Lean-on Bracing Reference Guide

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Lean-on Bracing Reference Guide

National Steel Bridge Alliance

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Preface

Cross-frames are one of the costliest elements in a steel bridge on a per-pound basis. The unit cost of cross-frame steel is typically two to three times the cost of girder steel. Reducing the number and complexity of cross-frames would have an immediate, direct, and significant impact on the cost and speed of fabricating and erecting steel bridges. Cost-effective cross-frames can be approached from two directions: a simplified design approach, like lean-on-bracing, a simplified cross-frame detail, or a combination of both. Implementation of lean-on bracing would potentially eliminate 50-75% of the full cross-frames required for a routine steel I-girder bridge without adding any cost to the girders. This is estimated to reduce the cost of the steel superstructure by 20% or more, reduce the time needed to erect a steel I-girder bridge by days or weeks, and reduce the number and severity of fit-up problems during erection.

One of the easiest methods for achieving cost-effectiveness would be to implement lean-on bracing concepts for straight steel I-girder bridges with little or no skew. Academic research, including real bridge demonstration projects, have already been completed; however, the industry has been slow to adopt the concept. This guide for how to implement lean-on bracing is written to educate designers and allay concerns about stability and strength implications. The guide provides design criteria, commentary, and example designs. This guide is also intended to show bridge designers how to implement lean-on bracing in routine bridge designs with confidence with minimal computational effort beyond that required for a traditional bracing system.

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Chapter 1. Introduction

1.1 STABILITY BRACING FUNDAMENTALS

Conventionally constructed steel bridge I-girders, with a concrete deck placed after girder erection, need bracing to optimize the strength of their compression flanges. Bridge designers provide bracing in their designs which works to prevent lateral-torsional buckling of the girders, with the critical construction stage for buckling usually occurring during placement of the deck.

Girder bracing comes in two forms—lateral bracing and torsional bracing—with the effectiveness of each form being determined by its ability to control girder twist and lateral-torsional buckling. A composite concrete deck, once it has achieved its design strength, is an example of continuous lateral bracing for top flanges. Another common form of steel girder lateral bracing is a horizontal truss placed between two girder flanges, which is frequently provided to resist lateral loads such as wind.

Provision for I-girder torsional bracing has been the focus of AASHTO bridge design specifications since their inception in 1931. This torsional bracing is in the form of cross-frames and diaphragms. Figure 1-1 and Figure 1-2 depict diaphragms and cross-frames, respectively. Prior to use of composite deck construction, cross-frames and diaphragms were generally the sole source of girder compression flange bracing for all loads. With modern composite girder design, cross-frames and diaphragms are primarily needed to brace for non-composite dead loads, construction loads, and wind loads. Once the composite deck has achieved its design strength, it becomes a continuous lateral brace for the top compression flange; bottom compression flanges remain braced by the cross-frames or diaphragms. However, a composite deck will provide some level of torsional bracing for the bottom flanges. Other roles for cross-frames and diaphragms include:

- Distribution of lateral forces such as wind into the deck and into supports.
- Potential provision of redundancy in an extreme event
- Assists in distribution of live load among girders

The last two items—provision for redundancy and live load distribution—are not explicitly addressed in the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO 2024). All references to these specifications in this Guide will be noted as AASHTO LRFD unless noted otherwise.

Figure 1-1, Example of Diaphragms (Modjeski and Masters)

Figure 1-2, Example of Cross-Frames (TxDOT)

The design of cross-frames and diaphragms for straight I-girders has remained relatively unchanged since the beginning of AASHTO design specifications, even with the introduction of composite girder design, with member sizes frequently determined by slenderness requirements, rules of thumb, and their ability to resist calculated wind loads (Reichenbach et al., 2021). Cross-frames and diaphragms for skewed straight bridges analyzed with refined methods, horizontally curved girders, box girders, and tub girders are an exception to this approach and are not addressed in this Guide unless noted otherwise.

1.1.1 Introduction of Lean-On Bracing

Lean-on bracing is relatively new to steel bridges but has been used in the building and construction industries for several years (Helwig and Yura, 2022). An example of lean-on bracing with bridge construction is depicted in Figure 1-3, where one diagonal brace is used for the formwork of multiple columns, with adjacent column forms "leaning-on" the one diagonal brace via horizontal struts.

Figure 1-3, Lean-On Bracing Used for Column Formwork (Zachry Construction Corporation)

The historic approach to bridge girder torsional bracing has generally worked well with few problems encountered (Reichenbach et al., 2021). As research on beam torsional bracing advanced (Yura, 2001), it was observed that torsional bracing must provide both strength and stiffness to be effective. Traditionally proportioned cross-frames and diaphragms were often observed to provide an abundance of bracing strength and stiffness, enough so that one cross-frame or diaphragm possessed stiffness and strength for more than two girders. This led to the fundamental concept of lean-on bracing with steel I-girders, where one cross-frame or diaphragm braces multiple girders through top and bottom horizontal struts connecting adjacent girders to the girders braced with the cross-frame or diaphragm. These other girders "lean-on" one cross-frame or diaphragm for their torsional brace needs via horizontal struts (Helwig and Wang, 2003). Figure 1-4 illustrates the top and bottom horizontal struts of a lean-on bracing system. In this case, both struts are single angle members.

Figure 1-4, Lean-On Struts Installed During Erection (TxDOT)

1.1.2 Benefits of Lean-On Bracing

Lean-on bracing provides multiple benefits that intersect with the needs and desires of steel bridge owners, fabricators, and erectors. These benefits outweigh the modest efforts involved in their design and erection engineering.

- Improved structural performance
- Improved long term durability
- Simplified inspections
- Lower cost
- Easier fabrication
- Easier erection

<u>Improved Structural Performance:</u> Stiffness can attract undesirable forces in a torsional brace system. Traditionally proportioned cross-frames or diaphragms for straight steel I-girders typically provide more than the minimum necessary brace strength and stiffness. Use of lean-on bracing allows these stiff brace elements to be minimized in number and optimally located in a framing plan, especially with skewed bridges. Reducing system brace stiffness to the minimum level required limits undesirable stiffness and the loads it attracts, which leads to an improvement in overall structural performance.

<u>Improved Long Term Durability:</u> Lean-on brace struts are typically end bolted to cross-frame connection plates that are welded to girders. Without Category E' details, which are commonly encountered with most cross-frame member ends, lean-on bracing will improve fatigue life of bracing. This further complements the overall lower live load induced stress ranges in a lean-on bracing system.

Except for unpainted weathering steel applications, the simplicity of the horizontal struts presents fewer details that are inherently more difficult to clean and coat with a coating system, during both fabrication and with maintenance coats. Multiple lean-on brace struts can be easily and economically hot-dipped galvanized, offering a bridge owner an excellent corrosion protection to bridge bracing elements.

Simplified Inspections: The top and bottom struts of a lean-on brace will typically have no welded details subject to fatigue cracking and have fewer places for corrosion to occur. Both of these characteristics will simplify in-service bridge inspections.

Lower Cost: Compared to traditional cross-frames, lean-on bracing reduces material usage and reduces the amount of time for fabrication and steel erection, resulting in reduced initial costs. Long-term costs are also reduced as lean-on bracing has fewer members to maintain and inspect when compared to cross-frames.

Easier Fabrication: Lean-on brace struts for a wide range of steel bridges require only single angle members that need to be cut to length and have bolt holes punched or drilled. Lean-on bracing removes multiple welding and handling steps needed for full cross-frames, which require a jig frame. These frequent handling and welding steps have led fabricators to state that cross-frame steel is the most costly on a per pound of steel basis for bridge fabrication..

Easier Erection: Lean-on brace struts for a wide range of steel bridges are expected to be single angles with two bolts per end connection. Lean-on brace struts are not as prone to fit-up issues when compared to stiffer cross-frames and diaphragms. The holes in these struts can be lined up quicker with drift pins and bolts than can be achieved with a stiff cross-frame or diaphragm member. This allows overall construction time to be reduced.

1.2 LITERATURE REVIEW

1.2.1 Beam Bracing

Yura (2001):

This paper provides a comprehensive look at stability bracing for beams. Various factors that affect bracing requirements are discussed, along with examples. Concepts developed for column bracing by George Winter (Lateral Bracing of Columns and Beams, 1960) are discussed and are used to determine bracing requirements for beams. Beam bracing is discussed for two structural system—lateral and torsion bracing. Comparisons between torsional bracing and lateral bracing are made where torsional bracing was found to be less sensitive to top flange loading, number of bracing and brace location, but more sensitive to cross-sectional distortion. The findings from this paper provide a contribution to the provisions made for beam bracing which were introduced in the 1999 AISC-LRFD Specification.

Reichenbach et al (2021):

NCHRP Report 962 proposes specifications for analysis and design of cross-frames in straight or curved steel I-girder bridges. The proposed specifications were adopted for use in the AASHTO Bridge Design Specification, 10th Edition (AASHTO 2024). Thorough analytical and testing programs were carried out to investigate the effects of design forces due to fatigue, skew angle of the bridge supports on cross-frame behavior, requirements for strength and stiffness of stability bracing, and the effect of cross-frame member end connection details on the stiffness of the cross-frame.

1.2.2 Lean-On Bracing

Helwig and Wang (2003):

This report helps provide an overall understanding of the bracing behavior of cross-frames and diaphragms in steel bridges with supports that are skewed from 0 degrees to 45 degrees. Parameters such as girder geometry are also discussed in the investigation in establishing general requirements for bracing. Doubly symmetric sections and single symmetric plate girders are used in the investigation. Improved bracing details are also discussed, with a focus on reducing brace forces induced from live load. Lean-on bracing systems, which help in reducing the total number of cross-frames and with reduced bracing member forces, are also discussed.

Helwig and Yura (2022):

This Steel Bridge Design handbook chapter explains the design of bracing systems for bridges with straight and curved girders. Bracing requirements for I-girders and tub girders are discussed and explained in detail. Design requirements for cross-frame members and end connections are discussed along which assist in sizing the members and end connection plates.

1.2.3 Erection Engineering

Garlich et al (2015):

This manual serves as a reference for erection engineering. It describes different critical construction stages and practices for bridge girder superstructures. Differences between the member local and global systems are made for stability analysis. It provides guidance on evaluation of girders for structural stability throughout the construction phase. Basic and advanced methods of stability analysis are explained. Emphasis is also given on different stages of construction and their corresponding load effects and factors to use while analyzing the bridge members for stability during construction. It also discusses development of erection plans which provide safe and economical erection of bridge superstructures.

Chapter 2.
Design Approach to Lean-On Bracing

2.1 GEOMETRY

A limited number of considerations need to be made by the owner and designer prior to using lean-on bracing. Its multiple benefits suggest its use for many steel bridges but there are cases where lean-on bracing should generally not be used such as with horizontally curved bridges or straight skewed bridges with staggered diaphragms or cross-frames.

Bridge geometry variables such as span length, support skew, bridge and girder alignment, bridge width and girder flare, constant web depth vs variable web depth, etc., all play a role in the decision-making process in girder bracing system design. How the bridge is built, such as phased or staged construction, is another design consideration.

Decisions made in the final structure's design impact how an erection engineer and contractor determine a safe and stable erection procedure.

2.1.1 Lean-On Bracing Applicability and Analysis Recommendations

Lean-on bracing is appropriate for straight steel girders, with either normal or skewed supports. Horizontally curved girders should generally be excluded from consideration unless the curvature is mild enough that the girders may be analyzed as individual straight girders as noted in AASHTO LRFD Article 4.6.1.2.4b. Lean-on bracing should not be used for horizontally curved bridges beyond the limits noted in this article. Cross-frames for horizontally curved girders need to resist shears that the horizontal struts of lean-on bracing are not designed and detailed to resist.

AASHTO LRFD Commentary C6.7.4.1 notes live load forces in cross-frames or diaphragms for straight bridges, in the finished structure, are usually small enough that a refined analysis to quantify live load forces is unnecessary. For bridges with a skew index, I_s, defined in AASHTO LRFD Equation 4.6.3.3.2-2, less than or equal to 0.3, skew effects on cross-frames or diaphragms are mild enough to not warrant a refined analysis. For bridges in this category, a simple line girder analysis that does not capture live load force effects in the finished structure's bracing is considered sufficient for safe and serviceable design. Since live load force effects do not need to be calculated, it is implied that the fatigue limit state is satisfactorily addressed without direct analysis.

For straight bridges with a large skew index, lean-on bracing can assist in reducing cross-frame and bracing forces, especially near span ends or intermediate supports (White et al., 2012)(Helwig and Wang, 2003). The presence of a high skew index does not preclude the use of lean-on bracing, given that refined analysis is performed by the designer.

2.1.2 Erection Sequence Assumptions

A steel bridge designer should have an assumed erection and deck placement sequence for decision-making during the design process. With lean-on bracing in the framing plan, it is recommended that the contract plans note the assumed erection sequence. Notes in the contract plans that allow a contractor or erector to use an alternative sequence are recommended. With conventional design-bid-build contracts, any contractor-proposed alternate should first be reviewed and approved by the Engineer of Record, then coordinated with the owner, fabricator and erector, as it will likely affect shop drawings, the erection plan, and as-built drawings. With other contract methods, such as design-build, the framing plan and erection sequence design can benefit from direct contractor or erector input in the design phase.

2.1.3 Framing Plan and Bracing Layout Recommendations

With lean-on bracing, the term n_{gc} is introduced to define the number of girders per cross-frame or diaphragms in a contiguous line. For example, if a six girder bridge has two cross-frames in two bays with the remaining three bays having lean-on bracing struts, its n_{gc} value is 3 (one cross-frame per three girders)—for that specific line of bracing. There should be only one design value of n_{gc} for an entire span or negative bending region. However, in a span or negative bending region, additional cross-frames or diaphragms may be added by the designer at discrete lines of bracing to potentially improve overall behavior during erection and in service. However, it is recommended to limit n_{gc} to a maximum of 4, i.e. at least one cross-frame or diaphragm for four girders. Note that the n_{gc} value for the bridge in the case study described in Section 4.1 is as high as 5; this bridge was successfully constructed and is performing as intended. As will be seen in in Section 2.2, Analysis, use of larger values of n_{gc} will likely require larger values of in-plane girder stiffness and will result in larger cross-frame brace forces. It should be repeated that lean-on bracing is not intended for use with a staggered pattern of diaphragms or cross-frames.

When using lean-on bracing in an overall stability bracing design, the goals are selective placement of cross-frames to minimize forces in cross-frames and provision of overall economy. The number of variables in bridge geometry is unlimited when considering skews, variation of skews, flares, superelevation, horizontal curves, etc. Staged construction and widenings are other variables that can lead to a large number of unique scenarios. With the focus of this Guide being routine steel girder bridges, general recommendations for a stability bracing layout using lean-on braces are:

- **Bearings**. A full line of cross-frames or diaphragms should be placed at lines of support.
- **Peak positive moment region**. The designer may consider a full line of cross-frames at, or adjacent to, the peak positive moment location or region. Note that neither case study bridge (see Chapter 4) used a full line of cross-frames at midspan.
- **Skewed support region**. Cross-frames should be located, where practicable, by a minimum of the larger of 4 times the girder flange width or 0.4 times the adjacent unbraced length from the end bearings (AASHTO LRFD C6.7.4.2). At a minimum for lean-on bracing systems, cross-frames should be placed no closer than approximately 4 ft from the end bearings (Helwig and Yura, 2022, and Helwig and Wang, 2003). Lean-on bracing can be helpful in reducing nuisance stiffness most effectively by placing cross-frames in the acute corners of skewed ends and lean-on braces in the obtuse corners (Helwig and Wang, 2003).
- **Field splices**. For optimal geometry control during erection, each girder should preferably have a cross-frame in place near the field splice locations which will help maintain the shape of the girder bay at the free-end before the next field section is erected. This applies to air splices, not ground splices. Note that this can be achieved without a cross-frame or diaphragm in each bay.
- **Flared girders**. Lean-on bracing struts are ideal for flared girder bays when contrasted with the comparative complexity of individual cross-frames. All stability bracing requirements still need to be met.
- **Staged construction or bridge widening**. Lean-on bracing struts can be effective for the bay between existing girders and newly erected girders. Differential girder deflection is a challenge for cross-frame or diaphragm installation in these bays and designers have employed a variety of approaches to address it. Lean-on bracing can be a useful tool for designers in these situations due to the minimal in-plane stiffness of lean-on brace struts.

Two important observations can be made from these recommendations. First, for skewed spans with parallel or nearly parallel supports, the general recommendations above lead to an S-pattern that starts at the end acute corner, crosses the peak positive bending region, and ends at the next acute corner. This can also be applied to bridges with normal supports, with the perceived benefit of having all girders in the cross-section having a cross-frame or diaphragm attached to it. This S-pattern is depicted in Figure 2-1 and Figure 2-2 for skewed and normal bridges, respectively. Both case study bridges (see Chapter 4) and examples (see Chapter 5) use this S-pattern.

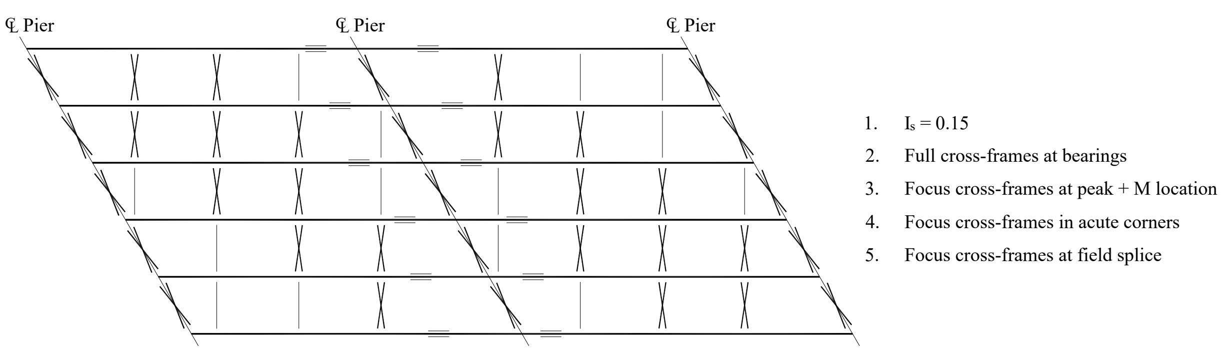

Figure 2-1, Skewed Bridge Framing Plan, with "S-Pattern" of Cross-Frames

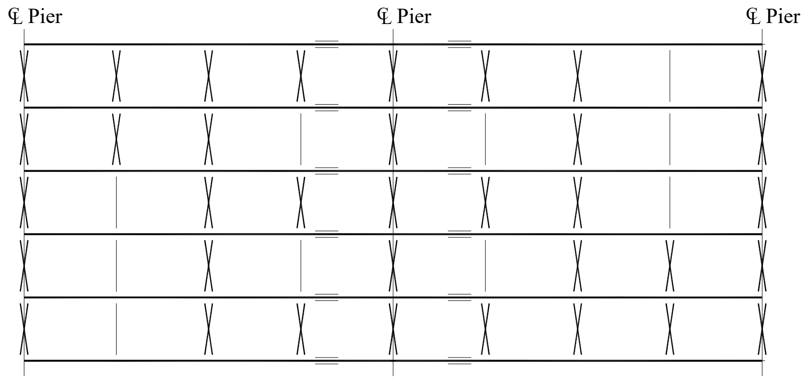

Figure 2-2, No Skew Bridge Framing Plan with "S-Pattern" of Cross-Frames

The second observation from these general recommendations is that erection safety and stability is ensured with cross-frames at supports, peak positive bending regions and at air field splices.

2.1.4 Preliminary Cross-frame and Diaphragm Sizing Considerations

AASHTO LRFD Article 6.7.4.2 recommends cross-frames and diaphragms depth be at least 75% of a girder's depth and at least 50% of a rolled beam's depth. A deeper bracing element offers two advantages, less strength demand on the bracing member(s) and improved stiffness in regard to cross-section distortion. As will be seen later in the section on analysis, the cross-sectional distortion component to overall stability bracing stiffness is optimized with deeper cross-frames and diaphragms.

2.2 ANALYSIS

The design approach for any stability brace system addresses two main items—stiffness and strength. For many bridge geometries that lean-on bracing is recommended for, such as straight bridges with mild skew, direct calculation of live load force effects is unnecessary, which removes the need to specifically address the fatigue and fracture limit state in the bracing design.

Stability bracing provisions were added to the AASHTO LRFD 10th edition, in Article 6.7.4.2.2, Stability Bracing Requirements (AASHTO 2024). These provisions are provided in the Appendix. In these design provisions, the construction stage is considered critical for bracing requirements. For the construction stage, the loads used for stability bracing are the non-composite dead loads and construction loads such as removable formwork, deck finishing equipment, work platforms, construction live load, etc. AASHTO LRFD Article 3.4.2.1 prescribes two sets of load factors for construction loads with steel structures at the strength limit state; load factors for wind are constant for both sets. Focusing on dead and live loads only, the first set uses load factors of 1.25 (minimum) for DC and DW with 1.5 (minimum) for construction live load. The second set is specific to *primary* steel superstructure components applied to the *fully erected* steelwork [emphasis added]. For this specific case, a minimum load factor of 1.4 is applied to both DC and construction LL. With cross-frames being defined as secondary members per AASHTO LRFD Table 6.6.2.1-1 and not primary members, for the straight steel girders for which lean-on bracing may be used there is ambiguity in load factors for the design of stability bracing. For routine steel girder bridges, any difference between these two sets of factors is unlikely to be of significance. It is noted that design engineers are frequently unaware of specific construction live loads in a design-bid-build environment; this lack of knowledge can lead to designs with high levels of conservatism.

In the provisions of AASHTO LRFD Article 6.7.4.2.2, there are three components to the overall stability brace stiffness provided, $(\beta_T)_{act}$:

- β_{br}, the brace stiffness of the diaphragm or cross-frame. For a lean-on bracing system, the equation for β_{br} requires modification from what is prescribed in AASHTO LRFD Article 6.7.4.2.2. (Helwig and Wang, 2003).

- β_{sec}, the cross-sectional distortion stiffness
- β_g, the effective in-plane girder stiffness. For a lean-on bracing system, the equation for β_g requires modification from that prescribed in AASHTO LRFD Article 6.7.4.2.2. (Helwig and Wang, 2003).

If a sufficiently deep cross-frame or diaphragm is provided, one that is at least 80% of the girder depth, β_{sec} can be taken as infinity and can therefore be ignored in determining $(\beta_T)_{act}$.

2.2.1 Cross-Frame Type Selection

AASHTO LRFD Article 6.7.4.2.2 addresses three types of cross-frames and two types of diaphragms when determining the brace stiffness, β_{br}. For routine steel girder bridges, a tension-only diagonal, X-type cross-frame is optimal for several span geometries and is an effective and efficient use of material (Helwig and Yura, 2022). The development of the brace stiffness equations for X-type, tension-only diagonal cross-frames used with lean-on bracing is presented in Helwig and Wang, 2003. This reference can be used by designers to derive the needed adjustment to β_{br} for other cross-frame types and diaphragms.

The changes needed to extend the provisions of AASHTO LRFD Article 6.7.4.2.2 for lean-on bracing used with X-type, tension-only diagonal cross-frames are presented in Table 2-1. Note that a simplification is made in the modified equations for β_{br} by assuming the horizontal and diagonal cross-frame members are the same size. This simplification is also used in the examples in Chapter 5 and Chapter 6.

