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Is Your Industrial Building Structure Suitably Braced:

Should the EOR be Concerned When Detailing and Fabrication Differ from Analysis, Modeling, and Design – The Sixth Study in a Series

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Abstract

Industrial buildings typically have large, open bays, are rectangular in plan, and have heavy equipment and moving loads such that the steel framed structures must be vertically braced. This paper will examine the modeling assumptions used for the structural analysis of vertical bracing and the often-encountered inconsistency with those design assumptions when the structural steel members and connections are detailed, fabricated, and erected. Decisions made early on in the design process, with regards to the type of vertical bracing selected, can have a profound effect on the overall stability of the structure. The paper will provide guidance to the following questions that should be asked by the Engineer of Record (EOR) as part of the overall work process: 1) What happens when "minor" detailing and fabrication deviations or field changes to aid erection are not enveloped by analysis and design of individual compression members and their connections from a stability standpoint, and 2) How do these same decisions affect the overall stability of the structural steel frame?

1. Introduction

Several types of brace-to-gusset connections for vertical Hollow Structural Section (HSS) braces have been used successfully in design: claw angles, concentric or eccentric lug plate(s), and slotted gusset plate connections (Dowswell and Lini 2023). The most common is the slotted HSS connection where a slot is cut in opposite HSS walls so that it can be inserted over a gusset plate and field welded. Erection tolerances must be considered in the detailing of the slot length, but due to tight clearances as shown in Fig. 1 the brace cannot be installed without either removing one of the beams which would be unsafe or elongating one or both of the slots provided in the HSS brace member to enable the brace to be installed into its final position.

Slots are typically fabricated with a 1/2 to 1 in. (12.7 to 25.4 mm) minimum clearance beyond the edge of the gusset plate and even at that length the gap is often inadequate for the required

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erection clearance; therefore, many fabricators prefer to use a standard clearance of 2 in. (50.8 mm). Even an extended slot length of 2 in. (50.8 mm) may not allow the brace to be maneuvered into its final position. Therefore, the slot length should be carefully evaluated by the detailer. Thornton and Fortney (2012) noted that the required clearance dimension can be as high as 6 in. (152.4 mm). Because the HSS wall is an unstiffened element over the non-welded portion of the slot length, it is more susceptible to local buckling than the other segments of the member. Therefore, especially for seismic applications, the slot length should be limited to a reasonable distance.

