideal}$. A complication associated with bridge geometry occurs in skewed girder systems. For skew angles larger than 20 degrees, AASHTO requires the cross-frame lines to be perpendicular to the longitudinal axis of the girders. As a result, the bracing lines intersection adjacent girders at different locations along the length. The braces therefore serve as both torsional and lateral bracing systems, which are often much more efficient compared to pure torsional bracing systems. As a result, the ideal stiffness behavior for skewed systems are determined from the normal bridge systems. The analysis will demonstrate the improved efficiency that occurs in terms of bracing performance in heavily skewed systems compared to systems with normal supports.

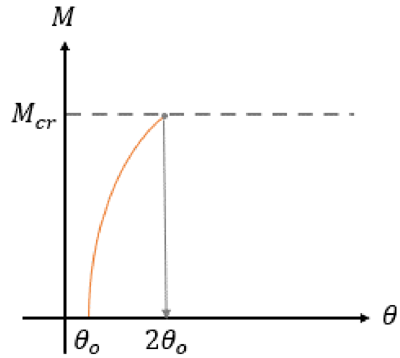

Figure 8. Example of normalized moment-twist curve.

The results will be normalized with respect to the load applied to each system (M_{cr}) and comparisons done between configurations will be normalized with respect to the conventional bracing case.

6. Conclusions and Future Work

The research paper provides an overview of an ongoing study on the performance of lean-on bracing systems. The study is considering a wide range of bracing configurations to develop modifications to design recommendations for lean-on bracing. The study is currently evaluating the efficiency of various bracing layouts for girder systems with both normal and skewed supports. A parametrical study is underway to evaluate the performance. The system is currently evaluating girders with normal supports with skewed girder system also to be evaluated. The finite element models for the system have been developed and validated using data from field instrumentations on three bridges with lean-on bracing.

The research team is currently in the process of completing the parametric studies described in the paper. Once completed, the research team will compare the results of each bridge configuration and grade each configuration based upon the stiffness and strength performance. Economic considerations such as the percentage of cross-frames to total bay locations will also be noted in the parametric study.

Acknowledgments
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