Table 2-1, Changes to AASHTO LRFD to Implement Lean-On Bracing with X-type, Tension-only Diagonal Cross-frames

Changes to AASHTO LRFD Article 6.7.4.2.2 Required for Lean-On Bracing with X-Type, Tension-Only Diagonal Cross-Frames			
Components to Brace Stiffness Provided, $(\beta_T)_{act}$	**Article 6.7.4.2.2**	**Modifications to Art. 6.7.4.2.2 for Lean-On Bracing (X-type, tension-only diagonal cross-frames)**	
		Exterior Bay	**Interior Bay**
β_{br}	$\dfrac{ES^2h_b^2}{\dfrac{2L_d^3}{A_d}+\dfrac{S^3}{A_s}}$	$\dfrac{ES^2h_b^2}{n_{gc}L_d^3+S^3\left(n_{gc}-1\right)^2}A_b$	$\dfrac{ES^2h_b^2}{n_{gc}L_d^3+S^3\left(n_{gc}/2\right)^2}A_b$
β_{sec}	∞ (infinity) if cross-frame or diaphragm is at least 80% of girder depth	No change	
β_g	$\dfrac{24\left(n_g-1\right)^2 S^2 E I_x}{n_g L^3}$	$\dfrac{12\left(n_g-1\right)^2 S^2 E I_x}{n_g L^3}$	

Note: In this Guide, for β_{br} with lean-on bracing, A_d and A_s are taken as the same size and are labeled A_b.

2.2.2 Design Process

Referencing AASHTO LRFD Article 6.7.4.2.2 (see Appendix), the modified design process for a framing plan that uses X-type, tension-only diagonal cross-frames and lean-on brace struts, is summarized below. This process is used in the examples presented in Chapters 5 and 6.

1. Determine bracing stiffness requirements
    a. Determine stiffness required $(\beta_T)_{req}$ for the span or negative bending region under consideration with AASHTO LRFD Equation 6.7.4.2.2-2.
    b. Determine the stiffness provided $(\beta_T)_{act}$ with the proposed layout of cross-frames and lean-on brace struts with AASHTO LRFD Equation 6.7.4.2.2-5, with its three components:

i. Brace stiffness component, β_{br}, using a modified version of AASHTO LRFD Equation 6.7.4.2.2-6 for tension-only diagonal, X-type cross-frames. For skews up to 20 degrees and with the cross-frames and lean-on brace struts placed parallel to the skew, β_{br} is reduced by taking $\beta_{br,\ skew} = \beta_{br} \cos^2 \theta$.
ii. In-plane girder stiffness component, β_g, using a modified version of AASHTO LRFD Equation 6.7.4.2.2-12.
iii. Cross-section distortion stiffness component, β_{sec}. For appropriately deep cross-frames, those that are at least 80% of the girder depth, this stiffness component may be ignored as it will not affect the overall stiffness provided.

2. Determine the required brace strength, M_{br}, for non-composite dead loads and construction loads, using AASHTO LRFD Equation 6.7.4.2.2-13 at the strength limit state. For skews up to 20 degrees and with the cross-frames and lean-on brace struts placed parallel to the skew, M_{br} is replaced with $M_{br,skew} = M_{br} / \cos \theta$, where θ is the skew angle. This accounts for additional forces resulting from the skewed brace orientation.
   a. Apportion loads to cross-frame members and struts and evaluate member tensile and compressive resistance.
3. Verify slenderness limits of members selected.
4. Design cross-frame and lean-on brace strut connections to connection plates, as normally done.
5. Address wind load on structure, WS, as normally done.

In the following design process, all variables and units are defined in AASHTO LRFD, 10th Edition, unless noted otherwise (see Appendix).

2.2.3 Step 1a, Determine Required Brace System Stiffness, $(\beta_T)_{req}$

AASHTO LRFD Equation 6.7.4.2.2-2 is used to determine the required torsional brace system stiffness.

AASHTO LRFD Eqn. 6.7.4.2.2-2, $(\beta_T)_{req} = \dfrac{3.6\,L}{\phi_{sb} C_b^2\, n\, I_{eff}\, E} M_u^2$

2.2.4 Step 1b, Determine Brace System Stiffness Provided, $(\beta_T)_{act}$

AASHTO LRFD Equation 6.7.4.2.2-5 is used to determine the stiffness of the brace system provided.

AASHTO LRFD Eqn. 6.7.4.2.2-5, $(\beta_T)_{act} = \dfrac{1}{\dfrac{1}{\beta_{br}} + \dfrac{1}{\beta_{sec}} + \dfrac{1}{\beta_g}}$

where

β_{br} = brace stiffness of the diaphragm or cross-frame
β_{sec} = cross-sectional distortion stiffness
β_g = effective in-plane girder stiffness

Determine β_{br}:

AASHTO LRFD Equation 6.7.4.2.2-6 is used to calculate the stiffness of the cross-frame in a normal torsional brace system. With a lean-on bracing system, this equation needs modification to account for the number of girders leaning on a cross-frame (Helwig and Wang, 2003). The modification is based on the location of the cross-frame, exterior bay vs. interior bay, as follows:

Modified AASHTO LRFD Eqn. 6.7.4.2.2-6, $\beta_{br} = \dfrac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc} - 1\right)^2} A_b$, for cross-frames in exterior bays

Modified AASHTO LRFD Eqn. 6.7.4.2.2-6, $\beta_{br} = \dfrac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc}/2\right)^2} A_b$, for cross-frames in interior bays

where

> n_{gc} = number of girders per cross-frame for a discrete line of bracing across the bridge width (recommended to not exceed 4).
>
> A_b = gross area of cross-frame members, assuming all members are the same size for simplicity (in.^{2}). For single-angle and flange-connected tee-sections, the gross area is factored by 0.65.

It is conservative to use the modified equation for exterior bays in all cases.

If the bracing is placed parallel to the skew (for skews up to 20 degrees), β_{br} is factored by the square of the cosine of the skew angle, θ.

$$\beta_{br,skew} = \beta_{br} \cos^2 \theta$$

Determine β_{sec}:

AASHTO LRFD Equation 6.7.4.2.2-11 is used to calculate β_{seci} for portions of the web or connection plate above and below the cross-frame:

AASHTO LRFD Eqn. 6.7.4.2.2-11, $\beta_{seci} = \dfrac{3.3E}{h_i}\left(\dfrac{D}{h_i}\right)^2 \left(\dfrac{1.5 h_i t_w^3}{12} + \dfrac{t_s b_s^3}{12}\right)$ and

$$1/\beta_{sec} = \Sigma(1/\beta_{\sec i})$$

For cross-frames that are sufficiently deep, at least 80% of the web depth, and connected to a full-depth connection plate, β_{sec} is sufficiently large that it will not affect $(\beta_T)_{act}$ and may be ignored.

Determine β_g:

AASHTO LRFD Equation 6.7.4.2.2-12 is used to calculate the in-plane girder stiffness, but it needs modification for a lean-on brace system (Helwig and Wang, 2003). The in-plane girder stiffness used for a-lean-on bracing system is conservatively taken as one-half that provided by AASHTO LRFD Equation 6.7.4.2.2-12, ,as follows:

Modified AASHTO LRFD Eqn. 6.7.4.2.2-12, $\beta_g = \dfrac{12\left(n_g - 1\right)^2 S^2 E I_x}{n_g L^3}$

With a lean-on bracing system's β_g being taken as one-half that of a normal torsional bracing system, the in-plane girder stiffness becomes very important in finding a design solution. In some cases, modest increases in girder depth, when possible, can benefit a bridge being designed with a lean-on bracing system.

2.2.5 Step 2, Determine Brace System Strength

In addition to satisfying the stiffness requirements, members of the cross-frame in a lean-on bracing system need to provide adequate strength.

The required strength of a torsional brace is calculated with AASHTO LRFD Equation 6.7.4.2.2-13:

AASHTO LRFD Eqn. 6.7.4.2.2-13, $M_{br} = \dfrac{0.008\, L\, L_b}{n\, E\, I_{yeff}\, h_o}\left(\dfrac{M_u}{C_b}\right)^2$

For skews up to 20 degrees, where cross-frames may be placed parallel to the skew, the required brace strength is increased to account for the skew, with the following equation as noted in AASHTO LRFD C6.7.4.2.2:

$$M_{br,skew} = \frac{M_{br}}{\cos\theta}$$

<u>Determine forces in brace members</u>:

It is important to recall that the equations for M_{br} (and $M_{br,skew}$) account for non-composite dead load (girders, bracing, formwork, and deck) and construction loads (screeds, work platforms, construction live loads, etc.) under the Strength I load combination. Strength I does not include wind on structure; Strength III and V include wind on the structure at the design wind speed per AASHTO LRFD Figure 3.8.1.1.2-1 and at a wind speed of 80 mph, respectively. Bridge owners may have specific construction loading or load combination requirements. Deck placement would not occur under these wind speeds and as such, engineering judgment needs to be applied in including wind on structure forces in the brace force members captured with M_{br} and $M_{br,skew}$.

For a X-type, tension-only diagonal cross-frame, the force in the top and bottom struts, F_{br}, is determined by (substitute $M_{br,skew}$ for M_{br} when appropriate) :

$$F_{br} = \frac{M_{br}}{h_b}$$

For a lean-on bracing system that relies on an X-type, tension-only diagonal cross-frame, Helwig and Wang (2003), provides equations to apportion F_{br} to the cross-frame members and lean-on brace struts, in consideration of the number of girders per cross-frame, n_{gc}, and whether the cross-frame is in an interior bay or exterior bay. These equations are as follows:

For exterior bays:

Diagonal member force, $$F_d = \frac{n_{gc} F_{br} L_d}{S}$$

Horizontal member force, $$F_s = \left(n_{gc} - 1\right) F_{br}$$

For interior bays:

Diagonal member force, $$F_d = \frac{n_{gc} F_{br} L_d}{S}$$

Horizontal member force, $$F_s = \left(n_{gc} / 2\right) F_{br}$$

With the design forces for the cross-frame tension diagonal (for X-type, tension-only diagonal cross-frame) and horizontal members, including lean-on brace horizontal struts, verify strength requirements using AASHTO LRFD Articles 6.8 and 6.9.

2.2.6 Final Steps

The remaining steps to complete the design are those normally required for any cross-frame design, which include verifying slenderness limits and connection design.

Chapter 3.
Fabrication and Erection Factors

3.1 FABRICATION

Lean-on bracing struts are likely the least complicated steel bridge members to fabricate. Considering that a typical lean-on bracing member is a single angle or WT section, a step-by step look at the advantages presented by lean-on bracing are evaluated individually:

- **Shop drawings**: Details are straightforward with uncomplicated geometry and multiple, simple to detail piece marks. If a shop drawing element is easy to develop, then it is also likely to be less demanding to review and approve. It is acknowledged that more coordination with the erection engineer may be needed during shop drawing development.
- **Material availability**: Single angles are typically sufficient for lean-on bracing members and these are readily available in most materials and sizes used for bridge construction. If a specific size is unavailable due to rolling schedules, it may be easy to substitute a slightly larger section without an undue cost penalty if the fabrication process is otherwise allowed to proceed without delay.
- **Handling**: Relatively light and short, single members used for lean-on bracing such as angles and WTs are obviously going to be less demanding for handling. In their simplest form for routine steel girder bridges, strut members are cut to length and bolt holes are drilled or punched at each end.
- **Welding**: Welding is typically not needed for lean-on bracing during fabrication—or erection—for typical steel bridges. Resources to weld and check workmanship are not required. This is likely the strongest fabrication advantage of lean-on bracing.
- **Surface Preparation**: Surface preparation needs will be minimal owing to lean-on brace strut member size and length and the compactness of bolted end connections.
- **Coatings**: Coatings are comparatively easy to provide due to the overall dimensions and weight of individual lean-on bracing strut members. Hot-dipped galvanizing is easily completed with lean-on brace struts.
- **Storage and Shipping**: It is self-evident that the size and compactness of lean-on bracing struts places less demand on storage area and allows for denser loading of piece marks per truck.
- **Repair or Replacement**: If a member is mis-fabricated, repair or replacement is straightforward.

With so many fabrication advantages of lean-on bracing, bridge design engineers should consider lean-on bracing and implement it where practical.

3.2 ERECTION

Erection of steel bridge girders is influenced by numerous factors such as the as-designed girder stiffness and weight, as-designed bracing type(s) and locations, contractor's or erector's equipment, site constraints on lift crane and/or holding crane placement, girder delivery location, crane capacity at extreme pick radius, shoring locations and limits, field splice locations, air splices vs ground splices, pier locations, etc. With so many factors unique for each bridge site, only general stability bracing requirements with respect to lean-on bracing can be identified here.

Garlich et al. (2015) is a recommended source for information on erection engineering. Many construction and erection factors are considered the contractor's means and methods and are beyond the control of a design engineer in a design-bid-build environment. As a result, bridge design engineers are limited with design actions impacting erection engineering other than satisfying AASHTO LRFD Articles 2.5.3 and 6.10.3 (AASHTO 2019). A bridge design engineer may be required to evaluate the acceptability of a girder erection plan.. At least one commercial software, mBrace3D, has the ability to analyze various erection sequences of a span or unit containing lean-on bracing.

Erection loads are generally dead load, construction live loads, and wind. These loads are constant with or without lean-on bracing.

3.2.1 Generic Erection Sequence and Procedure

Only a very generalized erection sequence and procedure for a routine continuous span steel girder bridge, straight and with none to moderate skew is outlined here, with consideration of lean-on bracing use. The sequence presented considers single girder erection only; paired girder erection typically provides for a more stable and quicker erection process. Steel units can be erected from end to end, or erect both ends and add drop-in sections afterwards. See Figure 3-1 for an example of a drop-in girder being erected.

Figure 3-1, Erection of Drop-in Girder (TxDOT)

Spans are generally erected in sections with all girder piece marks erected and braced together before moving to another section. These sections may include permanent bridge piers, shore towers, and/or holding cranes. In some cases, field splices are made on the ground, lengthening the girder section lengths. Again, site and equipment constraints will govern the exact sequence.

The first girder erected is commonly an exterior girder and the process adds adjacent girders until all girders in that section are erected. The first girder is erected on piers and/or shore towers. Some type of diagonal brace is used at these supports, such as the one shown in Figure 3-2. A holding crane or shore tower may be needed to maintain stability before the adjacent girder is erected and bracing is installed between them.

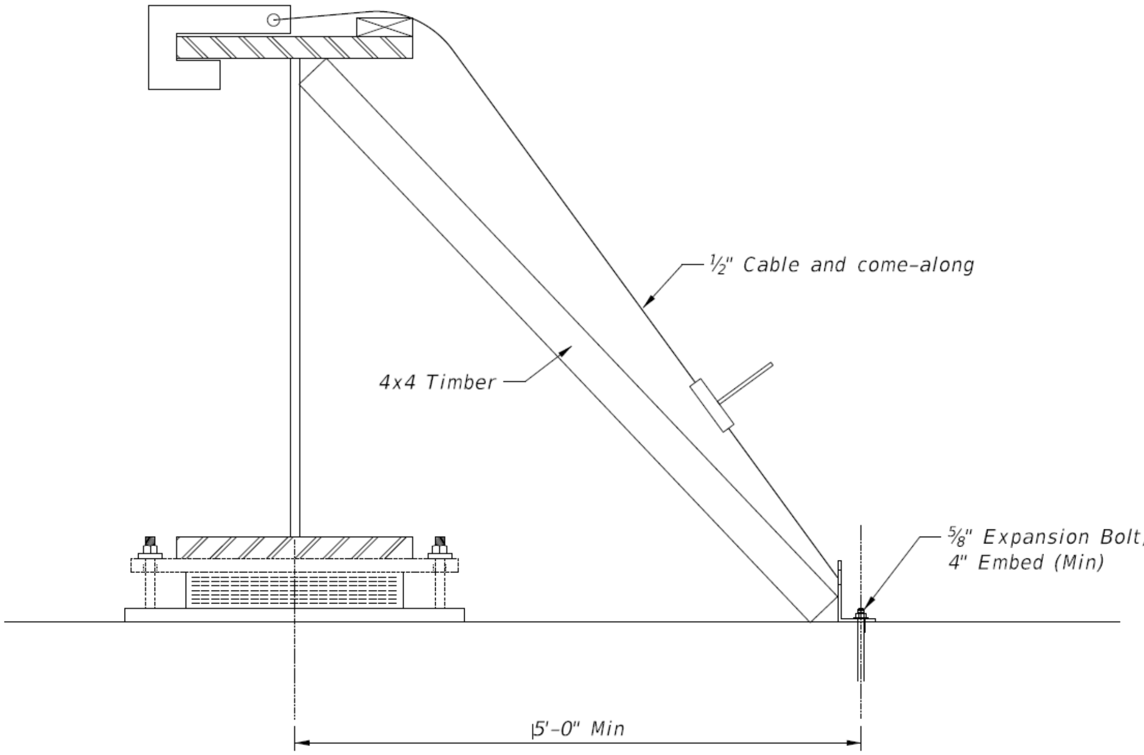

Figure 3-2, Example Diagonal Brace for First Girder Erected (Modjeski and Masters)

The adjacent girder sometimes has cross-frames—or lean-on brace struts—attached to it prior to being lifted into place. Once the second girder has been erected and a sufficient number of cross-frames have been installed to maintain stability, holding and lift cranes are typically released. It is not uncommon for only some cross-frames to be installed, which lets the erector proceed with further girder erection while crews follow-up with installing the remaining cross-frames. The erection engineer will be specific on the maximum spacing of cross-frames or list specific cross-frames that are mandatory prior to release of lift or holding cranes. The erection engineer will likewise be specific about the minimum number of bolts needed for the cross-frame connection to the girders.

3.2.2 Generic Erection Procedure Extended to Lean-On Bracing

With lean-on bracing, there is no change to the normal procedure for the first girder erected. This first girder will be braced at its supports and may have a holding crane to help maintain stability. If the designer followed the framing plan recommendations listed in Section 2.1.2, as the adjacent girder is brought in, permanent cross-frames will be installed at permanent bearings, near the maximum bending region, and near air splices. Based on past experience with routine steel girder bridges, this amount of permanent cross-frames should be sufficient to maintain stability and keep bending stresses within allowable limits, and allow subsequent girder erection. If not, the erection engineer can be expected to coordinate with the design engineer to develop an alternate bracing scheme. For the case study presented in Section 4.2, the maximum unbraced length for the center span drop-in girder was 128.6 ft. This was the maximum distance between cross-frames and stability was sufficient during erection.

Installation of lean-on brace struts is expected to improve erection speed, as they have been easier to install when compared to installation of cross-frames with connections at all four corners.

Figure 3-3, Partial Cross-frame Installation (Modjeski and Masters)

Chapter 4. Case Studies

4.1 CASE STUDY 1, US 82 MAINLANE UNDERPASS AT 19TH STREET WESTBOUND, LUBBOCK, TEXAS

4.1.1 Overview

The lean-on bracing system was used on three bridges in Lubbock, Texas. It was the first implementation of lean-on bracing by the Texas Department of Transportation (TxDOT). The bridges include the US 82 Underpass at 9th Street and the US 82 Mainlane (ML) Underpasses at 19th Street, Eastbound & Westbound (EB & WB). The US 82 ML Underpass at 19th Street WB bridge is the focus of this case study because it was the first bridge completely built and ultimately instrumented as part of a TxDOT research project.. In the finished state, the WB and EB underpass bridges are identical and are separated by a 1 in. open longitudinal joint between decks. Construction of the US 82 ML Underpass at 19th Street EB had not begun during instrumentation and live load tests of the WB Underpass. Detailed information on the instrumentation, field measurements, and results can be found in Romage-Chambers, 2003.

The bridge design and details were prepared by the Bridge Division of the Texas Department of Transportation. The winning contractor was Granite Construction Inc. Steel erection was by Choctaw Steel Erectors and steel fabrication was by W&W|AFCO Steel in San Angelo, TX. Construction was complete in 2008.

4.1.2 Unit Description

The two underpass bridges at 19th Street are 289.5 ft long, straight composite plate girder bridges. All supports are skewed approximately 60 degrees. Each bridge consists of two spans, with the first span 150.5 ft and the second span 139.0 ft. Overall bridge width is 47 ft, with an 8 in. thick composite concrete deck and two concrete barrier railings. The deck was formed with stay-in-place metal deck forms. Each bridge consists of six steel plate girders, spaced at 8.2 ft. The web depth is 54 in. and the flanges are 18 in. wide, with flange thickness varying from 1 in. to 2.75 in. All steel for the Lubbock bridges is uncoated weathering steel, ASTM A709 Grade 50W.

One or two cross-frames were provided along each intermediate bracing lines while top and bottom struts were provided in the remaining bays. The framing plan shown in Figure 4-1 illustrates the cross-frame and strut locations throughout the structure. The number of girders per cross-frame, n_{gc}, ranged from 6 to 3, depending on bracing line.

The bracing is oriented perpendicular to the girders at intermediate brace points. The braces at all supports are parallel to the skew and includes a cross-frame in each bay. Standard TxDOT cross-frames have top and bottom horizontal members(angles) paired with X-type diagonals. Connection to the girders is typically with a single erection bolt at each cross-frame corner to erect the girders, followed by field welds to fully connect the cross-frames to the connection plates or stiffeners. The lean-on brace horizontal struts were connected to girder connection plates with two bolts at each end; no field welding was used for the horizontal strut connections. All bracing members are L4×4×3/8.

With the exception of the first brace, cross-frames are primarily located in the acute corners of each span and near the center of the typical section at midspan, creating an "S-pattern." All other brace locations utilize lean-on brace struts. All lines of support have full cross-frames in each bay, placed parallel to the skew. The design used a strut at the first brace point adjacent to a support to reduce the overall stiffness of the system.

TxDOT designed and constructed this structure before the development of the current NCHRP Report 725 guidelines. The skew index, I_s, for this bridge is 0.51, well beyond the 0.3 limit suggested for line girder analysis without direct calculation of live load induced forces in bracing elements. It was analyzed with a line girder analysis program; however, the initial research team also analyzed the structure with a 3D finite analysis program. Hand calculations based on equations from Helwig and Wang (2003) were used to design the lean-on-bracing system.

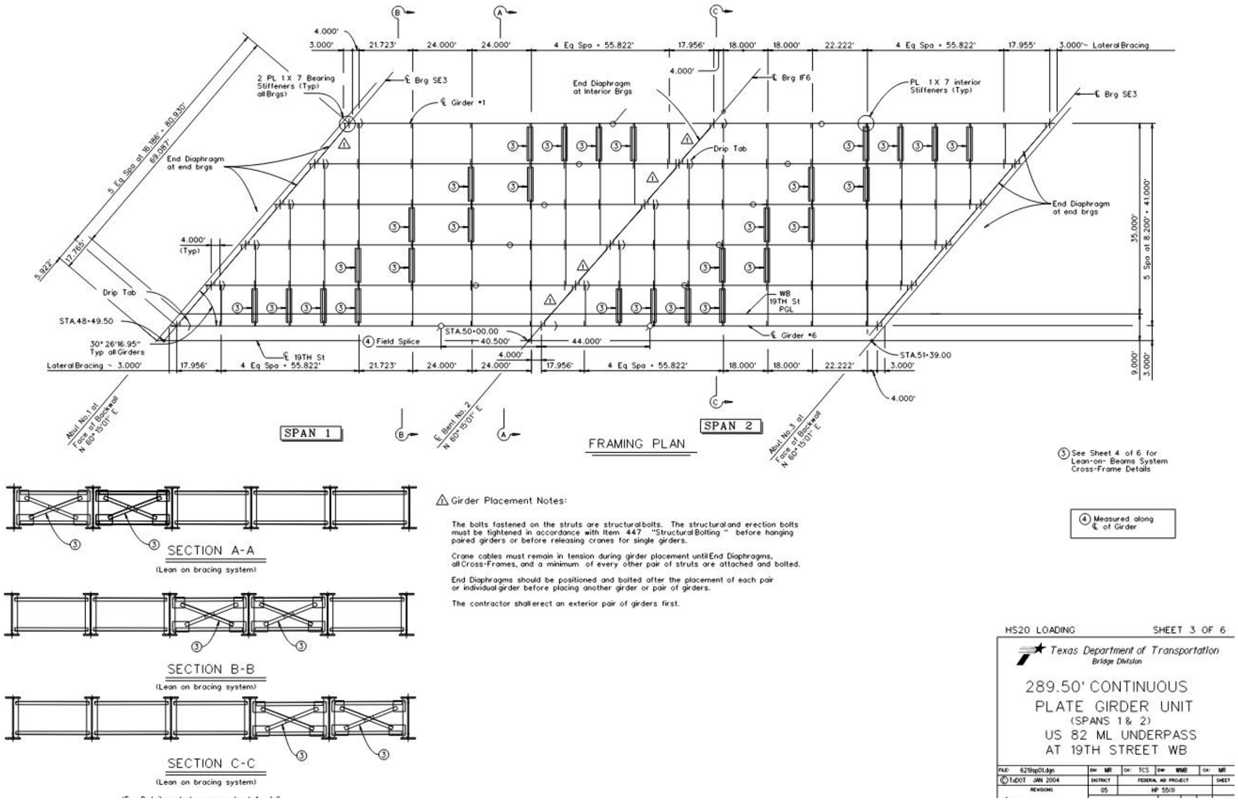

Figure 4-1, Framing Plan, US 82 Main Lane Underpass at 19th St WB (TxDOT)

4.1.3 Erection

The construction sequence listed in the contract plans was initially very prescriptive due to the reduction in cross-frames from a normal design. However, with contractor input, a few additional cross-frames were added, and the contractor was allowed more freedom in the erection procedure. The contractor chose to erect an exterior girder first, followed by adjacent girders, continuing across the structure width. Each girder was erected individually. Shore towers supported the girders beyond the interior bent until the completion of the second welded splice. TxDOT requires contractors to provide signed and sealed erection drawings for review. The erection sequence notes included in the plans required the erector to bolt up every cross-frame, end diaphragm and every other strut before releasing the cranes from the girder. The erector chose to bolt up all cross-frames and struts before erecting the next girder providing additional stability during girder erection. The erection went smoothly with no significant problems. Figure 4-2 shows girder erection and lean-on brace horizontal struts.