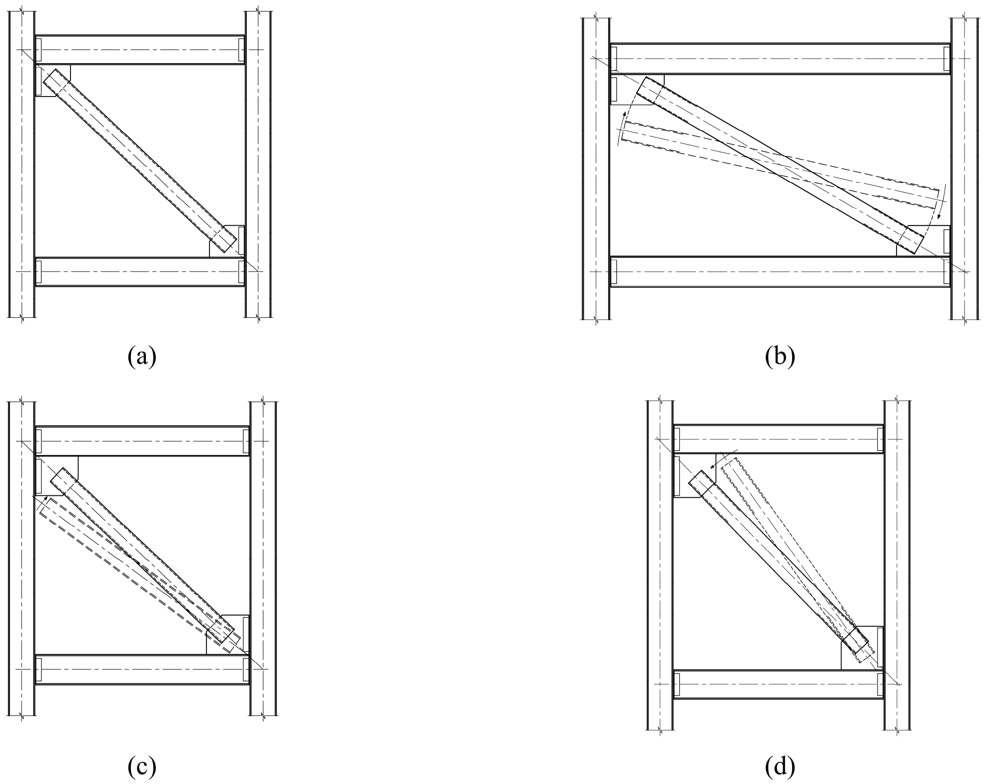


Figure 1: Erection Plans for Slotted Vertical HSS Braces (Dowswell and Lini 2023)

One real-world situation (separated into two case studies: single brace and X-brace) is presented in this paper that examines the results of decisions made in the design process that were changed, modified, or otherwise revised during detailing and/or fabrication for ease of constructability and erection of the vertical bracing systems that, when neglected or ignored, could have led to potential stability problems. Several of the following parameters were evaluated: brace material properties – elastic and inelastic, brace slot length for HSS members, brace size, and brace wall thickness; gusset plate geometry and properties; frame loading: beam transverse, or out-of-plane.

The two case studies concern the use of HSS members for vertical bracing. The braces are slotted on each end for “ease of erection” so that they can be field bolted or welded to gusset plates that have been shop welded to the columns. In practice though, there are numerous cases where the slot length in the HSS members specified by the EOR is not sufficient to allow the brace to be inserted into the partially erected structure, as the bay in which they need to be placed has been completed. This bracing system “works” in the structural analysis, but it cannot be erected unless the slot provided in the brace is lengthened. The Erector may do this without consulting the EOR, and if that is the case, there is a cross section length of the HSS that is now just two C-shaped sections opposite one another. If this HSS bracing member is subjected to a large axial compression load, there is potential for localized buckling in this slotted region. On the other hand, if the bracing member is subjected to a large axial tension load, there is potential for a net section rupture to occur. Results from finite element analyses will be presented that demonstrate the effects of various parameters described.

2. Background – Impetus for Study

2.1 Braced Framed Design (An Informal History)

This study provides a concise review of notable contributions to the field of braced frame design over the past four decades, with particular attention to the effect of slot length on the bracing capacity of HSS members. Four broad areas of investigation were examined to establish the context and identify gaps in prior research: (1) the analysis and design of vertical bracing systems under combined gravity and lateral loads; (2) the behavior of vertical bracing systems under static and cyclic seismic loading; (3) the application of HSS bracing members as alternatives to conventional structural shapes; and (4) the analysis and design of single brace gusset plate connections incorporating HSS members in heavy construction.

The use of steel tubular members as structural elements has historical roots dating back to the 1930s. A notable example is the Littoria Observation Tower in Milan, Italy, where tubular bracing connections were employed (Abrahams 1961). In this application, gussets were inserted into slots cut into the primary structural members, with the ends of the bracing elements forked, i.e., bent to fit over the gussets, and welded. This design not only provided structural integrity but also ensured a complete seal to prevent internal corrosion, marking an early and innovative use of tubular shaped steel elements in braced systems.

The findings summarized next underscore the historical progression of research into the practice of braced frame design and highlight areas where further exploration of HSS-specific applications, such as the effect of slot length on performance, remains necessary.

Four periods of research have been identified that examined vertical bracing systems, but not specifically HSS being used as vertical bracing members: (1) the pre-1980 era, characterized by foundational studies on the behavior and design of bracing systems; (2) the period from 1980 to 1994, preceding the Northridge Earthquake, which saw advancements in the understanding of bracing systems under static and dynamic loads; (3) the post-Northridge Earthquake period from 1994 to 2010, marked by significant developments in seismic design and performance evaluation of bracing systems; and (4) the era from 2011 to the present, which has focused on innovative design methodologies and the incorporation of performance-based design principles. It is important to note that while these periods reflect a comprehensive examination of vertical bracing systems, the specific application of HSS members as vertical bracing elements has not been the central focus of research within these timeframes.