Figure 4-2, Lean-on Brace Struts seen during erection (Romage-Chambers)

4.1.4 Deck Placement

On October 4, 2007, the deck of the US 82 ML Underpass at 19th Street WB bridge was placed. TxDOT required the contractor to place the concrete and the screed approximately sixty degrees to match the skew of the deck. The screed was skewed to control the differential deflections occurring during concrete placement as required by TxDOT's Standard Specifications. The 8-in. thick concrete deck was formed with stay-in-place metal forms. The contract plans listed a concrete placement sequence with an option for continuous placement (see Figure 4-3). The contractor chose to continuously pour the concrete with a target placement rate of 69 cy/hr. However, the skew of the screed combined with difficulties in concrete delivery based on early morning traffic interfered with the ability to pour the concrete at the planned rate.

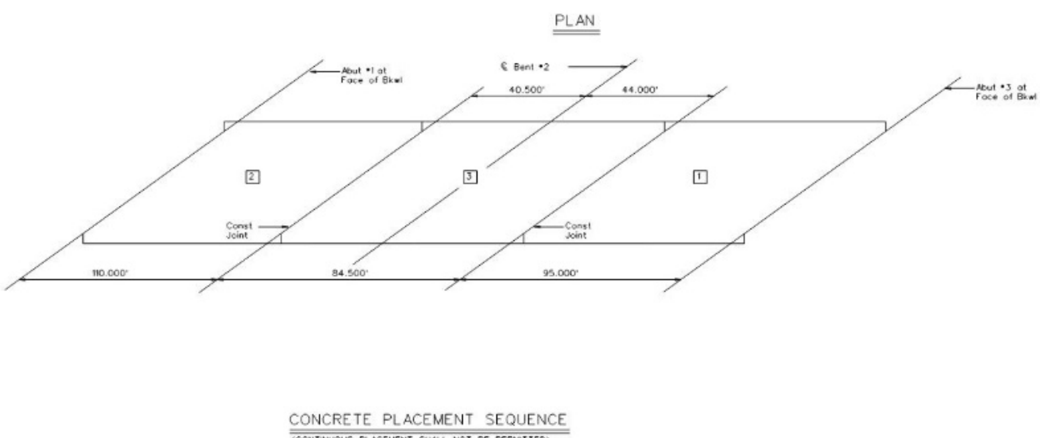

Figure 4-3, Plan Concrete Placement Sequence (TxDOT)

The contractor preloaded the end of the second span (see Figure 4-4) to control girder uplift during placement in the first span. Preloading of the second span began at approximately 7:00 while continuing the placement of concrete in the first span. At 11:15, deck placement was incomplete and the preloaded portion of the second span was no longer plastic, which led to consolidation problems in the second span. Concrete in the second span was placed and finished at 13:30. However, the consolidation problems with the deck were noted and later investigated by TxDOT, which showed excessive voids and honeycombing over much of the second span. The portion of the deck at end of the second span was removed and replaced later.

Although the original concrete in the second span had relatively poor quality, the effect on the live load testing after construction was minimal. The weight of the screed was 16.7 kips and was noted for the purposes of later analysis in comparisons with recorded data during the deck placement. Construction issues are normal. Overcoming construction issues signifies that the bracing will perform just as well as a full line of cross-frames regardless of the issues. No significant deflections or issues were noted during the removal and replacement of the problem sections of the deck, indicating that future deck replacements will not be problematic.

Figure 4-4, Pre-loaded Portion of Span 2 (Romage-Chambers)

4.1.5 Study Instrumentation

The US 82 ML Underpass at 19th Street WB was the first bridge to be constructed. It was selected for instrumentation during its construction as part of a TxDOT-funded research by the University of Texas-Austin. The cross-frames and struts of most interest were the cross-frame members near the skewed supports and at the center of the span. These two regions were deemed the most critical due to both the girder moments and bracing layout and were therefore selected for instrumentation. Strain gages were placed on three cross-frames, three pairs of lean-on brace struts and three girders. Tilt sensors were located along the girders to measure girder rotations. Deflections were manually measured using a laser distance meter.

Figure 4-5, Girder Instrumentation (Helwig)

Each cross-frame member and lean-on brace strut (all L4×4×3/8 single angle members) was instrumented with four strain gages. Three of the six girders were selected for instrumentation, with strain gages on each flange and three gages on the web at two locations along the length of the girder. Tilt sensors were placed to measure the girder rotations during the deck placement and live load test. Sixteen tilt sensors were placed on the girders 3, 4, 5, and 6. Four tilt sensors were placed on each girder (see Figure 4-5 and Figure 4-6). Deflection measurements were taken at eleven locations in the first span of the bridge.

Figure 4-6, Tilt Sensors Installed on Bottom Flanges (Helwig)

4.1.6 Cross-Frame and Strut Forces During Concrete Placement

The cross-frames and struts were designed for a 50 lbs./sf construction live load as well as the dead load of the girder and deck. The construction live load accounted for the weight of screed, workers, forms, and falsework. The live load is an upper bound that also covers imperfections, erection methods, and residual stresses.

Bracing is critical during deck placement. Before the deck is composite with the girders, properly designed torsional braces limit the twist of the girders, ensuring the stability of the structure. Once the deck hardens and becomes composite with the girders, the deck continuously braces the system by restraining lateral movement. Most intermediate cross-frames are no longer necessary in straight bridges once the deck is hardened.

The maximum change in cross-frame strains occurred in a cross-frame located near the center of span one at approximately 120 ft from the first abutment. The maximum measured force was 14.3 kips in tension that occurred in one of the diagonals with the peak cross-frame force in a horizontal member measured at 4.1 kips. The forces predicted by the equations from Helwig and Wang (2003) in this cross-frame were 26.4 kips (diagonal member) and 12.1 kips (horizontal member), in tension.

The maximum change in forces in a lean-on brace strut occurred in a top strut located at the edge of the bridge and near the supports. The predicted peak lean-on brace strut force was 14.9 kips, and the measured peak lean-on brace strut force was 9.2 kips. The actual forces were appreciably lower than the forces predicted by the equations. **The measured forces during deck placement were approximately one-half the predicted forces**.

4.1.7 Cross-Frame and Strut Forces During Live Load Test

On November 6, 2007, a live load test was performed on the US 82 ML Underpass at 19th Street WB. Two identical TxDOT sand trucks were utilized for the test. Each truck was weighed before and after the test. Prior to the tests, the total weights of the trucks were 48,500 pounds for the first truck and 48,960 pounds for the second truck.

The live load test consisted of six different live load patterns and one stationary measurement. The bridge rails were completed on the north end of the structure. However, the rail on the south end had not been placed. The patterns included staggered, side by side and front to back patterns. The trucks were moved to each location defined by a grid and held for sixty to ninety seconds to allow a minimum of three data readings. In each pattern, the outside truck was positioned so that the outside tire was positioned over the exterior girder.

The data collected indicated that the greatest change in forces occurred in the diagonals of the cross-frames. A maximum tensile force of 36.1 kips (cross-frame diagonal) was recorded. The maximum force occurred when the trucks were positioned 140 ft from the first abutment. The instrumented cross-frame is located 119.5 ft from the centerline of bearing and each truck is 17.8 ft in length. The maximum forces recorded during testing typically occurred in the end-to-end tests (see Figure 4-7 for an example end-to-end test).

Figure 4-7, End-to-End Live Load Test (Helwig)

4.1.8 Conclusions

Bracing systems with lean-on braces, using the recommendations from Helwig and Wang (2003), were successfully implemented on three skewed bridges in Lubbock. One of these bridges, the US 82 ML Underpass at 19th Street WB, was the subject of a research project during its construction. The objectives of the research were to compare predicted structural behavior with the measured structural behavior. Objectives were accomplished by measuring and evaluating the change in strains, rotations, and deflections during deck placement and live load tests. The design equations predict forces during deck placement. The forces measured proved that the equations were conservative. Once the deck is composite with the plate girders, the deck acts to restrain lateral movement and twist in the girders.

The live load tests provided an understanding of how the system with the composite deck displaces, tilts, and experiences forces as vehicles pass over the structure. The system performed as expected by deflecting, rotating as a unit like the lean-on systems used in buildings.

The observed structural behavior reinforced the conservatism of the lean-on brace recommendations in Helwig and Wang (2003). TxDOT, the researchers, and the contractor worked collaboratively during the bridge's construction to ensure the project's success.

TxDOT's routine bridge consultant inspectors examined all three structures multiple times over the years. The inspectors did not document bracing related issues with the three structures since completion of construction.

4.2 CASE STUDY 2, SH 155 AT BRAZOS RIVER BRIDGE, BRAZOS COUNTY, TEXAS

4.2.1 Overview

The Texas Department of Transportation let the construction of the State Highway 155 bridge at the Brazos River in 2014. This bridge has an overall length of 1780 ft and runs on a straight east-west horizontal alignment. The east approach has five spans at 142 ft each, all simply supported and framed with Type Tx62 precast, prestressed girders. The west approach has two 150 ft spans, framed with the same girder type. The advertised plans contained two alternatives for the 770 ft main river unit—a continuous steel plate girder unit and a continuous spliced precast concrete girder unit. All 9 bidding contractors selected the steel girder superstructure for their bid.

The site is subject to seasonal flooding, with ongoing stream migration to the west. The overall bridge length accommodates a nominal amount of stream migration. Bridge details accommodate future bridge lengthening.

Figure 4-8, SH 155 at Brazos River Bridge (TxDOT)

The bridge design and details, including both main unit alternates, were prepared by the Bridge Division of the Texas Department of Transportation. The winning contractor was James Construction Group, LLC. Steel erection was by Choctaw Steel Erectors; erection engineering was by Axiom Management & Engineering, Inc.; steel fabrication was by W&W|AFCO Steel in San Angelo, TX. .

4.2.2 Main Unit Description

The main river unit (Spans 6 through 8) is a 235 ft-300 ft-235 ft continuous plate girder fabricated with uncoated A709 Gr 50W steel.

Overall bridge width is 46 ft, with a nominal 44 ft wide roadway composed of two 10 ft shoulders and two 12 ft lanes. The supports are not skewed.

The main unit is composed of 5 girders spaced at 10 ft. The main unit typical section is shown in Figure 4-9. Precast, prestressed concrete sub deck panels are depicted for the 8 in. thick concrete deck; the contractor elected to use stay-in-place metal forms for the main unit in lieu of the concrete sub deck panels. Bridge railings are standard TxDOT Type T223, a concrete post-and-beam rail.

The girder web depth is constant at 92 in., with a thickness of 11/16 in. for the end and drop-in sections. The web thickness varies for the pier sections, which utilize 11/16 in. and 13/16 in. thick webs. The web designs are unstiffened, with no additional transverse stiffeners between cross-frame connection plates or bearing stiffeners.

All flange plates are 24 in. wide and vary in thickness from 1 in. to 3 in.

Length of the pier sections is 138 ft between field splices. The drop-in section in the center span is 140 ft between field splices. The end spans have a field splice 36 ft from the end supports which was ground spliced to a 140 ft long girder, for a total end section length of 176 ft. All field splices were designed as welded, with TxDOT providing optional bolted field splice details. The contractor selected bolted field splices.

All bearings are laminated neoprene. One interior support (Bent 8) is a fixed bearing, with all other bearings expansion-type. The end bearing furthest from the fixed bearing (Bent 6) utilizes a PTFE/stainless steel sliding interface. The other end bearing and expansion interior bearing (Bents 9 and 7, respectively) are conventional laminated neoprene bearings. Anchor bolts are provided to connect bearing sole plates to the reinforced concrete pier caps. The girders are field welded to the sole plates.

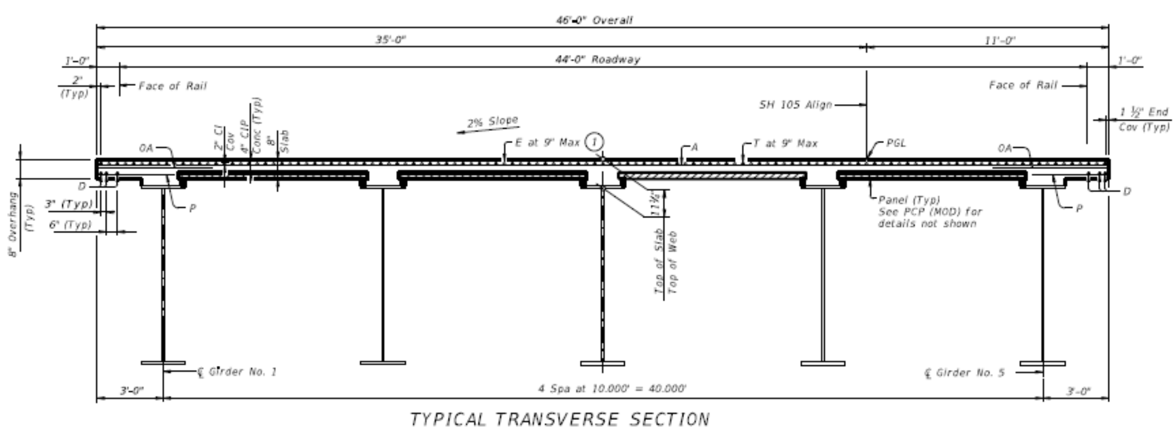

Figure 4-9, Typical Section (TxDOT)

4.2.3 Framing Plan

The framing plan for this bridge has cross-frames spaced at 21.27 ft in the end spans and at 21.43 ft in the main span. There are a total of 37 lines of bracing for the main unit. No lateral bracing is utilized.

At supports, full cross-frames are provided in each bay. Cross-frames and lean-on braces were used in all spans. Cross-frames are TxDOT Type XF-3, shown in Figure 4-10. All cross-frame members are L6×6×3/4, end welded to ½ in. gusset plates. The cross-frame is connected to the girders with a 1 in. Grade A325 erection bolt at each corner during erection. After erection, cross-frames are welded to the girder connection plates and bearing stiffeners.

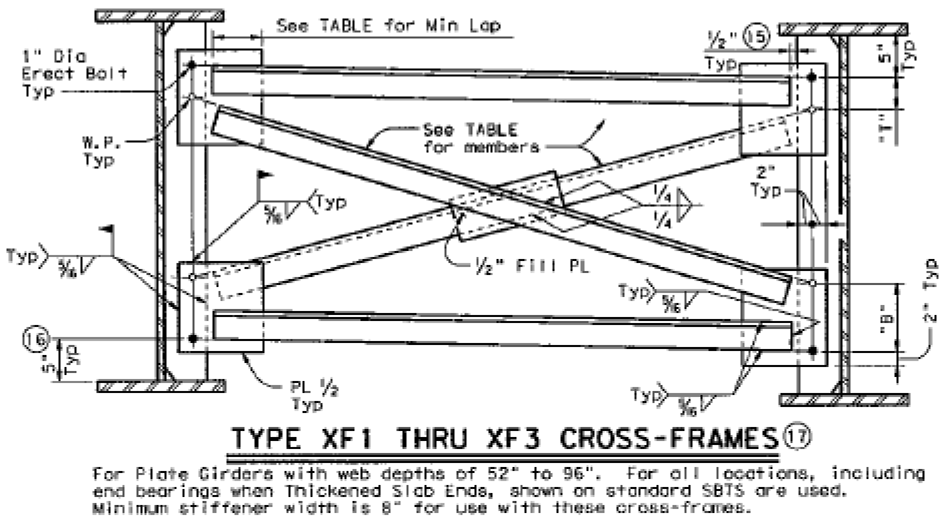

Figure 4-10, TxDOT Type XF-3 Cross-Frame (TxDOT)

Lean-on braces use L6×6×3/4 horizontal struts, top and bottom. These struts are bolted to the girder connection plates with two 1 in. Grade A325 bolts at each end. See Figure 4-11 and Figure 4-12.

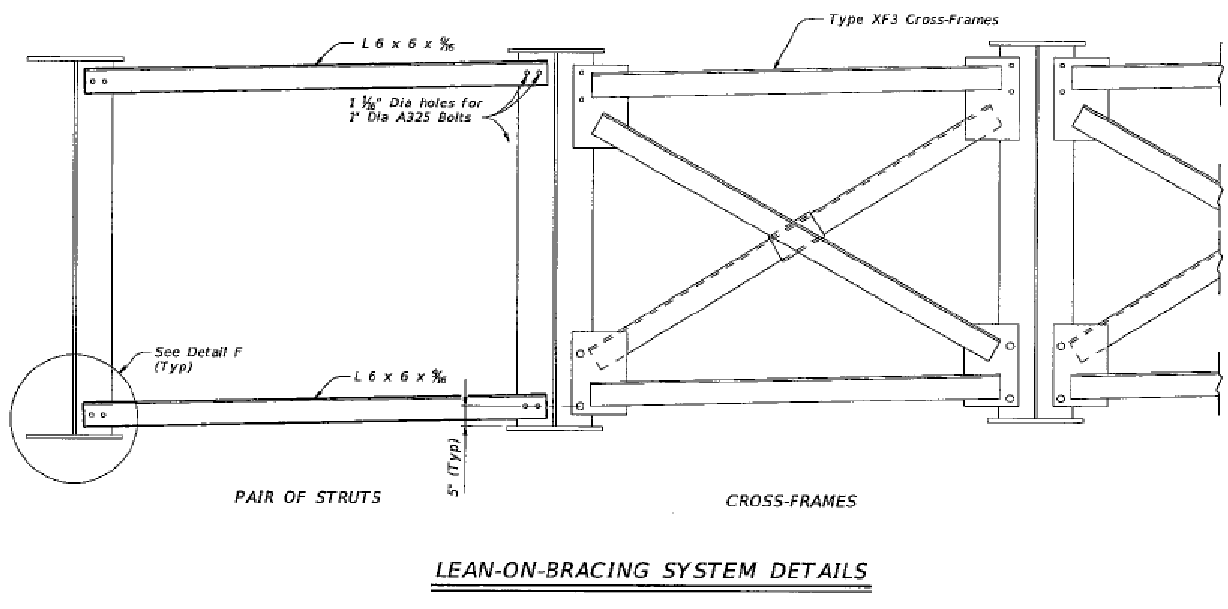

Figure 4-11, Lean-On Brace (TxDOT)

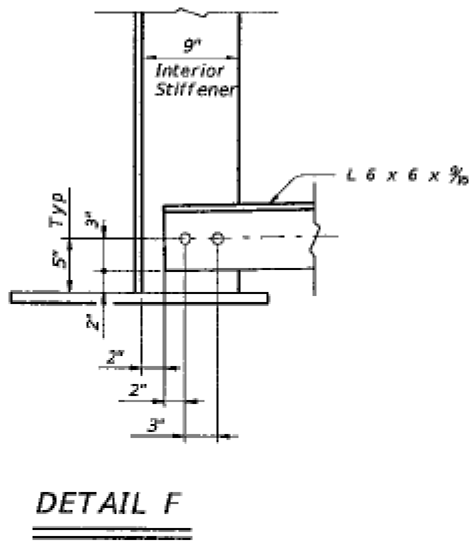

Figure 4-12, Lean-On Brace End Connection (TxDOT)

The framing plan (Figure 4-13), depicts interior cross-frames laid out in an S-pattern. Full lines of cross-frames are provided adjacent to air splices, to assist with geometry control during erection. There are a total of 40 interior braces in the end spans; 19 are cross-frames and 21 are lean-on braces. The interior span has 26 cross-frames and 26 lean-on braces. Again, cross-frames are provided in all bays at lines of support. For the entire unit, there are 148 total braces and 68 of these are lean-on braces. Figure 4-14 depicts combinations of cross-frames and lean-on braces at various points in each span.

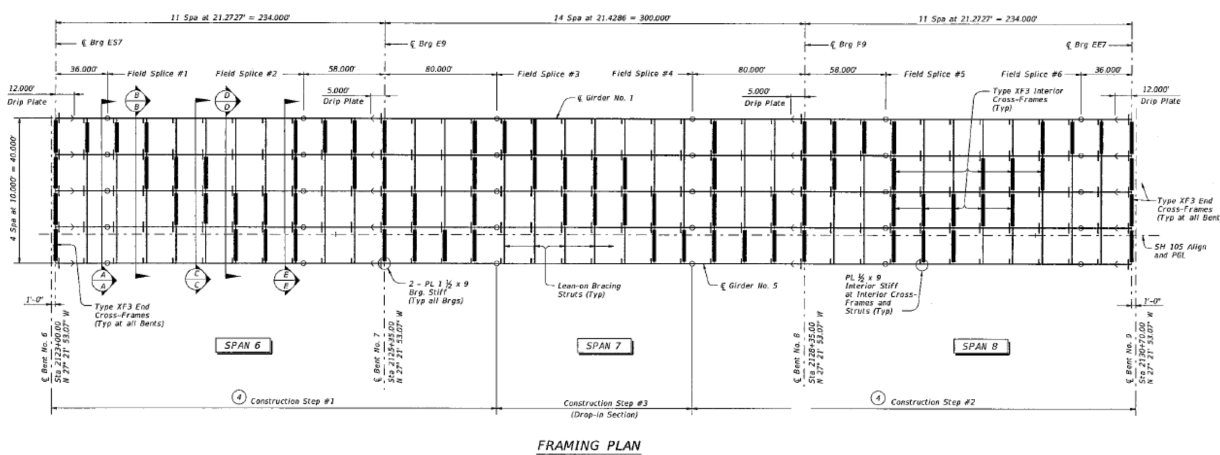

Figure 4-13, Framing Plan (TxDOT)

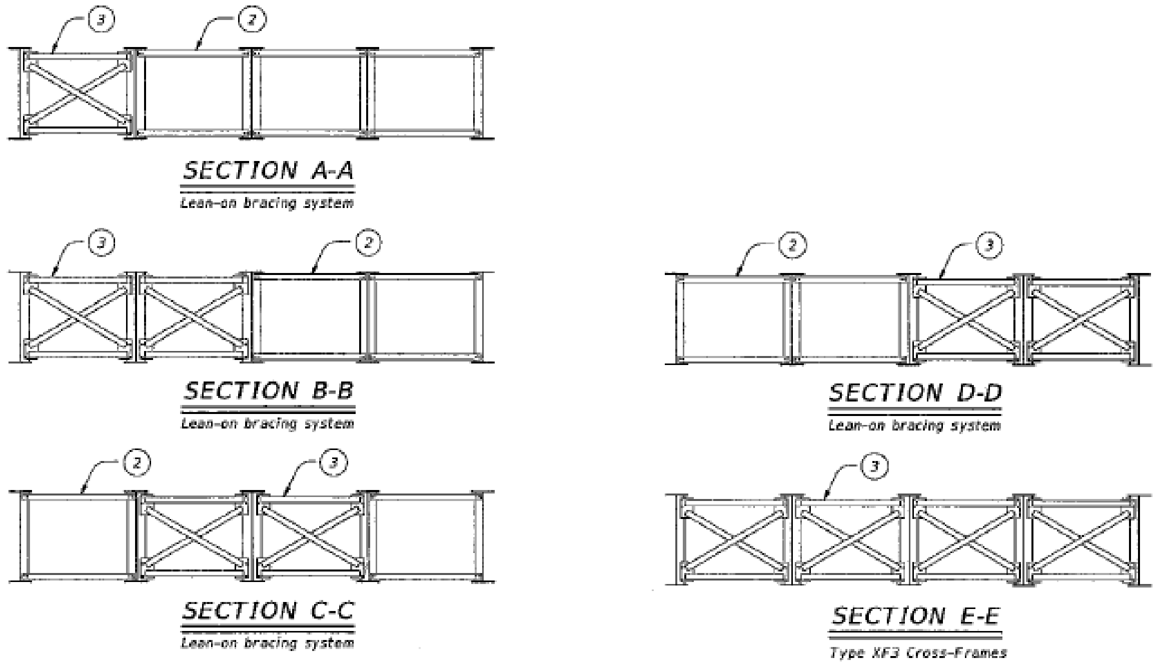

Figure 4-14, Cross-Frame and Lean-On Brace Combinations at Various Locations (TxDOT)

4.2.4 Erection

Girder erection notes were provided in the plans. These notes indicated an assumed erection procedure that had the end spans erected first, followed by pier sections, and the drop-in section last. The framing plan was developed assuming Girder 1 is placed first, followed by Girders 2 through 5, during all erection stages. The notes allowed the contractor to develop their own erection sequence and procedure but that the framing plan would need to be revised accordingly, prior to girder fabrication.

The contractor's erection procedures followed TxDOT's prescribed erection sequence. Shore towers were utilized in Spans 6 and 8 (end spans). For each span, the first girder was erected bare (no cross-frames pre-installed). Adjacent girders were each erected with cross-frames and lean-on braces installed prior to lifting as shown in Figure 4-15. Erection completion is shown in Figure 4-16, where lean-on brace locations relative to cross-frames are clearly seen.

Figure 4-15, Drop-In Girder Section with Cross-Frames and Lean-On Braces Attached (TxDOT)

Figure 4-16, Completion of Steel Erection (TxDOT)

4.2.5 Deck Placement

Deck placement was conventional, with end spans placed first (ending 69.5 ft from interior supports), followed by the interior span (ending at 73.75 ft from interior supports) and the final two sections placed over each interior support. Continuous placement of the deck concrete, end to end, was disallowed by plan note.

4.2.6 Conclusions

The SH 155 at Brazos River Bridge design was a successful and economical implementation of lean-on bracing for a non-skewed bridge. Success is attributable to leadership by the owner (TxDOT) and open communication between all parties involved in its fabrication and erection.

Chapter 5.
Design Example 1: Straight, Non-Skewed Bridge

5.1 EXAMPLE OVERVIEW

A design process for lean-on bracing is illustrated in this example. This example consists of a straight 3-span continuous steel I-girder bridge with spans of 140 ft – 175 ft – 140 ft. The overall bridge width is 43 ft, with a clear roadway width of 40 ft. This bridge has four girders spaced at 12 ft, and the deck overhangs are 3.5 ft. The bridge deck is 9.5 in. thick with an integral 0.5 in. wearing surface. The railings are conventional concrete safety shape barriers. The bridge design includes allowance for addition of a wearing surface of 25 psf. This example is taken from Grubb and Schmidt (2015), with revision for lean-on bracing.

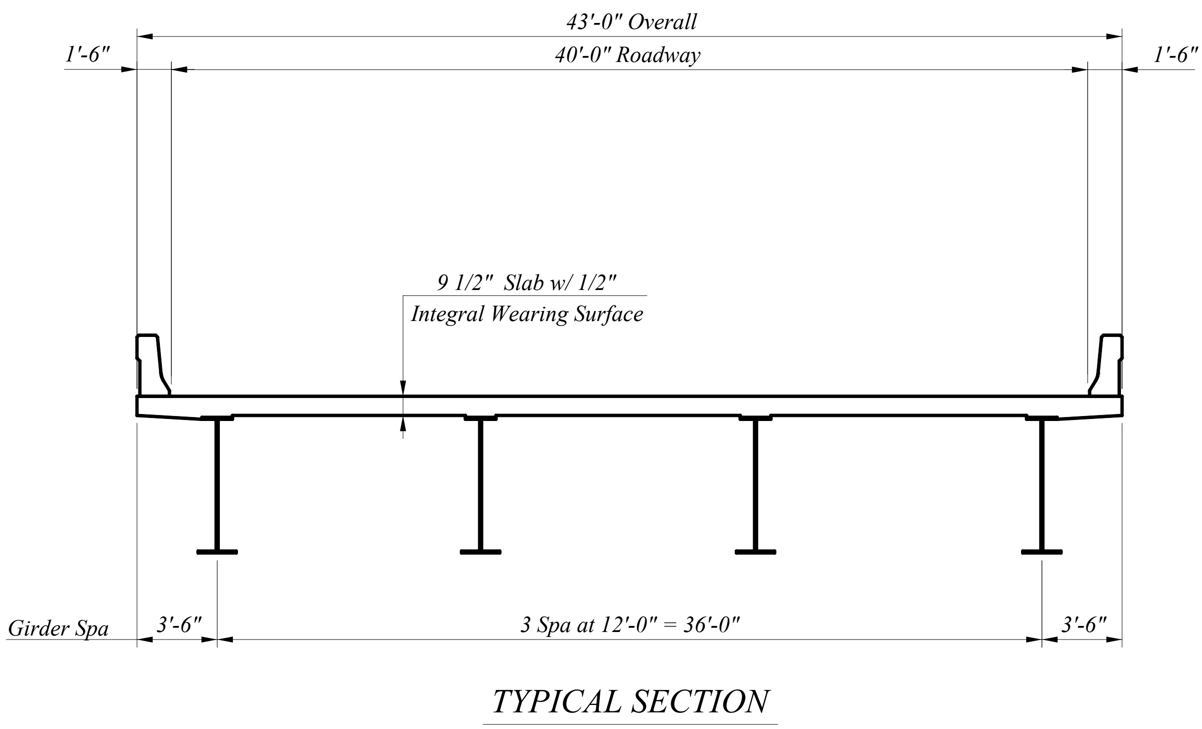

Figure 5-1, Example 1 Bridge, Typical Section

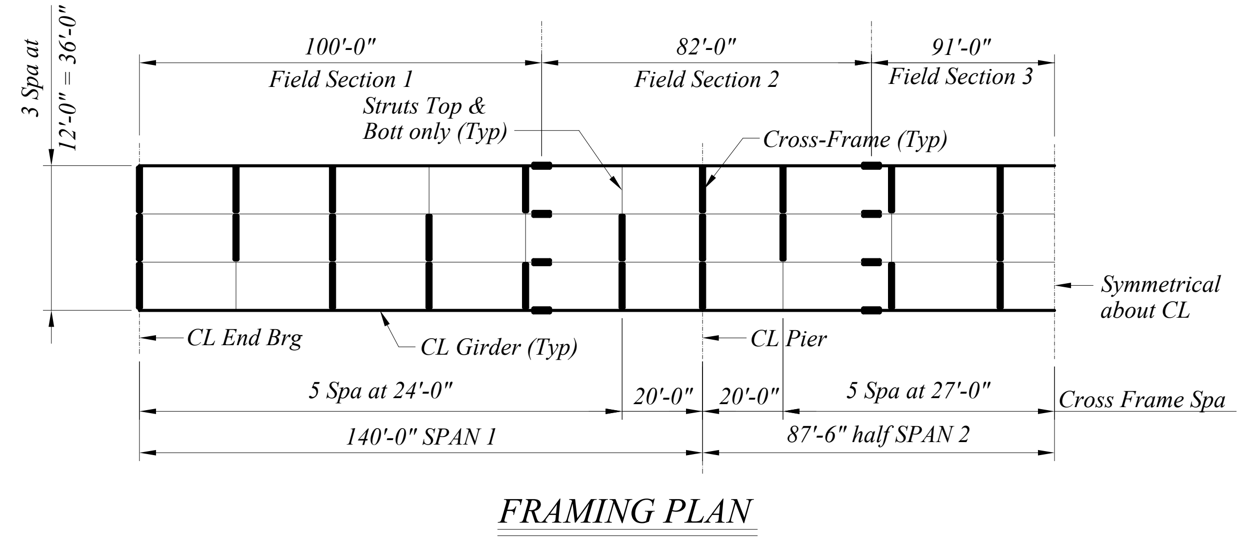

Figure 5-2, Example 1 Bridge, Framing Plan

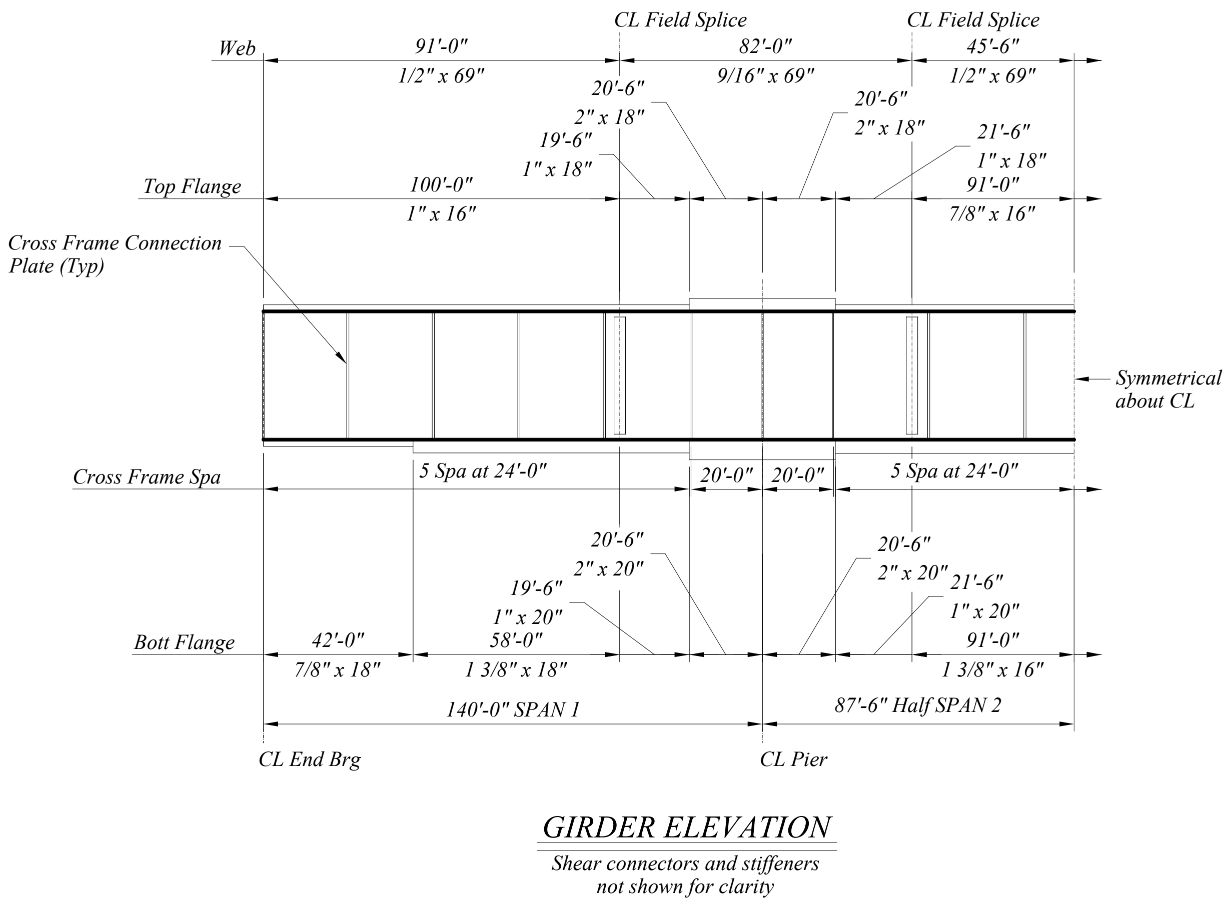

Figure 5-3, Example 1 Bridge, Girder Elevation

The design approach utilizes the stability bracing provisions in Article 6.7.4.2.2 as outlined in AASHTO LRFD 10th Edition (AASHTO 2024), with modification for lean-on bracing as outlined in Chapter 2.

This bridge has straight girders and supports are normal. AASHTO LRFD Commentary C6.7.4.1 notes live load force demands in bracing members for bridges with these geometric characteristics are small. Determination of live load forces through a refined analysis is unwarranted. As a result, addressing live load force effects in the cross-frames and lean-on braces for this bridge is unnecessary for satisfying the strength, fatigue and fracture, and service limit state provisions of AASHTO LRFD.

Calculations needed for lean-on bracing are shown for the positive moment region first (Section 5.2) followed by the negative moment region (Section 5.3).

5.2 LEAN-ON BRACING DESIGN, POSITIVE MOMENT REGION

In this example, the formulas used for lean-on braces are designed considering they are provided in the exterior bays which is a more conservative approach than using the formula for lean-on braces provided in interior bays. The bending moment values used in this example are for the exterior girders. The peak moments in the positive moment region are considered for design of the lean-on bracing system.

5.2.1 Input Data

Span under consideration	L	= Span 1 (140 ft)
Number of girders in cross-section	n_g	= 4
Girder Spacing	S	= 12 ft
Number of brace points in span	n	= 5
Unbraced length of the segment	L_b	= 24 ft (cross-frame spacing)

5.2.2 Cross-Section Properties

Item	Width, in.	Height, in.	y (from bottom of bottom flange), in.	Area, in.^{2}	A*y, in.^{3}	I_x, in.^{4}	I_y, in.^{4}	A*d^2
Bottom Flange	18.00	1.375	0.6875	24.75	17.02	3.90	668.3	23871
Web	0.50	69.00	35.875	34.5	1237.69	13688	0.7	589
Top Flange	16.00	1.000	70.875	16	1134.00	1.33	341.3	24500
	Sum	71.375		75	2389	13693	1010	48960

Web depth	D	= 69 in.
Web thickness	t_w	= 0.5 in.
Moment of Inertia (strong axis)	I_x	= 62653 in.^{4}
Moment of Inertia (weak axis)	I_y	= 1010 in.^{4}
Distance of centroid from bottom	d_{bottom}	= 31.74 in.
Distance of centroid from top	d_{top}	= 39.63 in.
Section modulus bottom	$S_{x\text{-}bot}$	= 1974 in.^{3}
Section modulus top	$S_{x\text{-}top}$	= 1581 in.^{3}

Distance from centroid of girder to centroid of compression flange

c = 39.13 in.

Distance from centroid of girder to centroid of tension flange

t = 31.06 in.

Distance between centroid of compression and tension flange

h_o = 39.63 + 31.06 = 70.69 in.

Moment of inertia of compression flange about vertical axis (I_y top flange)

I_{yc} = 341.3 in.^{4}

Moment of inertia of tension flange about vertical axis (I_y bottom flange)

I_{yt} = 668.3 in.^{4}

Effective Moment of Inertia of girder about vertical axis ($I_{y\text{-}eff}$ or I_{eff})

$$I_{eff} = I_{yc} + \left(\frac{t}{c}\right) I_{yt} = 341.4 + \left(\frac{31.06}{39.13}\right) \times 668.3 = 872\,\text{in.}^4$$

Steel modulus of elasticity E = 29000 ksi

5.2.3 Cross-Frame Properties

Connection Plate Properties:

Width of connection plate b_s = 6 in.

Thickness of connection plate t_s = 0.5 in.

Distance from top of cross-frame to bottom of top flange

h_i = 3.5 in.

Height of cross-frame h_b = $D - 2 \times h_i = 69 - 2 \times 3.5 = 62\,\text{in.}$

The connection plate is connected to top and bottom flanges of girder.

Diagonal Properties:

Length of Diagonal L_d = $\sqrt{\left((S \times 12) - 2 \times b_s\right)^2 + h_b^{\,2}} = 145\,\text{in.}$

Strut Properties:

Length of Strut L_s = $(S - 2 \times b_s) = 132\,\text{in.}$

5.2.4 Structural Loads

AASHTO LRFD Article 3.4.2.1 prescribes two sets of load factors for construction loads with steel structures at the strength limit state. The first set uses 1.25 (minimum) for DC and DW and 1.5 (minimum) for construction live load. The second set is specific to *primary* steel superstructure components applied to the *fully erected* steelwork [emphasis added]. For this specific case, a minimum load factor of 1.4 is applied to both DC and construction LL. With cross-frames being defined as secondary members per AASHTO LRFD Table 6.6.2.1-1 and not primary members, for the straight steel girders for which lean-on bracing may be used there is ambiguity in load factors for the design of stability bracing. The analysis for this example used a very conservative approach to construction LL and the first set of load factors is used to prevent an unwarranted level of conservatism.

A structural analysis was performed and the maximum positive moments are governed by the end spans. These moments are as follows:

Unfactored dead load moment M_{DL} = 26157 kip-in.

Unfactored construction live load moment M_{LL} = 14648 kip-in.

Construction dead load factor	γ_{DL}	= 1.25
Construction live load factor	γ_{LL}	= 1.50
Factored positive moment	M_u	= 54668 kip-in.
Moment gradient modifier	C_b	= 1.0

5.2.5 Bolt Details

Bolt diameter	d_b	= 1 in.
Bolt yield strength	F_{yb}	= 70 ksi
Bolt tensile strength	F_{ub}	= 120 ksi

5.2.6 Torsional Brace Stiffness Requirements (Positive Moment Case)

The relationship between the actual stiffness provided by lean-on bracing system and the required stiffness of the system can be determined by this equation:

$$(\beta_T)_{act} \geq (\beta_T)_{req}$$

Where:

$(\beta_T)_{act} =$ actual stiffness provided by LOB system

$(\beta_T)_{req} =$ required stiffness of LOB system

5.2.7 Determine Stiffness Provided with Lean-On Bracing $(\beta_T)_{act}$

The provided stiffness of a lean-on brace system for the bridge is a function of cross-frame or diaphragm stiffness, in-plane girder stiffness, and cross-sectional distortion stiffness. The following equation is used to determine the stiffness provided by lean-on brace system:

$$(\beta_T)_{act} = \frac{1}{\frac{1}{\beta_{br}} + \frac{1}{\beta_{\sec}} + \frac{1}{\beta_g}}$$

Where:

$\beta_{br} =$ brace stiffness of the diaphragm or cross-frame

$\beta_{\sec} =$ cross-sectional distortion stiffness

$\beta_g =$ effective in-plane girder stiffness

Each of the components needed to determine the provided stiffness of the bracing system are calculated in this section.

Brace Stiffness (Cross-Frame Stiffness)

Two equations are provided for cross-frame stiffness, one for exterior bays and one for interior bays. With a lean-on brace system, the normal equation for cross-frame stiffness must be modified. Use of the exterior bay equation for interior bays is conservative and used here. For X-type cross-frames with a tension-only diagonal system, the following equation can be used:

$$\beta_{br} = \frac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc} - 1\right)^2} A_b$$

Where:

$$n_{gc} = \frac{n_g}{N_c}$$

n_g = number of girders in bridge cross-section = 4

N_c = number of cross frames at each brace location = 2

$$n_{gc} = \frac{4}{2} = 2$$

Assuming a single angle section for both diagonals and struts, try L6×6×3/4 as brace member. Since a single angle is used the area of brace is reduced by a factor of 0.65

Area of brace	A_b	= 8.46 in.^{2}
Length of diagonal	L_d	= 145 in.
Girder spacing	S	= 12ft. x 12 in./ft. = 144 in.
Height of cross-frame	h_b	= 62 in.

Hence, the provided torsional stiffness is calculated as below:

$$\beta_{br} = \frac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc} - 1\right)^2} \left(A_b \times 0.65\right)$$

$$= \frac{29000 \times 144^2 \times 62^2}{2 \times 145^3 + 144^3 \left(2-1\right)^2} 8.46 \times 0.65$$

$$\boldsymbol{\beta_{br}} = \mathbf{1399425\ kip\text{-}in/rad}$$

Cross Sectional Stiffness

In this example the girder is a built-up I-section with cross-frames.

Height of cross-frame	h_b	= 62 in.
Web depth	D	= 69 in.
80 percent of web depth	0.8 x 69 = 55.2 in.	

Since the height of the cross-frame is more than 80 percent of the web depth, cross-sectional distortion stiffness can be assumed to be infinity.

$$\beta_{sec} = \infty$$

$$\boldsymbol{\beta_{sec}} = \mathbf{10 \times 10^{99}\ \ kip\text{-}in./rad}$$

In-plane Girder Stiffness

$$\beta_g = \frac{12\left(n_g - 1\right)^2 S^2 E\, I_x}{n_g L^3}$$

$$= \frac{12 \times \left(4-1\right)^2 \times \left(12 \times 12\right)^2 \times 29000 \times 62653}{4 \times \left(140 \times 12\right)^3}$$

$$\boldsymbol{\beta_g} = \mathbf{214536\ kip\text{-}in./rad}$$

Hence the actual stiffness provided by the lean-on brace system is:

$$\left(\beta_T\right)_{act} = \frac{1}{\dfrac{1}{\beta_{br}} + \dfrac{1}{\beta_{\sec}} + \dfrac{1}{\beta_g}}$$

$$= \frac{1}{\frac{1}{1399425} + \frac{1}{10\times10^{99}} + \frac{1}{214536}}$$

$$(\beta_T)_{act} \quad \mathbf{= 186019\ kip\text{-}in./rad}$$

5.2.8 Determine Required Stiffness of Torsional Brace System (β_T)$_{req}$

The following equation is used to determine the required torsional brace stiffness for the positive moment region:

$$(\beta_T)_{req} = \frac{3.6\,L}{\phi_{sb} C_b^2\, n\, I_{eff}\, E} M_u^2$$

Where:

n	= number of intermediate cross-frames lines in first span	= 5
ϕ_{sb}	= resistance factor for stability bracing	= 0.80

Therefore,

$$(\beta_T)_{req} = \frac{3.6\times140\times12}{0.8\times1.0^2\times5\times872\times29000} 54668^2$$

$$(\beta_T)_{req} \quad \mathbf{= 178691\ kip\text{-}in./rad}$$

5.2.9 Check Stiffness Requirement of the System

$$(\beta_T)_{act} \geq (\beta_T)_{req}$$

Actual provided stiffness	$(\beta_T)_{act}$	= 186019 kip-in. /rad
Required stiffness	$(\beta_T)_{req}$	= 178691 kip-in./rad

The provided actual stiffness using a single angle section (L6×6×3/4) for a tension-only diagonal X-type cross-frame in a lean-on brace system is greater than the required stiffness. Hence the angle used satisfies the stiffness requirement of a lean-on brace system. Use single angle L6×6×3/4 for diagonal and strut members.

If the provided stiffness was insufficient, the designer can reduce the number of girders per cross-frame (n_{gc}), increase the cross-frame member size (A_b), and/or increase in-plane girder stiffness (β_g).