<u>The First Period, pre-1980</u>: Prior to 1980, the design of braced frame connections predominantly relied on empirical rules and established heuristics rather than rigorous analytical methods. For example, gusset plate thickness was commonly determined by matching it to the flange thickness

of the brace member, a practice guided more by practical experience than by theoretical optimization. Similarly, in bolted connections, the number of bolts required to develop the brace force was calculated based on straightforward load transfer considerations, with bolt spacing and geometric layout adhering to the minimum requirements for Grade A325 (ASTM F3125 2018) per the steel specification. These approaches, while effective in practice, were characterized by their simplicity and a lack of comprehensive analytical validation, reflecting the design philosophies of the era.

<u>The Second Period, 1980 to 1994 (pre-Northridge Earthquake)</u>: Research conducted from 1980 to 1994 marked significant advancements in the understanding and design of gusset plate connections within braced frame systems. During this time, structural tubes gained widespread acceptance as a preferred choice for vertical bracing members in high-rise buildings and large-span structures. In response to this increased adoption, the American Institute of Steel Construction (AISC) established a Heavy Braced Connection Subcommittee to address emerging design challenges.

Key contributions during this period were made by the subcommittee (Thornton 1984) and other researchers (Richard 1986; Gross and Cheok 1998), who focused on developing improved methodologies for gusset plate design. This collective effort culminated in the publication of the Uniform Force Method (Thornton 1991), which is a comprehensive yet straightforward approach for analyzing and designing gusset plates in framed structures. The Uniform Force Method provided practical solutions for accounting for combined compression, tension, shear, and axial forces acting on gusset plates. However, it is noteworthy that while the method addressed various bracing configurations, its application was not explicitly limited to systems utilizing rectangular HSS as bracing members.

<u>The Third Period, 1994 (post-Northridge Earthquake) to 2010</u>: The period following the 1994 Northridge Earthquake marked a pivotal shift in the application and design of HSS bracing members in framed structures. This shift was influenced by a heightened emphasis on seismic performance, driven by the widespread structural failures observed during the earthquake. These events subjected existing seismic design methodologies to intense scrutiny, prompting a reevaluation of design priorities within the structural engineering discipline.

Research efforts during this period focused predominantly on improving connection ductility and controlling frame drift, reflecting a departure from traditional design philosophies centered on the elastic distribution of forces. This transition underscored the growing importance of ensuring adequate energy dissipation and deformation capacity within braced frames to enhance their seismic resilience. As a result, the design of HSS bracing members and their connections began to prioritize nonlinear performance characteristics, marking a significant evolution in the field of braced frame design.

In the early stages of this period, AISC published the first **Hollow Structural Sections Connections Manual** which included the 1997 AISC Specification for the Design of Steel Hollow Structural Sections (1997). This was a significant milestone, as it provided practicing engineers with a dedicated reference and specification for the design of HSS members. The Manual details the slotted HSS/Gusset Plate Connections both in axial tension and compression but does not consider the exposed gap between the end of the gusset plate and the overall slot length provided in the HSS member. Table 1 outlines the essential limit states required to be checked. Following the publication of this Manual and Specification, these guidelines (Manual) and rules (Specification) were incorporated into the Thirteenth Edition of the **Steel Construction Manual** that contains ANSI/AISC 360-05, Specification for Structural Steel Buildings (2005).

During this period, several significant research projects explored the static and dynamic behavior of HSS connections in braced steel frames, though none specifically investigated the impact of slot gaps on the tension or compression capacity of HSS bracing members (Packer 2006, Fell et al. 2006, Saucedo 2007). The culmination of these studies, along with earlier research on rectangular or round HSS used as vertical bracing members, braced frame systems, gusset plates, and HSS to gusset plate connections, is documented in AISC Steel Design Guide 24, **Hollow Structural Section Connections** (Packer et al. 2010).