5.2.10 Check Strength Requirements of Lean-On Brace System (Positive Moment Case)

Along with satisfying the stiffness requirements, members of the cross-frame in a lean-on brace system should also satisfy the following strength requirements:

The required strength of a torsional brace can be calculated as follows:

$$M_{br} = \frac{0.008\,L\,L_b}{n\,E\,I_{yeff}\,h_o}\left(\frac{M_u}{C_b}\right)^2$$

$$= \frac{0.008\times1680\times288}{7\times29000\times872\times70.69}\left(\frac{54668}{1}\right)^2$$

$$M_{br} \quad \mathbf{= 925\ kip\text{-}in.}$$

5.2.11 Calculate Applied Forces in the Cross-Frame Members

Force in brace:

$$F_{br} = \frac{M_{br}}{h_b}$$

$$= \frac{925}{62}$$

$$\boldsymbol{F_{br}} \mathbf{= 15.0\ kip}$$

Force in diagonal in tension-only diagonal X-type cross-frame is:

$$F_d = \frac{n_{gc} F_{br}\ L_d}{S}$$

$$= \frac{2\times 15.0\times 145}{144}$$

$$\boldsymbol{F_d} \mathbf{= 31\ kip}$$

Force in strut when cross-frame is located in exterior bay:

$$F_s = \left(n_{gc} - 1\right) F_{br}$$

$$= \left(2-1\right)15.0$$

$$\boldsymbol{F_s} \mathbf{= 15.0\ kip}$$

5.2.12 Determine Tensile Capacity of Cross-Frame Diagonal Members

Assume two 1-in. diameter bolts at each end of each diagonal and strut angles to fasten it to connection plate. Use a single angle section L6×6×3/4 for the diagonals and struts. For diagonals, only tension checks are calculated since a tension-only diagonal X-type cross-frame is considered. A compression check is calculated for horizontal struts as they may be in either tension or compression, with compression governing member capacity.

Gross area of angle A_g = 8.46 in.^{2}

Yield strength F_y = 50 ksi

Tensile strength F_u = 70 ksi

Bolt spacing L = 3 in.

Distance of centroid in x direction $\bar{x}$ = 1.77 in.

Shear lag reduction factor $U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.77}{3}$

$= 0.41$

Reduction factor for holes R_p = 1.0 (holes drilled to full size)

Net area of angle $A_n = A_g - 1\times\left(d_b + \frac{1}{8}\right)\times t_{leg}$

$$= 8.46 - 1\times\left(1 + \frac{1}{8}\right)\times\frac{3}{4}$$

A_n = 7.62 in.^{2}

Determine Gross Section Yielding Capacity in Tension:

Resistance factor for yielding ϕ_y = 0.95

Therefore,

Gross section yielding capacity $\phi_y P_{ny} = \phi_y F_y A_g$

$= 0.95 \times 50 \times 8.46$

$\boldsymbol{\phi_y P_{ny}}$ **= 401.8 kip**

Determine Net Section Fracture Capacity in Tension:

Resistance factor for fracture $\phi_u = 0.8$

Therefore,

Net section fracture capacity $\phi_u P_{nu} = \phi_u F_u A_n R_p U$

$= 0.8 \times 70 \times 7.62 \times 1.0 \times 0.41$

$\boldsymbol{\phi_u P_{nu}}$ **= 174.9 kip**

The factored tensile capacity of the angle is taken as the minimum of gross section yielding capacity and net section fracture capacity.

Factored Tensile Capacity $\boldsymbol{P_r}$ **=** $\boldsymbol{\phi_u P_{nu}}$ **= 174.9 kip >** $\boldsymbol{F_d}$ **= 31 kip** **(OK)**

5.2.13 Determine Compressive Capacity of Cross-Frame and Lean-On Brace Struts

AASHTO LRFD Article 6.9.4.4 addresses the compressive capacity of single-angle members. The capacity of the proposed L6×6×3/4 section for the horizontal cross-frame struts and lean-on brace struts is verified at this step.

As per AASHTO LRFD Article 6.9.4.4:

$$\frac{l}{r_x} = 72.53 < 80$$

Hence, effective slenderness ratio is calculated as:

AASHTO LRFD Eqn. 6.9.4.4-1, $\left(\frac{Kl}{r_x}\right)_{eff} = 72 + 0.75\frac{l}{r_x}$

$= 72 + 0.75 \times 72.53$

$$\left(\frac{Kl}{r_x}\right)_{eff} = 126.4$$

Elastic critical buckling resistance of single angle is calculated as:

AASHTO LRFD Eqn. 6.9.4.1.2-1, $P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_x}\right)_{eff}^2} A_g$

$$= \frac{\pi^2 \times 29000}{126.4^2} \times 8.46$$

$\boldsymbol{P_e}$ **= 151.6 kip**

Nominal yield resistance:

AASHTO LRFD Article 6.9.4.1, P_o $= F_y A_g$

$= 50 \times 8.46$

$\boldsymbol{P_o}$ **= 423 kip**

As per AASHTO LRFD Article 6.9.4.1:

$$\frac{P_o}{P_e} = 2.79 > 2.25$$

Hence, Nominal compressive resistance of single angle is:

AASHTO LRFD Eqn. 6.9.4.1-2 P_n $= 0.877 P_e$

$= 0.877 \times 151.6$

$\boldsymbol{P_n}$ **= 132.9 kip**

Factored compressive resistance of single angle is:

ϕ_c = 0.95

$\boldsymbol{\phi_c P_n}$ **= 126.3 kip >** $\boldsymbol{F_s}$ **= 15.0 kip** **(OK)**

5.2.14 Determine Bolt Capacity

Area of bolt A_b $= \frac{\pi}{4} d_b^2 = \frac{\pi}{4} 1^2 = 0.79$ in.^{2}

Resistance factor for bolts in shear ϕ_s = 0.8

Number of shear plane N_s = 1

Nominal shear resistance of bolts R_n $= 0.45 A_b F_{ub} N_s$ (threads included in shear plane)

$= 0.45 \times 0.79 \times 120 \times 1 \times 2$ (two bolts)

R_n = 85.3 kip

Factored shear resistance of bolts

$\boldsymbol{\phi_s R_n}$ **= 68.2 kip >** $\boldsymbol{F_d}$ **= 31 kip** **(OK)**

5.2.15 Determine Bearing Capacity of Bolt Holes

Thickness of bolt bearing element $t_{bearing}$ = min (connection plate thickness, angle leg thickness)

= min (0.5 in., 0.75 in.)

$t_{bearing}$ = 0.5 in.

As per AASHTO LRFD Article 6.13.2.9:

Spacing between bolts = 3 in. > 2.0d

End distance = 1.5 in. < 2.0d

Hence using AASHTO LRFD Eqn. 6.13.2.9-2:

Bearing capacity of bolt hole R_n $= 1.2\,d\,t_{bearing} F_u$

$= 1.2 \times 1 \times 0.5 \times 70 \times 2$ (two bolt holes)

R_n = 84 kip

Resistance factor for bolt bearing ϕ_b = 0.8

Factored bearing resistance of bolt holes

$\boldsymbol{\phi_b R_n}$ **= 67.2 kip >** $\boldsymbol{F_d}$ **= 60.4 kip** **(OK)**

5.2.16 Limiting Slenderness Ratio and Compression Capacity Check

As per AASHTO LRFD Article 6.9.3, the compression slenderness ratio of compression members or tension members subjected to stress reversal for secondary members is limited to 140.

$$\frac{Kl}{r} < 140$$

Slenderness ratio of L6×6×3/4 is calculated as follows:

K = 1.0 (For single angles)

L_s $= \left(S - 2 \times b_s\right) = 132\,\text{in.}$

r_x = 1.82 in.

$$\frac{Kl}{r_x} = \frac{1.0 \times 132}{1.82} = 72.53 < 140$$

Angle L6×6×3/4 satisfies the limiting slenderness ratio criteria.

5.3 LEAN-ON BRACING DESIGN, NEGATIVE MOMENT REGION

In this example, the formulas used for lean-on braces are designed considering they are provided in the exterior bay which is a more conservative approach than using the formulas for lean-on braces provided in interior bays. The bending moment values used in this example are for the exterior girders. The peak moments in the negative moment region are considered for design of the lean-on bracing system.

5.3.1 Input Data

Span under consideration	L	= Average of Span 1 and Span 2 (157.5 ft)
Number of girders in cross-section	n_g	= 4
Girder Spacing	S	= 12 ft
Number of brace points in span	n	= 5
Unbraced length of the segment	L_b	= 20 ft (cross-frame spacing)

5.3.2 Cross-Section Properties

Item	Width, in.	Height, in.	y (from bottom of bottom flange), in.	Area, in.^{2}	A*y, in.^{3}	I_x, in.^{4}	I_y, in.^{4}	A*d^2
Bottom Flange	20.00	2.000	1	40	40.00	13.33	1333.3	46959
Web	0.5625	69.00	36.5	38.81	1416.66	15399	1.0	59
Top Flange	18.00	2.000	72	36	2592.00	12.00	972.0	48585
	Sum	73		115	4049	15424	2306	95603

Web depth	D	= 69 in.
Web thickness	t_w	= 0.5 in.
Moment of Inertia (strong axis)	I_x	= 111028 in.^{4}
Moment of Inertia (weak axis)	I_y	= 2306 in.^{4}
Distance of centroid from bottom	d_{bottom}	= 35.26 in.
Distance of centroid from top	d_{top}	= 37.74 in.
Section modulus bottom	$S_{x\text{-}bot}$	= 3149 in.^{3}
Section modulus top	$S_{x\text{-}top}$	= 2942 in.^{3}

Distance from centroid of girder to centroid of compression flange

c = 34.26 in.

Distance from centroid of girder to centroid of tension flange

t = 36.74 in.

Distance between centroid of compression and tension flange

h_o = 34.26 + 36.74 = 71 in.

Moment of inertia of compression flange about vertical axis (I_y bottom flange)

I_{yc} = 1333.3 in.^{4}

Moment of inertia of tension flange about vertical axis (I_y top flange)

I_{yt} = 972 in.^{4}

Effective Moment of Inertia of girder about vertical axis ($I_{y\text{-}eff}$ or I_{eff})

$$I_{eff} = I_{yc} + \left(\frac{t}{c}\right) I_{yt} = 1333.3 + \left(\frac{36.74}{34.26}\right) \times 972 = 2375.5\,\text{in.}^4$$

Steel modulus of elasticity E = 29000 ksi

5.3.3 Cross-Frame Properties

Connection Plate Properties:

Width of connection plate b_s = 6 in.

Thickness of connection plate t_s = 0.5 in.

Distance from top of cross- frame to bottom of top flange

h_i = 3.5 in.

Height of cross-frame h_b = $D - 2\times h_i = 69 - 2\times 3.5 = 62\,\text{in.}$

The connection plate is connected to top and bottom flange of girder.

Diagonal Properties:

Length of Diagonal L_d = $\sqrt{\left((S\times 12) - 2\times b_s\right)^2 + {h_b}^2} = 145\,\text{in.}$

Strut Properties:

Length of Strut L_s = $\left(S - 2\times b_s\right) = 132\,\text{in.}$

5.3.4 Structural Loads

The discussion on load factors in section 5.2.4 applies here. A structural analysis was performed and the maximum negative moments obtained at the interior support are as follows:

Unfactored dead load moment M_{DL} = 58080 kip-in.

Unfactored construction live load moment M_{LL} = 23652 kip-in.

Construction dead load factor γ_{DL} = 1.25

Construction live load factor γ_{LL} = 1.50

Factored negative moment M_u = 108078 kip-in.

Moment gradient modifier C_b = 1.25

5.3.5 Bolt Details

Bolt diameter d_b = 1 in.

Bolt yield strength F_{yb} = 70 ksi

Bolt tensile strength F_{ub} = 120 ksi

5.3.6 Torsional Brace Stiffness Requirements (Negative Moment Case)

The relationship between the actual stiffness provided by lean-on bracing system and the required stiffness of the system can be determined by this equation:

$$(\beta_T)_{act} \geq (\beta_T)_{req}$$

Where:

$(\beta_T)_{act}$ = actual stiffness provided by LOB system

$(\beta_T)_{req}$ = required stiffness of LOB system

5.3.7 Determine Actual Stiffness Provided with Lean-On Brace $(\beta_T)_{act}$

The actual provided stiffness of a lean-on brace system of the bridge is calculated in this section. The actual provided stiffness is a function of cross-frame or diaphragm stiffness, in-plane girder stiffness and cross-sectional distortion stiffness. The following equation is used to determine the actual stiffness provided by a lean-on brace system:

$$(\beta_T)_{act} = \frac{1}{\dfrac{1}{\beta_{br}} + \dfrac{1}{\beta_{\sec}} + \dfrac{1}{\beta_g}}$$

Where:

β_{br} = brace stiffness of the diaphragm or cross-frame

$\beta_{\sec}$ = cross-sectional distortion stiffness

β_g = effective in-plane girder stiffness

Each of the components needed to determine the actual provided stiffness of the bracing system are calculated in this section.

Brace Stiffness (Cross-Frame Stiffness)

Two equations are provided for cross-frame stiffness, one for exterior bays and one for interior bays. With a lean-on brace system, the normal equation for cross-frame stiffness must be modified. Use of the exterior bay equation for interior bays is conservative and used here. For X-type cross-frames with a tension-only diagonal system, the following equation can be used:

$$\beta_{br} = \frac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc} - 1\right)^2} A_b$$

Where:

$$n_{gc} = \frac{n_g}{N_c}$$

n_g = number of girders in bridge cross-section = 4

N_c = number of cross-frames at each brace location = 2

$$n_{gc} = \frac{4}{2} = 2$$

Assuming single angle section for both diagonal and strut, using L6x6x3/8 as brace member. Since a single angle is used, the area of brace is reduced by a factor of 0.65

Area of brace	A_b	= 8.46 in.^{2}
Length of diagonal	L_d	= 145 in.
Girder spacing	S	= 12ft. x 12 in./ft. = 144 in.

Height of cross-frame h_b = 62 in.

Hence, the provided torsional stiffness is calculated as below:

$$\beta_{br} = \frac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc} - 1\right)^2} A_b \times 0.65$$

$$= \frac{29000 \times 144^2 \times 62^2}{2 \times 145^3 + 144^3 \left(2-1\right)^2} 8.46 \times 0.65$$

$$\boldsymbol{\beta_{br}} = \textbf{1399425 kip-in./rad}$$

Cross-Sectional Stiffness

In this example the girder is a built-up I-section with cross-frames.

Height of cross-frame h_b = 62 in.

Web depth D = 69 in.

80 percent of web depth 0.8 × 69 = 55.2 in.

Since the height of the cross-frame provided is more than 80 percent of web depth, cross-sectional distortion stiffness can be assumed to be infinity.

$$\beta_{sec} = \infty$$

$$\boldsymbol{\beta_{sec}} = \mathbf{10 \times 10^{99}} \textbf{ kip-in./rad}$$

In-Plane Girder Stiffness

$$\beta_g = \frac{12\left(n_g - 1\right)^2 S^2 E I_x}{n_g L^3}$$

$$= \frac{12 \times \left(4-1\right)^2 \times \left(12 \times 12\right)^2 \times 29000 \times 111028}{4 \times \left(157.5 \times 12\right)^3}$$

$$\boldsymbol{\beta_g} = \textbf{267013 kip-in./rad}$$

Hence the actual stiffness provided by the lean-on brace system is:

$$\left(\beta_T\right)_{act} = \frac{1}{\dfrac{1}{\beta_{br}} + \dfrac{1}{\beta_{\sec}} + \dfrac{1}{\beta_g}}$$

$$= \frac{1}{\dfrac{1}{1399425} + \dfrac{1}{10 \times 10^{99}} + \dfrac{1}{267013}}$$

$$\left(\boldsymbol{\beta_T}\right)_{\boldsymbol{act}} = \textbf{224230 kip-in./rad}$$

5.3.8 Determine Required Stiffness of Torsional Brace System $(\boldsymbol{\beta_T})_{req}$

The following equation is used to determine the required torsional brace stiffness of the negative moment region:

$$(\beta_T)_{req} = \frac{3.6 L}{\phi_{sb} C_b^2 \, n \, I_{eff} E} M_u^2$$

Where:

n = number of intermediate cross-frames lines in first span = 5

ϕ_{sb} = resistance factor for stability bracing = 0.80

Therefore, $(\beta_T)_{req} = \dfrac{3.6 \times 157.5 \times 12}{0.8 \times 1.25^2 \times 5 \times 2375.5 \times 29000} 108078^2$

$(\beta_T)_{req}$ **= 184587 kip-in/rad**

5.3.9 Check Stiffness Requirement of the System

$$\left(\beta_T\right)_{act} \geq \left(\beta_T\right)_{req}$$

Actual provided stiffness $(\beta_T)_{act}$ = 224230 kip-in. /rad

Required stiffness $(\beta_T)_{req}$ = 184587 kip-in./rad

The provided actual stiffness using a single angle section (L6×6×3/4) for a tension-only diagonal X-type cross-frame in a lean-on brace system is greater than the required stiffness. Hence the angle used satisfies the stiffness requirement of lean-on brace system. Use single angle L6×6×3/4 for diagonal and strut members.

If the provided stiffness was insufficient, the designer can reduce the number of girders per cross-frame (n_{gc}), increase the cross-frame member size (A_b), and/or increase in-plane girder stiffness (β_g).

5.3.10 Check Strength Requirements of Lean-On Brace System (Negative Moment Case)

Along with satisfying the stiffness requirements, members of the cross-frame in a lean-on brace system should also satisfy the following strength requirements:

The required strength of a torsional brace can be calculated as follows:

$$M_{br} = \frac{0.008\, L\, L_b}{n\, E\, I_{yeff}\, h_o}\left(\frac{M_u}{C_b}\right)^2$$

$$= \frac{0.008 \times 1890 \times 240}{7 \times 29000 \times 2375.5 \times 71}\left(\frac{108078}{1.25}\right)^2$$

M_{br} **= 792.3 kip-in.**

5.3.11 Calculate Applied Forces in the Cross-Frame Members

Force in Brace:

$$F_{br} = \frac{M_{br}}{h_b}$$

$$= \frac{792.3}{62}$$

F_{br} **= 12.8 kip**

Force in diagonal in tension-only X-frame is:

$$F_d = \frac{n_{gc} F_{br} L_d}{S}$$

$$= \frac{2 \times 12.8 \times 145}{144}$$

F_d **= 26 kip**

Force in strut when cross-frame is located in exterior bay:

$F_s = (n_{gc} - 1)F_{br}$

$= (2-1)12.8$

$\boldsymbol{F_s}$ **= 12.8 kip**

5.3.12 Determine Tensile Capacity of Cross-Frame Diagonal Members

Assuming two 1-in. diameter bolts at each end of each diagonal and strut angle to fasten it to connection plate. Use a single angle section L6×6×3/4 for both diagonal and strut. For diagonals, only tension checks are calculated since a tension-only diagonal X-type cross-frame is considered. A compression check is calculated for horizontal struts as they may be in either tension or compression, with compression governing member capacity.

Gross area of angle A_g = 8.46 in.^{2}

Yield strength F_y = 50 ksi

Tensile strength F_u = 70 ksi

Bolt spacing L = 3 in.

Distance of centroid in x direction $\bar{x}$ = 1.77 in.

Shear lag reduction factor $U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.77}{3}$

$= 0.41$

Reduction factor for holes R_p = 1.0 (holes drilled to full size)

Net area of angle $A_n = A_g - 1\times\left(d_b + \frac{1}{8}\right)\times t_{leg}$

$= 8.46 - 1\times\left(1+\frac{1}{8}\right)\times\frac{3}{4}$

A_n = 7.62 in.^{2}

Determine Gross Section Yielding Capacity in Tension:

Resistance factor for yielding ϕ_y = 0.95

Therefore,

Gross section yielding capacity $\phi_y P_{ny} = \phi_y F_y A_g$

$= 0.95\times50\times8.46$

$\boldsymbol{\phi_y P_{ny}}$ **= 401.8 kip**

Determine Net Section Fracture Capacity in Tension:

Resistance factor for fracture ϕ_u = 0.8

Therefore,

Net section fracture capacity $\phi_u P_{nu} = \phi_u F_u A_n R_p U$

$= 0.8\times70\times7.62\times1.0\times0.41$

$\boldsymbol{\phi_u P_{nu}}$ **= 174.9 kip**

The factored tensile capacity of the angle is taken as the minimum of gross section yielding capacity and net section fracture capacity.

Factored Tensile Capacity $\quad \boldsymbol{P_r} = \boldsymbol{\phi_u P_{nu}}$ **= 174.9 kip >** $\boldsymbol{F_d}$ **= 26 kip** **(OK)**

5.3.13 Determine Compressive Capacity of Cross-Frame and Lean-On Brace Struts

AASHTO LRFD Article 6.9.4.4 addresses the compressive capacity of single-angle members. The capacity of the proposed L6×6×3/4 section for the horizontal cross-frame struts and lean-on brace struts is verified at this step.

As per AASHTO LRFD, Article 6.9.4.4:

$$\frac{l}{r_x} = 72.53 < 80$$

Hence, effective slenderness ratio is calculated as:

AASHTO LRFD Eqn. 6.9.4.4-1,

$$\left(\frac{Kl}{r_x}\right)_{eff} = 72 + 0.75\frac{l}{r_x}$$

$$= 72 + 0.75 \times 72.53$$

$$\left(\frac{Kl}{r_x}\right)_{eff} = 126.4$$

Elastic critical buckling resistance of single angle is calculated as:

AASHTO LRFD Eqn. 6.9.4.1.2-1

$$P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_x}\right)_{eff}^2} A_g$$

$$= \frac{\pi^2 \times 29000}{126.4^2} \times 8.46$$

$$\boldsymbol{P_e = 151.6\ \text{kip}}$$

Nominal yield resistance:

AASHTO LRFD Article 6.9.4.1,

$$P_o = F_y A_g$$

$$= 50 \times 8.46$$

$$\boldsymbol{P_o = 423\ \text{kip}}$$

As per AASHTO LRFD Article 6.9.4.1:

$$\frac{P_o}{P_e} = 2.79 > 2.25$$

Hence, Nominal compressive resistance of single angle is:

AASHTO LRFD Eqn. 6.9.4.1-2

$$P_n = 0.877\,P_e$$

$$= 0.877 \times 151.6$$

$$\boldsymbol{P_n = 132.9\ \text{kip}}$$

Factored compressive resistance of single angle is:

ϕ_c = 0.95

$\boldsymbol{\phi_c P_n}$ **= 126.3 kip > $\boldsymbol{F_s}$ = 12.8 kip** **(OK)**

5.3.14 Determine Bolt Capacity

Area of bolt $A_b = \frac{\pi}{4} d_b^2 = \frac{\pi}{4} 1^2 = 0.79 \text{ in.}^2$

Resistance factor for bolts in shear $\phi_s = 0.8$

Number of shear planes $N_s = 1$

Nominal shear resistance of bolts $R_n = 0.45 A_b F_{ub} N_s$ (threads included in shear plane)

$= 0.45 \times 0.79 \times 120 \times 1 \times 2$ (two bolts)

R_n = 85.3 kip

Factored shear resistance of bolts

$\boldsymbol{\phi_s R_n}$ **= 68.2 kip > $\boldsymbol{F_d}$ = 26 kip** **(OK)**

5.3.15 Determine Bearing Capacity of Bolt Holes

Thickness of bolt bearing element $t_{bearing}$ = min (connection plate thickness, angle leg thickness)

= min (0.5 in., 0.75 in.)

$t_{bearing}$ = 0.5 in.

As per AASHTO LRFD Article 6.13.2.9:

Spacing between bolts = 3 in. > $2.0d$

End distance = 1.5 in. < $2.0d$

Hence using Eq 6.13.2.9-2

Bearing capacity of bolt hole $R_n = 1.2\, d\, t_{bearing} F_u$

$= 1.2 \times 1 \times 0.5 \times 70 \times 2$ (two bolt holes)

R_n = 84 kip

Resistance factor for bolt bearing $\phi_b = 0.8$

Factored bearing resistance of bolt holes

$\boldsymbol{\phi_b R_n}$ **= 67.2 kip > $\boldsymbol{F_d}$ = 52 kip** **(OK)**

5.3.16 Limiting Slenderness Ratio Check

As per AASHTO LRFD Article 6.9.3, the compression slenderness ratio of compression members or tension members subjected to stress reversal for secondary members is limited to 140.

$$\frac{Kl}{r} < 140$$

Slenderness ratio of L6×6×3/4 is calculated as follows:

K = 1.0 (For single angles)

L_s = $\left(S - 2\times b_s\right) = 132$ in.

r_x = 1.82 in.

$$\frac{Kl}{r_x} = \frac{1.0\times 132}{1.82} = 72.53 < 140$$

Angle L6×6×3/4 satisfies the limiting slenderness ratio criteria.

5.4 SUMMARY

To complete the design, the cross-frame members and lean-on brace struts should be checked for wind loads.

All cross-frame bracing members are L6×6×3/4, ASTM A709 Gr 50 Steel and are end-welded to gusset plates, which are attached to girder connection plates with 1-in. ASTM F3125 Grade A325 bolts. Lean-on brace struts use the same members and are directly bolted to girder connection plates.

For the designed positive bending region, a n_{gc} value of 2 was used but a full line of cross-frames is provided at the peak moment location at the designer's discretion. Additional cross-frames were added adjacent to field splices to assist with geometry control. Due to the small scale of this example bridge, the ability of lean-on bracing to make a significant reduction in cross-frames is limited.

Chapter 6.
Design Example 2: Straight, Skewed Bridge

6.1 EXAMPLE OVERVIEW

The application of Lean-on bracing system is illustrated in this example. This example consists of a skewed 3-span continuous steel I-girder bridge with spans of 85.0 ft – 125.0 ft – 85.0 ft. The bridge has a 20 deg right forward skew. The components of the Lean-on bracing system are designed and illustrated in this example.

The design approach utilizes the stability bracing provisions in Article 6.7.4.2.2 as outlined in AASHTO LRFD 10th Edition, with modification for lean-on bracing as outlined in Chapter 4.

This bridge has straight girders and supports are skewed, with a skew index, I_s, of 0.12. AASHTO LRFD Commentary C6.7.4.1 notes live load force demands in bracing members for bridges with straight girders and $I_s < 0.3$ are small and determination of live load forces through a refined analysis is unwarranted. As a result, addressing live load force effects in the cross- frames and lean-on braces for this bridge is unnecessary for satisfying the strength, fatigue and fracture, and service limit state provisions of AASHTO LRFD.

Calculations needed for a lean-on brace system are shown for the positive moment region first (Section 6.2) followed by the negative moment region (Section 6.3).

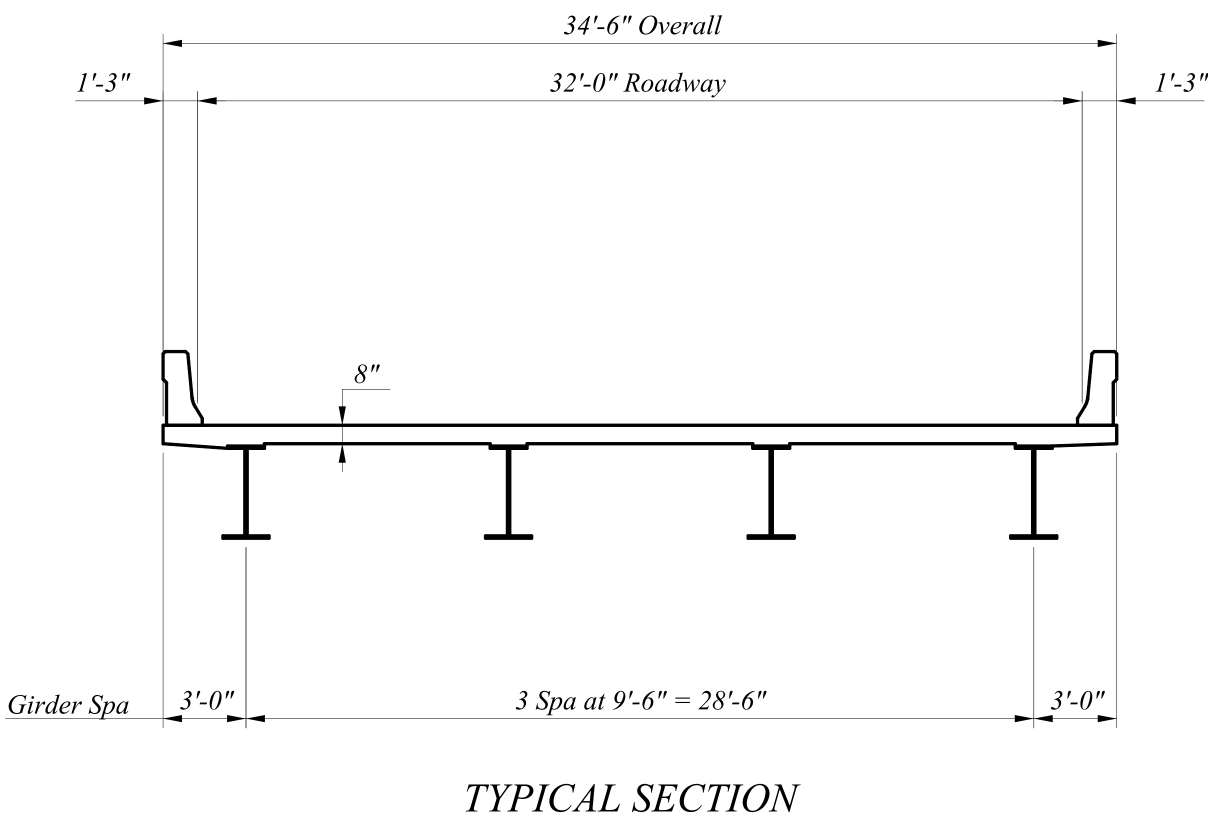

Figure 6-1, Example 2 Bridge, Typical Section

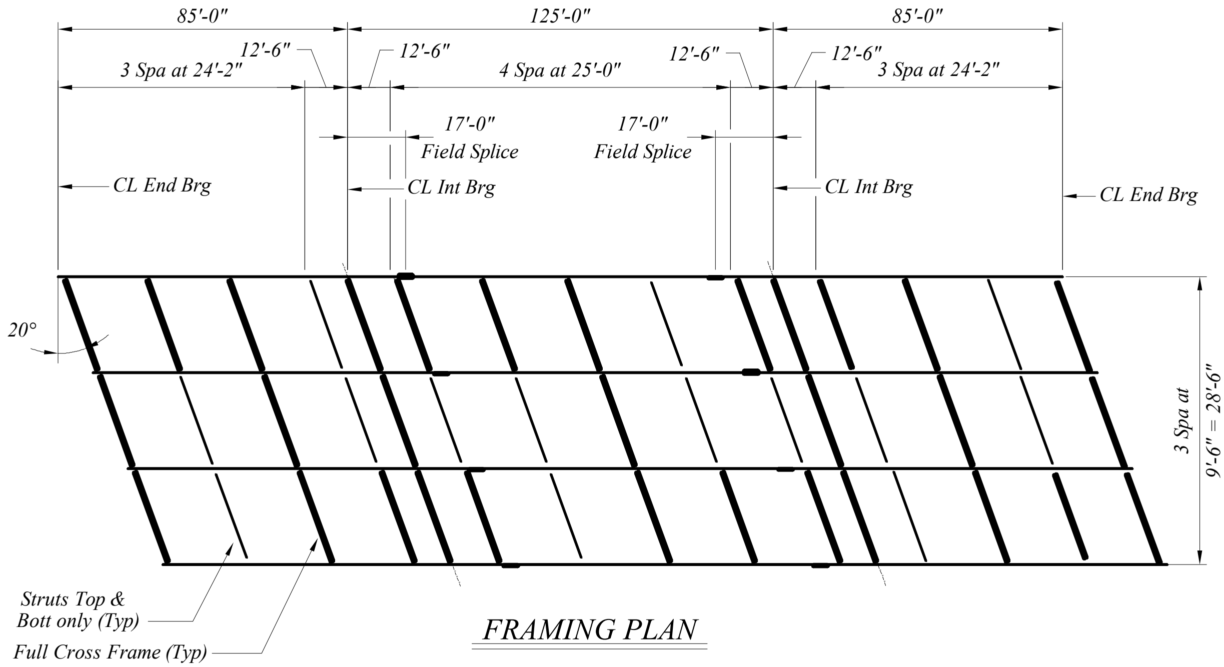

Figure 6-2, Example 2 Bridge, Framing Plan

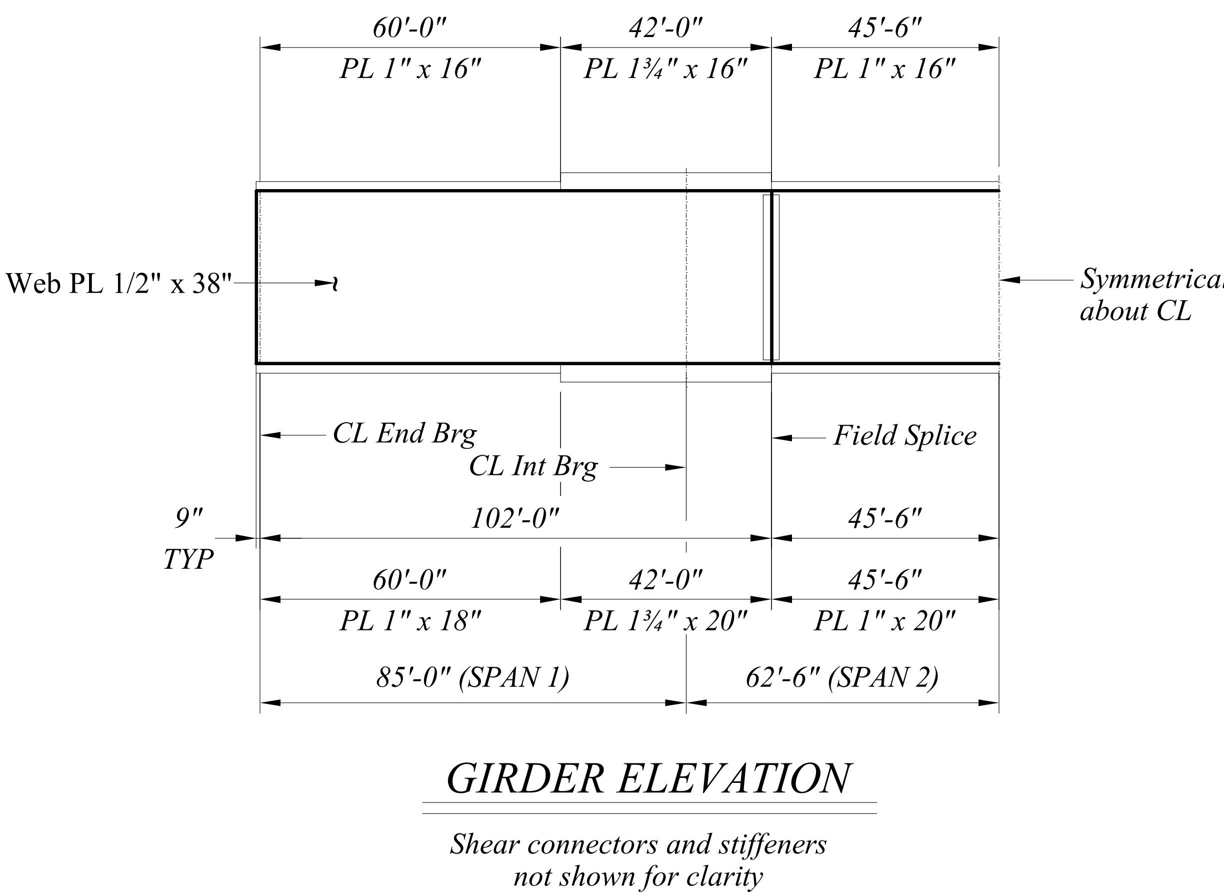

Figure 6-3, Example 2 Bridge, Girder Elevation

6.2 LEAN-ON BRACING DESIGN, POSITIVE MOMENT REGION

In this example the formulas used for lean-on braces are designed considering they are provided in the exterior bays which is a more conservative approach than using the formulas for lean-on braces provided in interior bays. The bending moment values used in this example are for the exterior girders. The peak moments in the positive moment region are considered for design of the lean-on bracing system.

6.2.1 Input Data

Span under consideration	L	= Span 1 (85 ft)
Number of girders in cross-section	n_g	= 4
Girder Spacing	S	= 9.5 ft
Skew	θ	= 20 deg
Number of brace points in span	n	= 3
Unbraced length of the segment	L_b	= 24 ft (cross-frame spacing)

6.2.2 Cross-Section Properties

Item	Width, in.	Height, in.	y (from bottom of bottom flange), in.	Area, in.^{2}	A*y, in.^{3}	I_x, in.^{4}	I_y, in.^{4}	A*d^2
Bottom Flange	18.00	1.000	0.5	18	9.00	1.50	486.0	6338
Web	0.5000	38.00	20	19	380.00	2286	0.4	10
Top Flange	16.00	1.000	39.5	16	632.00	1.33	341.3	6552
	Sum	40		53	1021	2289	828	12900

Web depth	D	= 38 in.
Web thickness	t_w	= 0.5 in.
Moment of Inertia (strong axis)	I_x	= 15189 in.^{4}
Moment of Inertia (weak axis)	I_y	= 828 in.^{4}
Distance of centroid from bottom	d_{bottom}	= 19.26 in.
Distance of centroid from top	d_{top}	= 20.74 in.
Section modulus bottom	$S_{x\text{-}bot}$	= 788 in.^{3}
Section modulus top	$S_{x\text{-}top}$	= 732 in.^{3}

Distance from centroid of girder to centroid of compression flange

c = 20.24 in.

Distance from centroid of girder to centroid of tension flange

t = 18.76 in.

Distance between centroid of compression and tension flange

h_o = 20.24 + 18.76 = 39 in.

Moment of inertia of compression flange about vertical axis (I_y top flange)

I_{yc} = 341.3 in.^{4}

Moment of inertia of tension flange about vertical axis (I_y bottom flange)

I_{yt} = 486 in.^{4}

Effective Moment of Inertia of girder about vertical axis ($I_{y\text{-}eff}$ or I_{eff})

$$I_{eff} = I_{yc} + \left(\frac{t}{c}\right) I_{yt} = 341.4 + \left(\frac{18.76}{20.24}\right) \times 486 = 792 \text{ in.}^4$$

Steel modulus of elasticity E = 29000 ksi

6.2.3 Cross-Frame Properties

Connection Plate Properties:

Width of connection plate b_s = 9 in.

Thickness of connection plate t_s = 0.5 in.

Distance from top of cross-frame to bottom of top flange

h_i = 3.5 in.

Height of cross-frame h_b = $D - 2 \times h_i = 38 - 2 \times 3.5 = 31$ in.

The connection plate is connected to top and bottom flange of girder.

Diagonal Properties:

Length of Diagonal L_d = $\sqrt{\left((S \times 12) - 2 \times b_s\right)^2 + h_b^{\,2}} = 100$ in.

Strut Properties:

Length of Strut L_s = $\left(S - 2 \times b_s\right) = 96$ in.

6.2.4 Structural Loads

AASHTO LRFD Article 3.4.2.1 prescribes two sets of load factors for construction loads with steel structures at the strength limit state. The first set uses 1.25 (minimum) for DC and DW and 1.5 (minimum) for construction live load. The second set is specific to *primary* steel superstructure components applied to the *fully erected* steelwork [emphasis added]. For this specific case, a minimum load factor of 1.4 is applied to both DC and construction LL. With cross-frames being defined as secondary members per AASHTO LRFD Table 6.6.2.1-1 and not primary members, for the straight steel girders for which lean-on bracing may be used there is ambiguity in load factors for the design of stability bracing. The analysis for this example used a very conservative approach to construction LL and the first set of load factors is used to prevent an unwarranted level of conservatism.

A structural analysis was performed and the maximum positive moments are governed by the end spans. These moments are as follows:

Unfactored dead load moment M_{DL} = 5700 kip-in.

Unfactored construction live load moment M_{LL} = 4050 kip-in.

Construction dead load factor γ_{DL} = 1.25

Construction live load factor γ_{LL} = 1.50

Factored positive moment M_u = 13200 kip-in.

Moment gradient modifier C_b = 1.0

6.2.5 Bolt Details

Bolt diameter d_b = 0.875 in.

Bolt yield strength F_{yb} = 70 ksi

Bolt tensile strength F_{ub} = 120 ksi

6.2.6 Torsional Brace Stiffness Requirements (Positive Moment Case)

The relationship between the actual stiffness provided by lean-on bracing system and the required stiffness of the system can be determined by this equation:

$$(\beta_T)_{act} \geq (\beta_T)_{req}$$

Where:

$(\beta_T)_{act} =$ actual stiffness provided by LOB system
$(\beta_T)_{req} =$ required stiffness of LOB system

6.2.7 Determine Actual Stiffness Provided with Lean-On Brace System $(\beta_T)_{act}$

The actual provided stiffness of a lean-on brace system for the bridge is a function of cross-frame or diaphragm stiffness, in-plane girder stiffness, and cross-sectional distortion stiffness. The following equation is used to determine the actual stiffness provided by the lean-on brace system:

$$(\beta_T)_{act} = \frac{1}{\frac{1}{\beta_{br}} + \frac{1}{\beta_{\sec}} + \frac{1}{\beta_g}}$$

Where:

$\beta_{br} =$ brace stiffness of the diaphragm or cross-frame
$\beta_{\sec} =$ cross-sectional distortion stiffness
$\beta_g =$ effective in-plane girder stiffness

Each of the components needed to determine the actual provided stiffness of the bracing system are calculated in this section.

Brace Stiffness (Cross-Frame Stiffness)

Two equations are provided for cross-frame stiffness, one for exterior bays and one for interior bays. With a lean-on brace system, the normal equation for cross-frame stiffness must be modified. Use of the exterior bay equation for interior bays is conservative and used here. For X-type cross-frames with a tension-only diagonal system, the following equation can be used:

$$\beta_{br} = \frac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc} - 1\right)^2} A_b$$

Where:

$n_{gc} = \frac{n_g}{N_c}$

n_g = number of girders in bridge cross-section = 4

N_c = number of cross frames at each brace location = 1

$n_{gc} = \frac{4}{1} = 4$

Assuming single angle section for both diagonal and strut, using L4×4×1/2 as brace member. Since a single angle is used the area of brace is reduced by a factor of 0.65

Area of brace	A_b	= 3.75 in.^{2}
Length of diagonal	L_d	= 100 in.
Girder spacing	S	= 9.5 ft. x 12 in./ft. = 114 in.
Height of cross-frame	h_b	= 31 in.

Hence, the provided torsional stiffness is calculated as below:

$$\beta_{br} = \frac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc} - 1\right)^2} \left(A_b \times 0.65\right)$$

$$= \frac{29000 \times 114^2 \times 31^2}{4 \times 100^3 + 114^3 \left(4 - 1\right)^2} 3.75 \times 0.65$$

$\boldsymbol{\beta_{br}}$ **= 50931 kip-in./rad**

Skew should be accounted for when the brace is placed parallel to the skew. The reduction factor is:

$$\beta_{br,skew} = \beta_{br} \cos^2 \theta$$

$$= 50931 \times \cos^2 \left(20\right)$$

$\boldsymbol{\beta_{br,skew}}$ **= 44973 kip-in./rad**

Cross-Sectional Stiffness

In this example the girder is a built-up I-section with cross-frames.

Height of cross-frame	h_b	= 31 in.
Web depth	D	= 38 in.
80 percent of web depth	0.8 x 38	= 30.4 in.

Since the height of the cross-frame provided is more than 80 % of web depth, cross-sectional distortion stiffness can be assumed to be infinity.

$\beta_{sec} = \infty$

$\boldsymbol{\beta_{sec}}$ **=** $\mathbf{10 \times 10^{99}}$ **kip-in./rad**

In-Plane Girder Stiffness

$$\beta_g = \frac{12\left(n_g-1\right)^2 S^2 E\, I_x}{n_g L^3}$$

$$= \frac{12\times\left(4-1\right)^2\times\left(9.5\times12\right)^2\times29000\times15189}{4\times\left(85\times12\right)^3}$$

$$\boldsymbol{\beta_g} \quad \mathbf{= 145646\ kip\text{-}in./rad}$$

Hence the actual stiffness provided by the lean-on brace system is:

$$\left(\beta_T\right)_{act} = \frac{1}{\dfrac{1}{\beta_{br,skew}}+\dfrac{1}{\beta_{\sec}}+\dfrac{1}{\beta_g}}$$

$$= \frac{1}{\dfrac{1}{44973}+\dfrac{1}{10\times10^{99}}+\dfrac{1}{145646}}$$

$$\boldsymbol{\left(\beta_T\right)_{act}} \quad \mathbf{= 34362\ kip\text{-}in./rad}$$

6.2.8 Determine Required Stiffness of Torsional Brace System $(\boldsymbol{\beta_T})_{req}$

The following equation is used to determine the required torsional brace stiffness of the positive moment region:

$$(\beta_T)_{req} = \frac{3.6\,L}{\phi_{sb} C_b^2\; n\; I_{eff} E} M_u^2$$

Where:

n = number of intermediate cross-frames lines in first span = 3

ϕ_{sb} = resistance factor for stability bracing = 0.80

Therefore, $$(\beta_T)_{req} = \frac{3.6\times85\times12}{0.8\times1.0^2\times3\times792\times29000}13200^2$$

$$\boldsymbol{(\beta_T)_{req}} \quad \mathbf{= 11607\ kip\text{-}in./rad}$$

6.2.9 Check Stiffness Requirement of the System

$$\left(\beta_T\right)_{act} \geq \left(\beta_T\right)_{req}$$

Actual provided stiffness $(\beta_T)_{act}$ = 34362 kip-in./rad

Required stiffness $(\beta_T)_{req}$ = 11607 kip-in./rad

The provided actual stiffness using a single angle section (L4×4×1/2) for a tension-only diagonal X-type cross-frame in a lean-on brace system is greater than the required stiffness. Hence the angle used satisfies the stiffness requirement of a lean-on brace system. Use single angle L4×4×1/2 for diagonal and strut members.

If the provided stiffness was insufficient, the designer can reduce the number of girders per cross-frame (n_{gc}), increase the cross-frame member size (A_b), and/or increase in-plane girder stiffness (β_g).