The Fourth Period, 2011 to present: From 2011 to the present, significant advancements have been made in the material behavior of HSS. Beyond previous research, it is well-established that HSS offers structural efficiency as bracing members, presenting an alternative to wide-flanges, channels, or angles. The development in manufacturing processes, including the fabrication of cold-formed HSS (ASTM A500/A500M, A847/A847M, A1065/A1065M, A1085/A1085M) and hot-formed HSS (ASTM A501/A501M, A618/A618M), has made their use in structural bracing applications both feasible and cost-effective. HSS members, with their closed, uniform cross section, demonstrate high resistance to torsional loads, axial efficiency, and suitability in resisting lateral forces due to seismic or wind loads. Their versatility and performance under dynamic loads have ensured the continued use of HSS vertical bracing members in the industry today.

Ongoing research in HSS continues to make strides in gusset plate design, focusing on applications in framed building systems (Roeder et al. 2011; Unterweger and Taras 2013), determining effective length factors (Dowswell 2012), and most recently, shear lag factors (Dowswell 2021). An alternative or enhancement to Table 1 is provided in Table 2 (Roeder et al. 2011). Additionally, significant research is being conducted on the performance of HSS in braced frames under cyclic or seismic loading (Thornton and Fortney 2012). The National Institute of Standards and Technology (NIST) has sponsored a program to examine the seismic design of steel Special Concentrically Braced Frames (SCBFs) (Sabelli et al. 2013), while the Steel Tube Institute (STI) has been exploring ways to enhance the use of HSS in Seismic Frame Systems (McCormick 2017).

During this period, two significant publications emerged alongside ongoing code development. The first, AISC Steel Design Guide 29, **Vertical Bracing Connections - Analysis and Design** (Muir and Thornton 2014), covers the analysis and design of HSS, including slotted HSS bracing connections that are field welded to a gusset plate. It highlights that the slot must be sufficiently long to accommodate erection and notes that, although HSS sections are more expensive than wide-flange sections, they are more efficient for buckling and can be a lighter alternative to equivalent-strength wide-flange sections. HSS sections are extensively used in seismic design due to the closer alignment of tensile and compressive strengths compared to wide-flange sections. The second notable publication is AISC Design Guide 24, **Hollow Structural Section Connections** Second Edition (Packer and Olson 2024).

Table 1: HSS Bracing Connections – Applicable Limit States from AISC (1997)

Axial Tension Limit State		Axial Compression Limit State	
1.	Strength based on member shear	1.	Strength based on member shear
2.	Strength of the weld connection of the gusset plate to the HSS	2.	Strength of the weld connection of the gusset plate to the HSS
3.	Strength based on gusset plate shear	3.	Strength based on gusset plate shear
4.	Strength based on bolting to the gusset plate	4.	Strength based on bolting to the gusset plate
		5.	Strength based on buckling of the gusset plate

Table 2: HSS Bracing Connections – Applicable Limit States from Roeder et al. (2011)

Limit State			
1.	Whitmore Yielding	6.	Whitmore Fracture
2.	Brace Net Section Fracture	7.	Gusset Plate Buckling
3.	Brace to Gusset Plate Weld	8.	Bolt Bearing Strength
4.	Brace to Gusset Plate Base Metal Strength	9.	Bolt Rupture Strength
5.	Brace Block Shear	10.	(*)Prying Action, Brace Compactness, Brace Slenderness

(*) Not specifically limit states in AISC provisions

2.2 The Overall Work Process – It's Not about the Actual Design

The impetus for this study was not to look at actual designs, but to understand how those designs developed through the numerous entities responsible for the evolution of a project. In terms of personnel the four major players are: the Design Engineer; the Detailer; the Fabricator; and the Erector. Each part is orchestrated by Supervisors, Project Engineers, Project Managers, and if the Project is large or complex enough overseen by a Project Director. Since many of these players do not have a direct relationship with one another there are times when decisions that are made by one group are not properly communicated with a person, party, or another group that should be responsible for the (engineering) work, e.g., design. A high-level overview of a typical project workflow is summarized in Table 3.