6.2.10 Check Strength Requirements of Lean-On Brace System (Positive Moment Case)

Along with satisfying the stiffness requirements, members of the cross-frame in a lean-on brace system should also satisfy the following strength requirements:

The required strength of a torsional brace can be calculated as follows:

$$M_{br} = \frac{0.008\,L\,L_b}{n\,E\,I_{yeff}\,h_o}\left(\frac{M_u}{C_b}\right)^2$$

$$= \frac{0.008\times1020\times288}{5\times29000\times792\times39}\left(\frac{13200}{1}\right)^2$$

$$\boldsymbol{M_{br} = 91.4\ \text{kip-in.}}$$

Skew should be accounted for when the brace is placed parallel to the skew. The additional moment due to skewed cross-frame is calculated as:

$$M_{br,skew} = \frac{M_{br}}{\cos\theta}$$

$$= \frac{91.4}{\cos(20)}$$

$$\boldsymbol{M_{br,skew} = 97.3\ \text{kip-in.}}$$

6.2.11 Calculate Applied Forces in the Cross-Frame Members

Force in Brace:

$$F_{br} = \frac{M_{br,skew}}{h_b}$$

$$= \frac{97.3}{31}$$

$$\boldsymbol{F_{br} = 3.14\ \text{kip}}$$

Force in diagonal in tension-only X-frame is:

$$F_d = \frac{n_{gc} F_{br} L_d}{S}$$

$$= \frac{4 \times 3.14 \times 100}{114}$$

$$\boldsymbol{F_d} \ \mathbf{= 11.02\ kip}$$

Force in strut when cross-frame is located in exterior bay:

$$F_s = \left(n_{gc} - 1\right) F_{br}$$

$$= \left(4 - 1\right) \times 3.14$$

$$\boldsymbol{F_s} \ \mathbf{= 9.42\ kip}$$

6.2.12 Determine Tensile Capacity of Cross-Frame Diagonal Members

Assuming two 7/8-in.-diameter bolts at each end of diagonal and strut angles to fasten it to connection plate. Using single angle section L4×4×1/2 for both diagonal and strut. For diagonals, only tension checks are calculated since a tension-only diagonal X-type cross-frame is considered. A compression check is calculated for horizontal struts as they may be in either tension or compression, with compression governing member capacity.

Gross area of angle A_g = 3.75 in.^{2}

Yield strength F_y = 50 ksi

Tensile strength F_u = 70 ksi

Bolt spacing L = 2.625 in.

Distance of centroid in x direction $\bar{x}$ = 1.18 in.

Shear lag reduction factor $U = 1 - \frac{\bar{x}}{L} = 1 - \frac{1.18}{2.625}$

$= 0.55$

Reduction factor for holes R_p = 1.0 (holes drilled to full size)

Net area of angle $A_n = A_g - 1 \times \left(d_b + \frac{1}{8}\right) \times t_{leg}$

$$= 3.75 - 1 \times \left(0.875 + \frac{1}{8}\right) \times \frac{1}{2}$$

A_n = 3.25 in.^{2}

Determining Gross Section Yielding Capacity in Tension:

Resistance factor for yielding ϕ_y = 0.95

Therefore,

Gross section yielding capacity $\phi_y P_{ny} = \phi_y F_y A_g$

$$= 0.95 \times 50 \times 3.75$$

$\boldsymbol{\phi_y P_{ny}}$ **= 178.1 kip**

Determining Net Section Fracture Capacity in Tension:

Resistance factor for fracture ϕ_u = 0.8

Therefore,

Net section fracture capacity $\phi_u P_{nu} = \phi_u F_u A_n R_p U$

$= 0.8 \times 70 \times 3.25 \times 1.0 \times 0.55$

$\boldsymbol{\phi_u P_{nu}}$ **= 100.1 kip**

The factored tensile capacity of the angle is taken as the minimum of gross section yielding capacity and net section fracture capacity.

Factored Tensile Capacity $\boldsymbol{P_r = \phi_u P_{nu}}$ **= 100.1 kip >** $\boldsymbol{F_d}$ **= 11.02 kip** **(OK)**

6.2.13 Determine Compressive Capacity of Cross-Frame and Lean-On Struts

AASHTO LRFD Article 6.9.4.4 addresses the compressive capacity of single-angle members. The capacity of the proposed L4×4×1/2 section for the horizontal cross-frame struts and lean-on struts is verified at this step.

As per AASHTO LRFD Article 6.9.4.4:

$$\frac{l}{r_x} = 79.34 < 80$$

Hence, effective slenderness ratio is calculated as:

AASHTO LRFD Eqn. 6.9.4.4-1,
$$\left(\frac{Kl}{r_x}\right)_{eff} = 72 + 0.75\frac{l}{r_x}$$

$$= 72 + 0.75 \times 79.34$$

$$\left(\frac{Kl}{r_x}\right)_{eff} = 131.5$$

Elastic critical buckling resistance of single angle is calculated as:

AASHTO LRFD Eqn. 6.9.4.1.2-1
$$P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_x}\right)_{eff}^2} A_g$$

$$= \frac{\pi^2 \times 29000}{131.5} \times 3.75$$

$\boldsymbol{P_e}$ **= 62.1 kip**

Nominal yield resistance:

AASHTO LRFD Article 6.9.4.1, $P_o = F_y A_g$

$= 50 \times 3.75$

$\boldsymbol{P_o} \quad \mathbf{= 187.5\ kip}$

As per AASHTO LRFD Article 6.9.4.1:

$$\frac{P_o}{P_e} = 3.02 > 2.25$$

Hence, nominal compressive resistance of a single angle is:

AASHTO LRFD Eqn. 6.9.4.1-2 $\quad P_n = 0.877\,P_e$

$= 0.877 \times 62.1$

$\boldsymbol{P_n} \quad \mathbf{= 54.4\ kip}$

Factored compressive resistance of a single angle is:

$\phi_c = 0.95$

$\boldsymbol{\phi_c P_n} \quad \mathbf{= 51.7\ kip > \boldsymbol{F_s} = 9.42\ kip}$ **(OK)**

6.2.14 Determine Bolt Capacity

Area of bolt $\quad A_b = \frac{\pi}{4}d_b^2 = \frac{\pi}{4}0.875^2 = 0.60\text{ in.}^2$

Resistance factor for bolts in shear $\quad \phi_s = 0.8$

Number of shear plane $\quad N_s = 1$

Nominal shear resistance of bolts $\quad R_n = 0.45 A_b F_{ub} N_s$ (threads included in shear plane)

$= 0.45 \times 0.6 \times 120 \times 1 \times 2$ (two bolts)

$R_n = 64.8$ kip

Factored shear resistance of bolts

$\boldsymbol{\phi_s R_n} \quad \mathbf{= 51.9\ kip > \boldsymbol{F_d} = 11.02\ kip}$ **(OK)**

6.2.15 Determine Bearing Capacity of Bolt Holes

Thickness of bolt bearing element $\quad t_{bearing}$ = min (connection plate thickness, angle leg thickness)

= min (0.5 in., 0.5 in.)

$t_{bearing}$ = 0.5 in.

As per AASHTO LRFD Article 6.13.2.9:

Spacing between bolts = 2.625 in. > 2.0d

End distance = 1.5 in. < 2.0d

Hence using Eq 6.13.2.9-2

Bearing capacity of bolt hole $\quad R_n = 2.4\,d\,t_{bearing} F_u$

$$= 2.4 \times 0.875 \times 0.5 \times 70 \times 2 \qquad \text{(two bolt holes)}$$

R_n = 147 kip

Resistance factor for bolt bearing ϕ_b = 0.8

Factored bearing resistance of bolt holes

$$\boldsymbol{\phi_b R_n = 117.6 \text{ kip} > F_d = 11.02 \text{ kip}} \qquad \textbf{(OK)}$$

6.2.16 Limiting Slenderness Ratio Check

As per AASHTO LRFD Article 6.9.3, the compression slenderness ratio of compression members or tension members subjected to stress reversal for secondary members is limited to 140.

$$\frac{Kl}{r} < 140$$

Slenderness ratio of L4×4×1/2 is calculated as follows:

K = 1.0 (For single angles)

$L_s = \left(S - 2 \times b_s\right) = 96$ in.

r_x = 1.21 in.

$$\frac{Kl}{r_x} = \frac{1.0 \times 96}{1.21} = 79.34 < 140$$

Angle L4×4×1/2 satisfies the limiting slenderness ratio criteria.

6.3 LEAN-ON BRACING DESIGN, NEGATIVE MOMENT REGION

In this example, the formulas used for lean-on braces are designed considering they are provided in the exterior bay which is a more conservative approach than using the formulas for lean-on braces provided in interior bays. The bending moment values used in this example are for the exterior girders. The peak moments in the negative moment region are considered for design of the lean-on bracing system.

6.3.1 Input Data

Span under consideration L = Average of Span 1 and Span 2 (105 ft)

Number of girders in cross-section n_g = 4

Girder Spacing S = 9.5 ft

Skew θ = 20 deg

Number of brace points in span n = 3

Unbraced length of the segment L_b = 12.5 ft (cross-frame spacing)

6.3.2 Cross-Section Properties

Item	Width, in.	Height, in.	y (from bottom of bottom flange), in.	Area, in.^{2}	A*y, in.^{2}	I_x, in.^{4}	I_y, in.^{4}	A*d^2
Bottom Flange	20.00	1.750	0.875	35	30.63	8.93	1166.7	11566
Web	0.5000	38.00	20.75	19	394.25	2286	0.4	55
Top Flange	16.00	1.750	40.625	28	1137.50	7.15	597.3	13029
	Sum	41.5		82	1562	2302	1764	24650

Web depth D = 38 in.

Web thickness t_w = 0.5 in.

Moment of Inertia (strong axis) I_x = 26952 in.^{4}

Moment of Inertia (weak axis) I_y = 1764 in.^{4}

Distance of centroid from bottom d_{bottom} = 19.05 in.

Distance of centroid from top d_{top} = 22.45 in.

Section modulus bottom $S_{x\text{-}bot}$ = 1415 in.^{3}

Section modulus top $S_{x\text{-}top}$ = 1201 in.^{3}

Distance from centroid of girder to centroid of compression flange

c = 18.18 in.

Distance from centroid of girder to centroid of tension flange

t = 21.57 in.

Distance between centroid of compression and tension flange

h_o = 18.18 + 21.57 = 39.75 in.

Moment of inertia of compression flange about vertical axis (I_y bottom flange)

I_{yc} = 1166.7 in.^{4}

Moment of inertia of tension flange about vertical axis (I_y top flange)

I_{yt} = 597.3 in.^{4}

Effective Moment of Inertia of girder about vertical axis ($I_{y\text{-eff}}$ or I_{eff})

$$I_{eff} = I_{yc} + \left(\frac{t}{c}\right) I_{yt} = 1166.7 + \left(\frac{18.18}{21.57}\right) \times 597.3 = 1875.5 \text{ in.}^4$$

Steel modulus of elasticity E = 29000 ksi

6.3.3 Cross-Frame Properties

Connection Plate Properties:

Width of connection plate b_s = 9 in.

Thickness of connection plate t_s = 0.5 in.

Distance from top of cross-frame to bottom of top flange

h_i = 3.5 in.

Height of cross-frame h_b = $D - 2 \times h_i = 38 - 2 \times 3.5 = 31 \text{ in.}$

The connection plate is connected to top and bottom flange of girder.

Diagonal Properties:

Length of Diagonal L_d = $\sqrt{\left((S \times 12) - 2 \times b_s\right)^2 + h_b^{\,2}} = 100 \text{ in}$

Strut Properties:

Length of Strut L_s = $(S - 2 \times b_s) = 96\, in$

6.3.4 Structural Loads

The discussion on load factors in section 6.2.4 applies here. A structural analysis was performed and the maximum negative moments obtained at the interior support are as follows:

Unfactored dead load moment M_{DL} = 20640 kip-in.

Unfactored construction live load moment M_{LL} = 6539 kip-in.

Construction dead load factor γ_{DL} = 1.25

Construction live load factor γ_{LL} = 1.50

Factored positive moment M_u = 35608 kip-in.

Moment gradient modifier C_b = 1.25

6.3.5 Bolt Details

Bolt diameter d_b = 0.875 in.

Bolt yield strength F_{yb} = 70 ksi

Bolt tensile strength F_{ub} = 120 ksi

6.3.6 Torsional Brace Stiffness Requirements (Negative Moment Case)

The relationship between the actual stiffness provided by lean-on bracing system and the required stiffness of the system can be determined by this equation:

$$(\beta_T)_{act} \geq (\beta_T)_{req}$$

Where:

$(\beta_T)_{act}$ = actual stiffness provided by LOB system
$(\beta_T)_{req}$ = required stiffness of LOB system

6.3.7 Determine Actual Stiffness Provided with Lean-On Brace System $(\beta_T)_{act}$

The actual provided stiffness of a lean-on brace system of the bridge is calculated in this section. The actual provided stiffness is a function of cross-frame or diaphragm stiffness, in-plane girder stiffness and cross-sectional distortion stiffness. The following equation is used to determine the actual stiffness provided by a lean-on brace system:

$$(\beta_T)_{act} = \frac{1}{\dfrac{1}{\beta_{br}} + \dfrac{1}{\beta_{\sec}} + \dfrac{1}{\beta_g}}$$

Where:

β_{br} = brace stiffness of the diaphragm or cross-frame
$\beta_{\sec}$ = cross-sectional distortion stiffness
β_g = effective in-plane girder stiffness

Each of the components needed to determine the actual provided stiffness of the bracing system are calculated in this section.

Brace Stiffness (Cross-Frame Stiffness)

For X-type cross-frame with tension-only diagonal system, the following equation can be used:

Assuming the lean-on brace is provided in the exterior bay.

$$\beta_{br} = \frac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc} - 1\right)^2} A_b$$

Where:

$n_{gc} = \dfrac{n_g}{N_c}$

n_g = number of girders in bridge cross-section = 4

N_c = number of cross frames at each brace location = 1

$n_{gc} = \dfrac{4}{1} = 4$

Assuming single angle section for both diagonal and strut, try L4×4×1/2 as brace member. Since a single angle is used the area of brace is reduced by a factor of 0.65

Area of brace	A_b	= 3.75 in.^{2}
Length of diagonal	L_d	= 100 in.
Girder spacing	S	= 9.5 ft. x 12 in./ft. = 114 in.
Height of cross-frame	h_b	= 31 in.

Hence, the provided torsional stiffness is calculated as below:

$$\beta_{br} = \frac{ES^2 h_b^2}{n_{gc} L_d^3 + S^3 \left(n_{gc} - 1\right)^2} \left(A_b \times 0.65\right)$$

$$= \frac{29000 \times 114^2 \times 31^2}{4 \times 100^3 + 114^3 \left(4 - 1\right)^2} 3.75 \times 0.65$$

$\boldsymbol{\beta_{br}}$ **= 50931 kip-in/rad**

Skew should be accounted for when the brace is placed parallel to the skew. The reduction factor is:

$$\beta_{br,skew} = \beta_{br} \cos^2 \theta$$

$$= 50931 \times \cos^2 \left(20\right)$$

$\boldsymbol{\beta_{br,skew}}$ **= 44973 kip-in./rad**

Cross-Sectional Stiffness

In this example the girder is a built-up I-section with cross-frames.

Height of cross-frame	h_b	= 31 in.
Web depth	D	= 38 in.
80 percent of web depth		= 0.8 x 38 = 30.4 in.

Since the height of the cross-frame provided is more than 80 percent of web depth, cross-sectional distortion stiffness can be assumed to be infinity.

$$\beta_{sec} = \infty$$

$\boldsymbol{\beta_{sec}}$ **=** $\mathbf{10 \times 10^{99}}$ **kip-in./rad**

In-plane Girder Stiffness

$$\beta_g = \frac{12\left(n_g-1\right)^2 S^2 E\, I_x}{n_g L^3}$$

$$= \frac{12\times\left(4-1\right)^2\times\left(9.5\times12\right)^2\times29000\times26952}{4\times\left(105\times12\right)^3}$$

$$\boldsymbol{\beta_g} \quad \mathbf{= 137106\ kip\text{-}in./rad}$$

Hence the actual stiffness provided by the lean-on brace system is:

$$\left(\beta_T\right)_{act} = \frac{1}{\dfrac{1}{\beta_{br,skew}}+\dfrac{1}{\beta_{\sec}}+\dfrac{1}{\beta_g}}$$

$$= \frac{1}{\dfrac{1}{44973}+\dfrac{1}{10\times10^{99}}+\dfrac{1}{137106}}$$

$$\left(\boldsymbol{\beta_T}\right)_{\boldsymbol{act}} \quad \mathbf{= 33865\ kip\text{-}in./rad}$$

6.3.8 Determine Required Stiffness of Torsional Brace System $(\beta_T)_{req}$

The following equation is used to determine required torsional stiffness of the lean-on brace system:

$$(\beta_T)_{req} = \frac{3.6\,L}{\phi_{sb} C_b^2\; n\; I_{eff} E} M_u^2$$

Where:

n	= number of intermediate cross-frames lines in first span	= 3
ϕ_{sb}	= resistance factor for stability bracing	= 0.80

Therefore, $$(\beta_T)_{req} = \frac{3.6\times105\times12}{0.8\times1.25^2\times3\times1875.5\times29000}35608^2$$

$$\boldsymbol{(\beta_T)_{req}} \quad \mathbf{= 28198\ kip\text{-}in./rad}$$

6.3.9 Check Stiffness Requirement of the System

$$\left(\beta_T\right)_{act} \geq \left(\beta_T\right)_{req}$$

Actual provided stiffness	$(\beta_T)_{act}$	= 33865 kip-in./rad
Required stiffness	$(\beta_T)_{req}$	= 28198 kip-in./rad

The provided actual stiffness using a single angle section (L4×4×1/2) for a tension-only diagonal X-type cross-frame in a lean-on brace system is greater than the required stiffness. Hence the angle used satisfies the stiffness requirement of a lean-on brace system. Use single angle L4×4×1/2 for diagonal and strut members.

If the provided stiffness was insufficient, the designer can reduce the number of girders per cross-frame (n_{gc}), increase the cross-frame member size (A_b), and/or increase in-plane girder stiffness (β_g).

6.3.10 Check Strength Requirements of Lean-On Brace System (Negative Moment Case)

Along with satisfying the stiffness requirements, members of the cross-frame in a lean-on brace system should also satisfy the following strength requirements:

The required strength of a torsional brace can be calculated as follows:

$$M_{br} = \frac{0.008\,L\,L_b}{n\,E\,I_{yeff}\,h_o}\left(\frac{M_u}{C_b}\right)^2$$

$$= \frac{0.008\times1260\times150}{5\times29000\times1875.5\times39.75}\left(\frac{35608}{1.25}\right)^2$$

$$\boldsymbol{M_{br} = 113.5 \text{ kip-in.}}$$

Skew should be accounted for when the brace is placed parallel to the skew. The additional moment due to skewed cross-frame is calculated as:

$$M_{br,skew} = \frac{M_{br}}{\cos\theta}$$

$$= \frac{113.5}{\cos(20)}$$

$$\boldsymbol{M_{br,skew} = 120.7 \text{ kip-in.}}$$

6.3.11 Calculate Applied Forces in the Cross-Frame Members

Force in Brace:

$$F_{br} = \frac{M_{br,skew}}{h_b}$$

$$= \frac{120.7}{31}$$

$$\boldsymbol{F_{br} = 3.89 \text{ kip}}$$

Force in diagonal in tension-only X-frame is:

$$F_d = \frac{n_{gc}F_{br}\,L_d}{S}$$

$$= \frac{4\times3.89\times100}{114}$$

$$\boldsymbol{F_d = 13.66 \text{ kip}}$$

Force in strut when cross-frame is located in exterior bay:

$F_s \quad = \left(n_{gc} - 1\right) F_{br}$

$\quad = \left(4 - 1\right) \times 3.89$

$\boldsymbol{F_s} \quad$ **= 11.67 kip**

6.3.12 Determine Tensile Capacity of Cross-Frame Diagonal Members

Assuming two 7/8-in. diameter bolts at each end of diagonal and strut angle to fasten it to connection plate. Using single angle section L4×4×1/2 for both diagonal and strut. For diagonals, only tension checks are calculated since a tension-only diagonal X-type cross-frame is considered. A compression check is calculated for horizontal struts as they may be in either tension or compression, with compression governing member capacity.

Gross area of angle $\quad A_g = 3.75 \text{ in.}^2$

Yield strength $\quad F_y = 50 \text{ ksi}$

Tensile strength $\quad F_u = 70 \text{ ksi}$

Bolt spacing $\quad L = 2.625 \text{ in.}$

Distance of centroid in x direction $\quad \bar{x} = 1.18 \text{ in.}$

Shear lag reduction factor $\quad U = 1 - \dfrac{\bar{x}}{L} = 1 - \dfrac{1.18}{2.625}$

$\quad = 0.55$

Reduction factor for holes $\quad R_p = 1.0$ (holes drilled to full size)

Net area of angle $\quad A_n = A_g - 1 \times \left(d_b + \dfrac{1}{8}\right) \times t_{leg}$

$\quad = 3.75 - 1 \times \left(0.875 + \dfrac{1}{8}\right) \times \dfrac{1}{2}$

$A_n \quad = 3.25 \text{ in.}^2$

Determine Gross Section Yielding Capacity in Tension:

Resistance factor for yielding $\quad \phi_y = 0.95$

Therefore,

Gross section yielding capacity $\quad \phi_y P_{ny} = \phi_y F_y A_g$

$\quad = 0.95 \times 50 \times 3.75$

$\boldsymbol{\phi_y P_{ny}} \quad$ **= 178.1 kip**

Determining Net Section Fracture Capacity in Tension:

Resistance factor for fracture $\quad \phi_u = 0.8$

Therefore,

Net section fracture capacity $\quad \phi_u P_{nu} = \phi_u F_u A_n R_p U$

$\quad = 0.8 \times 70 \times 3.25 \times 1.0 \times 0.55$

$\boldsymbol{\phi_u P_{nu}} \quad$ **= 100.1 kip**

The factored tensile capacity of the angle is taken as the minimum of gross section yielding capacity and net section fracture capacity.

Factored Tensile Capacity $\quad P_r \quad = \phi_u P_{nu}$ **= 100.1 kip >** F_d **= 13.66 kip** **(OK)**

6.3.13 Determine Compressive Capacity of Cross-Frame and Lean-On Brace Struts

AASHTO LRFD Article 6.9.4.4 addresses the compressive capacity of single-angle members. The capacity of the proposed L4×4×1/2 section for the horizontal cross-frame struts and lean-on brace struts is verified at this step.

As per AASHTO LRFD Article 6.9.4.4:

$$\frac{l}{r_x} = 79.34 < 80$$

Hence, effective slenderness ratio is calculated as:

AASHTO LRFD Eqn. 6.9.4.4-1,

$$\left(\frac{Kl}{r_x}\right)_{eff} = 72 + 0.75\frac{l}{r_x}$$

$$= 72 + 0.75 \times 79.34$$

$$\left(\frac{Kl}{r_x}\right)_{eff} = 131.5$$

Elastic critical buckling resistance of single angle is calculated as:

AASHTO LRFD Eqn. 6.9.4.1.2-1

$$P_e = \frac{\pi^2 E}{\left(\frac{Kl}{r_x}\right)_{eff}^2} A_g$$

$$= \frac{\pi^2 \times 29000}{131.5} \times 3.75$$

$$\boldsymbol{P_e = 62.1\ \text{kip}}$$

Nominal yield resistance:

AASHTO LRFD Article 6.9.4.1,

$$P_o = F_y A_g$$

$$= 50 \times 3.75$$

$$\boldsymbol{P_o = 187.5\ \text{kip}}$$

As per AASHTO LRFD Article 6.9.4.1:

$$\frac{P_o}{P_e} = 3.02 > 2.25$$

Hence, Nominal compressive resistance of single angle is:

AASHTO LRFD Eqn. 6.9.4.1-2

$$P_n = 0.877 P_e$$

$$= 0.877 \times 62.1$$

$$\boldsymbol{P_n} = \mathbf{54.4\ kip}$$

Factored compressive resistance of single angle is:

$$\phi_c = 0.95$$

$$\boldsymbol{\phi_c P_n} = \mathbf{51.7\ kip} > \boldsymbol{F_s} = \mathbf{11.67\ kip} \qquad \textbf{(OK)}$$

6.3.14 Determine Bolt Capacity

Area of bolt $A_b = \frac{\pi}{4}d_b^2 = \frac{\pi}{4}0.875^2 = 0.60\ \text{in.}^2$

Resistance factor for bolts in shear $\phi_s = 0.8$

Number of shear planes $N_s = 1$

Nominal shear resistance of bolts $R_n = 0.45 A_b F_{ub} N_s$ (threads included in shear plane)

$= 0.45 \times 0.6 \times 120 \times 1 \times 2$ (two bolts)

$R_n = 64.8$ kip

Factored shear resistance of bolts

$$\boldsymbol{\phi_s R_n} = \mathbf{51.9\ kip} > \boldsymbol{F_d} = \mathbf{13.66\ kip} \qquad \textbf{(OK)}$$

6.3.15 Determine Bearing Capacity of Bolt Holes

Thickness of bolt bearing element $t_{bearing}$ = min (connection plate thickness, angle leg thickness)

= min (0.5 in., 0.5 in.)

$t_{bearing} = 0.5$ in.

As per AASHTO LRFD Article 6.13.2.9:

Spacing between bolts = 2.625 in. > $2.0d$

End distance = 1.5 in. < $2.0d$

Hence using Eq 6.13.2.9-2

Bearing capacity of bolt hole $R_n = 2.4\, d\, t_{bearing} F_u$

$= 2.4 \times 0.875 \times 0.5 \times 70 \times 2$ (two bolt holes)

$R_n = 147$ kip

Resistance factor for bolt bearing $\phi_b = 0.8$

Factored bearing resistance of bolt holes

$$\boldsymbol{\phi_b R_n} = \mathbf{117.6\ kip} > \boldsymbol{F_d} = \mathbf{13.66\ kip} \qquad \textbf{(OK)}$$

6.3.16 Limiting Slenderness Ratio Check

As per AASHTO LRFD Article 6.9.3, the compression slenderness ratio of compression members or tension members subjected to stress reversal for secondary members is limited to 140.

$$\frac{Kl}{r} < 140$$

Slenderness ratio of L4×4×1/2 is calculated as follows:

K = 1.0 (For single angles)

L_s = $\left(S-2\times b_s\right)=96\,\text{in.}$

r_x = 1.21 in.

$$\frac{Kl}{r_x}=\frac{1.0\times96}{1.21}=79.34<140$$

Angle L4×4×1/2 satisfies the limiting slenderness ratio criteria.

6.4 SUMMARY

To complete the design, the cross-frame members and lean-on brace struts should be checked for wind loads.

All cross-frame bracing members are L4×4×1/2, ASTM A709 Gr 50 Steel and are end-welded to gusset plates, which are attached to girder connection plates with 7/8-in. diameter ASTM F3125 Grade A325 bolts. Lean-on brace struts use the same members and are directly bolted to girder connection plates.

For the designed positive bending region, a n_{gc} value of 4 was used but a full line of cross-frames is provided at the peak moment location at the designer's discretion. Additional cross-frames were added adjacent to field splices to assist with geometry control. Due to the small scale of this example bridge, the ability of lean-on bracing to make a significant reduction in cross-frames is limited.

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Appendix

AASHTO Bridge Committee Agenda Item: 37

6.7.4.2.2—Stability Bracing Requirements

In addition to the minimum design requirements specified in Article 6.7.4.1, diaphragms or cross-frames for all rolled-beam and plate-girder bridges shall satisfy the following stability bracing stiffness requirement for the applicable noncomposite *DC* loads and any construction loads applied to the fully erected steelwork:

$$(\beta_T)_{act} \geq (\beta_T)_{req} \qquad (6.7.4.2.2\text{-}1)$$

in which:

$(\beta_T)_{req}$ = required stiffness of the torsional brace system (kip-in./rad)

$$= \frac{3.6L}{\phi_{sb} n E I_{yeff}} \left(\frac{M_u}{C_b} \right)^2 \qquad (6.7.4.2.2\text{-}2)$$

I_{yeff} = effective out-of-plane moment of inertia of the girder (in.^{4}) calculated as follows:

- For doubly symmetric girders:

$$= I_y \qquad (6.7.4.2.2\text{-}3)$$

- For singly symmetric girders:

$$= I_{yc} + \left(\frac{t}{c} \right) I_{yt} \qquad (6.7.4.2.2\text{-}4)$$

where:

ϕ_{sb} = resistance factor for stability bracing specified in Article 6.5.4.2

c = distance from the centroid of the girder section at the brace point under consideration to the centroid of the compression flange (in.). The distance shall be taken as positive.

C_b = moment gradient modifier within the critical unbraced beam or girder segment under consideration determined as specified in Article 6.10.8.2.3 or A6.3.3, as applicable

C6.7.4.2.2

The provisions in this article are adapted from the 2010 AISC Specification Appendix 6.3.2a as modified based on Liu et al. (2020a) and Liu and Helwig (2020b). These provisions may be applied to skewed I-girder bridges with discontinuous cross-frames or diaphragms but should not be applied to straight skewed I-girder bridges utilizing lean-on bracing systems. Such systems should be instead investigated as described in Helwig and Wang (2003).

Effective stability bracing can be achieved by either preventing lateral movement of the compression flange with lateral bracing (Yura, 2001) or by controlling twist of the cross-section with torsional bracing. Cross-frames and diaphragms enhance the lateral-torsional buckling resistance of longitudinal beams and girders by restraining twist of the cross-section at discrete locations along the length and are therefore categorized as torsional bracing. Although the beam or girder bending moments during construction are smaller in magnitude than the bending moments acting on the completed structure, the critical stage for stability bracing is during construction, including when the fully erected steelwork supports the applicable *DC* loads and any construction loads acting on the noncomposite structure during the deck casting. Cross-frames and diaphragms typically serve as the only source of bracing to the beams or girders at this stage, although in some situations, lateral bracing may also be necessary to help control lateral deflections of the fully erected steelwork due to wind and/or to provide additional stability to the fully erected steelwork during construction, as discussed further in Articles 6.7.5.2 and 6.10.3.4.2.

In the completed structure, the shear studs and composite deck provide continuous lateral and torsional restraint to the top compression flange in regions of positive flexure. Even in the negative moment regions, where shear studs may be omitted, the deck provides continuous lateral restraint to the top flange as well as some tipping restraint to limit flange twist. In this case, the demand on cross-frames and diaphragms is substantially reduced from the case where the deck is either not present or its stiffness is not yet fully effective; that is, when the stiffness of the deck is fully effective, the cross-frame diagonals mainly serve as struts that laterally restrain the bottom compression flange. The cross-frame brace stiffness in this case greatly exceeds

I_y = noncomposite moment of inertia about the vertical centroidal axis of the girder section at the brace point under consideration (in.^{4})

I_{yc}, I_{yt} = moments of inertia of the girder compression and tension flange, respectively, about the vertical centroidal axis of the section at the brace point under consideration (in.^{4})

L = length of the span under consideration (in.)

M_u = factored major-axis bending moment for the limit-state load combination specified in Article 3.4.1 or 3.4.2.1, as applicable (kip-in.). In regions of positive flexure, use the maximum factored major-axis bending moment within the region. In regions of negative flexure, use the factored major-axis bending moment at the interior support under consideration.

n = number of brace points within the span under consideration

t = distance from the centroid of the girder section at the brace point under consideration to the centroid of the tension flange (in.). The distance shall be taken as positive.

In lieu of a more refined analysis, the actual overall stiffness of the torsional bracing system, $(\beta_T)_{act}$, shall be calculated as follows:

$$(\beta_T)_{act} = \frac{1}{\left(\dfrac{1}{\beta_{br}} + \dfrac{1}{\beta_{\sec}} + \dfrac{1}{\beta_g}\right)} \qquad (6.7.4.2.2\text{-}5)$$

in which:

β_{br} = brace stiffness of the diaphragm or cross-frame that restrains twist of a beam or girder (kip-in./rad.) determined as follows:

the stiffness as a pure torsional brace. The bridge bearings and anchor bolts provide some additional lateral and torsional restraint to the bottom compression flange in these regions as well. Therefore, the stability bracing requirements for cross-frames or diaphragms are not considered to be critical for the design of the composite structure at the strength limit state. These requirements should instead be checked for the critical construction stage, which generally occurs during the placement of the concrete deck.

The stiffness requirement for the torsional brace system specified in Eq. 6.7.4.2.2-2 is based on the buckling strength equation for a beam with a continuous torsional brace along its length, derived by Taylor and Ojalvo (1966), and modified for cross-section distortion by Yura (2001). The original expression is based on the assumption that providing twice the ideal stiffness limits the out-of-plane deformations to a value equal to the initial imperfection as the critical buckling load is approached. The ideal stiffness is defined as the brace stiffness required for a perfectly straight element to reach a specified buckling capacity between the brace points. However, Liu et al., (2020a) observed that providing three times the ideal stiffness is more appropriate for flexural applications. The constant of 3.6 in the numerator of Eq. 6.7.4.2.2-2 reflects this observation.

A cross-frame or diaphragm on only one side of the beam or girder should be considered a brace point. In lieu of a more refined approach, for spans with discontinuous or staggered cross-frame or diaphragm layouts, the number of brace points, *n*, in Eqs. 6.7.4.2.2-2 and 6.7.4.2.2-13 may be taken as the maximum of the total number of brace points on each side of the beam or girder within the span under consideration. As an example, for an interior girder with 8 brace points on one side and 7 brace points on the other, *n* is to be taken as 8. For a fascia girder with 8 brace points on only one side, *n* is also to be taken as 8.

The actual overall stiffness of a torsional bracing system is significantly impacted not only by the stiffness of the diaphragm or cross-frame brace itself, but also by the girder web cross-sectional distortion at the brace point and by the in-plane stiffness of the girders that it braces. The combined effects are commonly represented as springs in series in Eq. 6.7.4.2.2-5. The total system stiffness, $(\beta_T)_{act}$, is always less than minimum of the β_{br}, β_{sec}, and β_g terms.

The equations specified herein for estimating the brace stiffness in the plane of a diaphragm or cross-frame that restrains twist of a beam or girder, β_{br}, are taken from Yura, (2001).

- For an X-type cross-frame, tension-only diagonal system, or a Z-type cross-frame with a single compression diagonal:

$$= \frac{ES^2 h_b^2}{\frac{2L_d^3}{A_d} + \frac{S^3}{A_s}} \qquad (6.7.4.2.2\text{-}6)$$

- For an X-type cross-frame, compression-diagonal system:

$$= \frac{A_d ES^2 h_b^2}{L_d^3} \qquad (6.7.4.2.2\text{-}7)$$

- For a K-type cross-frame:

$$= \frac{2ES^2 h_b^2}{\frac{8L_d^3}{A_d} + \frac{S^3}{A_s}} \qquad (6.7.4.2.2\text{-}8)$$

- For a diaphragm attached at or above mid-height of the beam or girder:

$$= \frac{6EI_b}{S} \qquad (6.7.4.2.2\text{-}9)$$

- For diaphragms attached below mid-height of the beam or girder:

$$= \frac{2EI_b}{S} \qquad (6.7.4.2.2\text{-}10)$$

In computing the brace stiffness, β_{br}, in Eqs. 6.7.4.2.2-6 through 6.7.4.2.2-8, the softening effect of eccentric end connections on the stiffness of single-angle and flange-connected tee-section cross-frame members is to be considered in accordance with the provisions of Article 4.6.3.3.4c; that is, the cross-sectional area of the cross-frame members is to be reduced by the factor specified herein to account for the inherent flexibility of the connections when investigating the noncomposite condition during construction.

The position of the torsional brace with respect to the beam or girder cross-section does not impact its effectiveness in terms of girder response. Torsional bracing attached at the level of the tension flange is just as effective as a brace attached at mid-depth or at the level of the compression flange as long as distortion of the cross-section is controlled. Although the beam or girder response is generally not sensitive to the brace location, the position of the brace on the cross-section influences the stiffness of the brace itself. For example, a torsional brace attached below mid-height of the beam or girder will tend to bend in single curvature, as reflected in Eq. 6.7.4.2.2-10, while a brace attached at or above mid-height of the beam or girder will tend to bend in reverse curvature, as reflected in Eq. 6.7.4.2.2-9.

For skewed intermediate cross-frames or diaphragms, the stiffness given by Eqs. 6.7.4.2.2-6 through 6.7.4.2.2-10 should be modified to account for the reduced effectiveness of a skewed cross-frame relative to the longitudinal axis of the girder and the additional member lengths in the skewed orientation. In such cases, $\beta_{br,\ skew} = \beta_{br} \cos^2 \theta$, where θ is the angle of a skewed intermediate diaphragm or cross-frame relative to a line normal to the longitudinal axis of the bridge (Wang and Helwig, 2008), as limited by the provisions of Article 6.7.4.2.1.

At skewed end or interior supports, cross-frames or diaphragms are routinely oriented about an axis parallel to the skewed support line, which can result in a skew angle that exceeds 20 degrees. In these instances, the effective brace stiffness can be significantly impacted by the skew angle, particularly when bent-plate connections are utilized. Despite this apparent stiffness reduction, there are several mitigating factors at these support regions, including the tipping restraint provided by bridge bearings, that improve the effectiveness of the brace and the lateral-torsional buckling resistance of the girder. Therefore, the calculation of $\beta_{br,skew}$ does not need to be considered for skewed cross-frames or diaphragms at end or interior supports. As an alternative to bent-plate connections, the use of half-round bearing stiffeners is recommended to increase the connection stiffness, the

β_{sec} = cross-sectional distortion stiffness for stability bracing (kip-in./rad.) determined as follows:

- For diaphragms and cross-frames, whose depth is at least 0.8 times the beam or girder depth, attached to full-depth connection plates positively attached to both flanges:

$$= \infty \text{ (infinity)}$$

- Otherwise:

$$\beta_{\sec i} = \frac{3.3E}{h_i}\left(\frac{D}{h_i}\right)^2\left(\frac{1.5h_i t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \quad (6.7.4.2.2\text{-}11)$$

β_g = effective in-plane girder stiffness for stability bracing (kip-in./rad.)

$$= \frac{24\left(n_g - 1\right)^2 S^2 EI_x}{n_g L^3} \quad (6.7.4.2.2\text{-}12)$$

where:

A_d = gross cross-sectional area of a cross-frame diagonal member (in.^{2}). For single-angle and flange-connected tee-section members, the area shall be multiplied by 0.65.

A_s = gross cross-sectional area of a cross-frame strut (in.^{2}). For single-angle and flange-connected tee-section members, the area shall be multiplied by 0.65.

b_s = for a connection plate or end angle on only one side of the web, width of the connection plate or projecting leg size of the end angle (in.). For connection plates or end angles on both sides of the web, total combined width of the connection plates or projecting leg sizes of the end angles (in.)

D = web depth (in.)

torsional warping restraint, and the lateral-torsional buckling resistance of the girder, as discussed further in Article C6.10.8.2.3.

Cross-sectional distortion significantly impacts the behavior of beams with relatively shallow braces with respect to the beam or girder depth. The portion of the web that impacts distortion is the region above and below the brace. Therefore, web distortion essentially cannot occur for cross-frames or diaphragms that are nearly the full depth of the web. For braces that are less than the full web depth, distortion is controlled with the connection plates. The impact of cross-sectional distortion stiffness is accounted for with the term, β_{sec}, in the calculation of the actual overall bracing system stiffness in Eq. 6.7.4.2.2-5. For cross-frames or diaphragms whose depth is at least 0.8 times the beam or girder depth, attached to full-depth connection plates positively attached to both flanges as specified in Article 6.6.1.3.1, the term, β_{sec}, is sufficiently large such that web distortion is generally not an issue (Yura, 2001). In such cases, β_{sec} is taken equal to infinity, and only the β_{br} and β_g terms need to be considered, as applicable. Otherwise, Eq. 6.7.4.2.2-11 is to be used to compute β_{seci} for the portions of the web above and below a less than full-depth connection plate or end angle, or above and below the cross-frame or diaphragm member, as applicable. In such cases, $1/\beta_{sec} = \sum(1/\beta_{\sec i})$.

End moments acting on cross-frames or diagonals are equilibrated by a vertical shear force acting on the longitudinal girders. The magnitude of the shear force is related to the number of girders, their in-plane flexural stiffness, and the overall bridge width in Eq. 6.7.4.2.2-12. For systems with more than three girders, in-plane girder effects are generally not a significant issue. However, narrow I-girder bridge units are most susceptible to in-plane girder effects. For cases in which the β_g term dominates the overall system stiffness in Eq. 6.7.4.2.2-5, e.g., I-girder bridge units with three or fewer girders, then global displacement amplification should also be checked based on the provisions specified in Article 6.10.3.4.2.

h_b = height of the cross-frame measured between the centroids of the top and bottom struts, or depth of the diaphragm, as applicable (in.)

h_i = for less than full-depth connection plates or end angles, distances along the web from the top and bottom of the connection plate or end angle to the adjacent flange; otherwise, distances along the web from the top and bottom of the cross-frame or diaphragm member to the adjacent flange (in.)

I_b = moment of inertia of the diaphragm member about the horizontal centroidal axis of the section (in.^{4})

I_x = noncomposite girder moment of inertia about the horizontal centroidal axis of the section at the brace point under consideration (in.^{4})

L_d = length of a diagonal member (in.)

n_g = number of girders within the span under consideration connected by the diaphragms or cross-frames

S = beam or girder spacing (in.)

t_s = thickness of the connection plate or projecting leg of the end angle (in.)

t_w = web thickness (in.)

In lieu of a more refined analysis, diaphragms or cross-frames in straight rolled-beam or plate-girder bridges with or without skew, and in horizontally curved rolled-beam or plate-girder bridges satisfying all the conditions specified in Article 4.6.1.2.4b for neglecting the effects of curvature, shall also satisfy the following stability bracing strength requirement for the applicable noncomposite *DC* loads and any construction loads applied to the fully erected steelwork:

$$M_{br} = \frac{0.008 L L_b}{n E I_{yeff} h_o}\left(\frac{M_u}{C_b}\right)^2 \qquad (6.7.4.2.2\text{-}13)$$

where:

ϕ_{sb} = resistance factor for stability bracing specified in Article 6.5.4.2

h_o = distance between the flange centroids of the girder section at the brace point under consideration (in.)

L = length of the span under consideration (in.)

L_b = unbraced length of the segment under consideration (in.). Use the average unbraced

Traditionally, Engineers have often designed diaphragms or cross-frames for bridges without significant skew or curvature based on wind loads and individual member slenderness criteria, as calculated force effects in these members are typically not available from the analysis. In most cases, standard diaphragm or cross-frame designs, based on generic calculations and/or successful past use, and requiring no bridge-specific analysis by the Engineer, have been utilized. While such approaches have proven adequate, the requirements herein are intended to provide a method for Engineers to ensure that these members have adequate strength and stiffness, in the absence of any calculated force effects other than wind, to act as effective torsional braces for the longitudinal beam or girder for loads applied to the fully erected steelwork during construction. Alternatively, stability bracing forces can be determined directly by a large-displacement analysis provided the effects of imperfections are considered.

The bracing strength requirement specified in Eq. 6.7.4.2.2-13 is equivalent to the product of the required stiffness of the torsional brace system given by Eq. 6.7.4.2.2-2 and an initial twist imperfection. The assumed critical shape imperfection for torsional bracing consists of a lateral sweep of the compression flange equal to length when investigating a point adjacent to two unbraced segments.

M_{br} = required strength of a torsional brace (kip-in.)

n = number of brace points within the span under consideration

L_b/500 and a straight tension flange, producing an initial twist equal to $L_b/500h_o$, where L_b is the unbraced length and h_o is distance between flange centroids (Wang and Helwig, 2005).

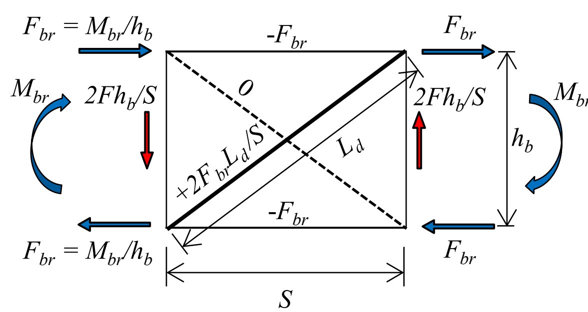
To determine the critical unbraced segment within the span under consideration, Eqs. 6.7.4.2.2-2 and 6.7.4.2.2-13 should be evaluated for the unbraced segment containing the point of maximum positive moment, or the segments adjacent to that point if a brace is located right at that point, and for the unbraced segments immediately adjacent to the interior piers. Regions of positive flexure have smaller moment magnitudes but typically have C_b values close to unity, especially for cases with many intermediate braces between the supports; for simplicity, C_b, may be taken equal to 1.0 for the evaluation of the segment or segments adjacent to the point of maximum positive moment. Regions of negative flexure adjacent to interior piers, although generally having larger moment magnitudes, are aided by steeper moment gradients, i.e., larger C_b factors, and restraint offered by the supports. An appropriate value of C_b should be calculated for the evaluation of the segments adjacent to interior piers. Regions subject to reverse-curvature bending do not significantly alter the torsional brace requirements (Helwig and Yura, 2015). For a given span, the critical unbraced segment for each respective stability requirement is the segment that maximizes that requirement. The resulting maximum requirements, as applicable, should then be applied to all the cross-frames or diaphragms within that span.

For skewed intermediate cross-frames or diaphragms, the required strength of a torsional brace given by Eq. 6.7.4.2.2-13 should be modified to account for the additional brace forces developed in the skewed orientation. In such cases, $M_{br,\,skew} = M_{br}/\cos\theta$, where θ is the angle of a skewed intermediate diaphragm or cross-frame relative to a line normal to the longitudinal axis of the bridge (Wang and Helwig, 2008), as limited by the provisions of Article 6.7.4.2.1.

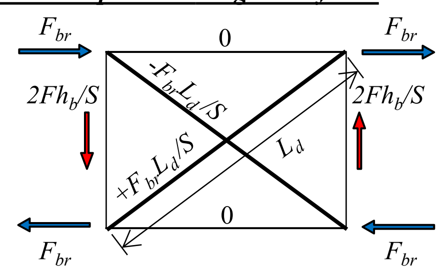
The required bracing strength, M_{br}, from Eq. 6.7.4.2.2-13 is converted to stability bracing forces in cross-frame members by dividing M_{br} by the distance between the centroids of the top and bottom struts, h_b, to obtain the required stability strut forces, F_{br}. The required stability forces in the diagonals may be obtained by multiplying F_{br} by L_d/S for an X-type cross-frame compression-diagonal configuration, and by $2L_d/S$ for a K-type cross-frame configuration and for an X-type tension-only diagonal cross-frame configuration, where L_d is the length of the diagonal and S is the beam or girder spacing. This is demonstrated schematically in Figure C6.7.4.2.2-1. The required stability bracing forces are combined with the force effects produced from other concurrently acting load cases during construction, as determined from a first-order elastic analysis, in accordance with the applicable limit-state load combination specified in Article 3.4.1 or 3.4.2.1.

Despite select top and bottom struts being represented as zero-force members in Figure C6.7.4.2.2-1, these members are essential to the effectiveness of the brace and the overall stability of the girder. Therefore, it is recommended to include both top and bottom strut members in X- and K-type cross-frames (AASHTO/NSBA, 2020).

X-Frame: Tension-Only Diagonal System



X-Frame: Compression Diagonal System



K-Frame

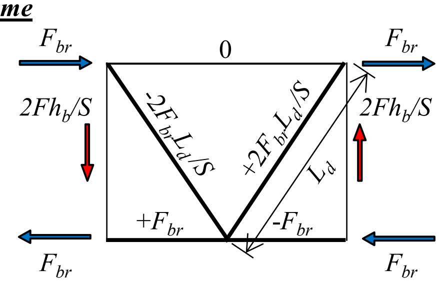


Figure C6.7.4.2.2-1—Distribution of the Required Bracing Moment, M_{br}, to Cross-Frame Members in Various Configurations

The stability bracing forces determined from Eq. 6.7.4.2.2-13, or alternatively from a large-displacement analysis considering nonlinear geometry, are to be considered an independent load effect. Given that the stability bracing forces are based on the factored beam or girder moment, M_r, an additional load factor is not to be applied to these forces.

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