If one were to "walk-through" the lateral force resisting system or seismic force resisting system for a structure the following scenario might occur: the **Design Engineer** talks about their approach (i.e., the analysis model approximates a braced frame using HSS with pinned connections, K = 1.0); the **Detailer** talks about their approach (i.e., the drawings utilize certain member sizes, plate thicknesses for smaller gussets, material specifications, bolted connections, and work point locations); the **Fabricator** talks about their approach (i.e., the shop is set up for welded connections, material inventory, typical connection details, hole sizes, tolerances, gages); and the **Erector** talks about their approach (i.e., realizes the HSS braces won't fit as detailed so the slots provided are lengthened for constructability). Of course, all this "talk" is backed up by preliminary studies, structural models, and FEM analyses that has been taken from the **Research** has demonstrated that there are implications to the concentrically braced frame behavior when using first-order elastic analysis and then moving to second-order inelastic analysis.

Table 3: Typical Project Workflows

Design Engineer with oversight of the EOR	
Step 1	Establishes the Design Criteria: IBC 2021 (2021), ASCE/SEI 7-22 (2022); including Local Building Codes and other client requirements;
Step 2	Selects Materials, Structural System, and Layout: Importance Factor, Seismic Design Category; Owner/Architect; Operational constraints such as floor/roof framing – fire ratings, materials, etc.;
Step 3	Modeling: Uses commercially available software, e.g., BIM – REVIT;
Step 4	Analysis: Uses commercially available software, e.g., ETABS, GTSTRUDL, RISA, SAP; Code requirements: AISC 360-22 (2022), AISC 341-22 (2022); ASCE/SEI 7-22 (2022); Gravity loading, lateral loading – wind, seismic, crane; Serviceability criteria;
Step 5	General Member Design: Uses commercially available software, e.g., ETABS, GTSTRUDL, RISA, SAP; Performs Connection Design In-house or Delegated: Bolted connections in accordance with RCSC, AISC and welded connections in accordance with AWS, AISC; Prepares Issued For Construction (IFC) drawings
Step 6	Design Drawing Production: Uses commercially available software, e.g., BIM – REVIT, AutoCAD; Produces drawings providing full information of the structural design: Plans, Sections, Details
General Contractor / Fabricator / Detailer / Delegated Connection Designer / Erector	
Step 7	Prepares Bid based on IFC documents; Contract Award;
Step 8	Releases Detailer to produce Shop Drawings; Detailers/Steel Fabricators create their own detailing models based on IFC documents; Prepares Requests for Information (RFIs) for the Design Engineer; Design Engineer responds to RFIs: Provides updated structural steel member design information, comment resolution, and ensures consistency with IFC documents;
Step 9	Connections: Delegated Connection Designer creates connection design drawings; Prepares and submits RFIs to the Design Engineer as needed; Design Engineer reviews and responds to RFIs: Provides comment resolution and ensures consistency with existing IFC documents; Revises documents when necessary;
Step 10	Shop Drawing Review: Design Engineer ensures all structural steel design information has been properly captured in the construction drawings; Design Engineer provides resolutions to all questions (RFIs), concerns, inconsistencies between shop drawings and IFC documents;
Step 11	Structural Steel Fabrication: Fabricator orders all structural steel and connection materials, i.e., for bolted connections or welded connections; Performs all required testing in accordance with contract documents; Documents issues with material availability; Design Engineer confirm acceptable material substitutions; Ships fabricated structural steel and associated connection materials to project site per established delivery schedule; Construction Administration: Ensure all Detailer, Fabricator, Erector questions (RFIs) have been documented, answered by the Design Engineer; Affected drawings revised as necessary;
Step 12	Structural Steel Erection: Structural steel members are erected (connected) in accordance with erection drawings; Questions pertaining to orientation or other inconsistencies between drawings are documented; Design Engineer answers; Drawing revisions issued when necessary; Design Engineer may perform periodic site visits until structural steel erection is complete; Constructability issues may or may not be brought to the attention of the Design Engineer if they can be resolved in the field.

3. Responsibility for Design

The ANSI/AISC 303-22, Code of Standard Practice (COSP) for Steel Buildings and Bridges (2022) provides two overarching statements regarding the responsibility for design. The first states, "When the Owner's Designated Representative for Design (*ODRD*) provides the design, *design documents*, and *specifications*, the *fabricator* and the *erector* are not responsible for the suitability, adequacy, or building-code conformance of the design" while the second statement clarifies the parties' responsibilities, "When the *owner* enters into a direct contract with the *fabricator* to both design and fabricate an entire, completed steel structure, the *fabricator* shall be responsible for the suitability, adequacy, conformance with *owner*-established performance criteria, and building-code conformance of the *structural steel* design. The *owner* shall be responsible for the suitability, adequacy, and building-code conformance of the non-*structural steel* elements and shall establish the performance criteria for the *structural steel* frame."

In the first contractual organizational layout just described it is imperative to have a means of communication established between the EOR and the Fabricator, and eventually the Erector. What has been noted in numerous cases is that when structural members do not fit properly "as-designed" they are altered or modified in the field without the knowledge or consent of the EOR. Is it okay if this happens? The topic germane to this paper is the slot length provided by the Fabricator that is not sufficiently long to allow the rectangular HSS bracing elements to be placed in a partially erected structural frame without the slot being elongated (Dowswell and Lini 2023).

The AISC COSP (2022) addresses the various methods used for conveying the structural design documents and specifications whether they are issued for construction or issued as contract documents, i.e., a traditional design-bid-build delivery method or an alternative delivery method. Regardless of which method is employed there are explicit requirements for structural connections. The ODRD is required to indicate one of the following options for each connection:

Option 1: The complete *connection* design shall be shown in the structural *design documents*.

Option 2: The *connection* shall be designated in the structural *design documents* or *specifications* to be selected or completed by an experienced *steel detailer*.

Option 3: The *connection* shall be designated in the structural *design documents* or *specifications* to be designed by a licensed *engineer* working for the *fabricator*.

Additional information needs to be included for the specific connection design method as enumerated for each Option in the COSP. Commentary provided for Option 1 describes the information that should be provided in Items (a) through (l). Item (k) for example states "Consideration of fit-up and constructability."

All parties need to be informed when this item is an issue and all parties need to be involved in the appropriate decisions made from an analysis, design, fabrication, fabrication modification, and erection perspective to ensure that the subject member, in this specific instance a rectangular HSS bracing member, is adequately and properly modified so as not to decrease its compression or tension capacity as determined in accordance with ANSI/AISC 360-22 (2022) or ANSI/AISC 341-22 (2022) as applicable.

4. HSS Vertical Brace Member Case Studies to Study Effects of Field Modification

4.1 Case Studies Overview

As mentioned previously, one common field modification to HSS brace members is to extend the length of the slot at each end of the HSS brace that fits over the gusset plate to allow easier installation (i.e., rotating the brace into place as illustrated in Fig. 1). The length of these end slots is typically not captured in analytical models. To study the effects of neglecting the possible reduction in buckling capacity of a steel braced frame due to this type of field modification of

HSS vertical brace members in an industrial application, two buckling capacity parametric case studies from a real-world application were considered. The first case study looked at a single diagonal braced frame. The second case study used identical beam and column sizes and geometry except two HSS braces are installed in an X-braced frame configuration where the brace members are connected at midspan of the braces. Both case studies considered a braced frame comprised of slotted HSS brace members with connections where a slot is cut in opposite HSS walls so that it can be inserted over a gusset plate and field welded.

The two vertical bracing case studies were evaluated using a single bay frame system. The geometry of the single bay frame was selected from an industrial building that has been constructed recently. An isometric view of a representative analytical beam element (frame) model of the building where the braced frame geometry was selected from for the case studies is shown in Fig. 2. The building was a two-story concentrically braced frame steel structure, with floor-to-floor heights of approximately 39.33 ft (11.99 m), eight (8) bays at 19.0 ft. (5.79 m) spacing in the short direction by seven (7) bays at 30.0 ft. (9.14 m) spacing in the long direction. The structure was only braced on exterior bays.

To study the sensitivity of the HSS brace slot length on buckling capacity, a range of HSS brace wall thicknesses, gusset plate thicknesses, and slot lengths in the HSS brace members were evaluated (i.e., they were parameterized) for both case studies. To evaluate buckling capacity, eigen (linear) buckling analyses were performed for all parameters. For select parameter values, nonlinear buckling analyses were also performed.

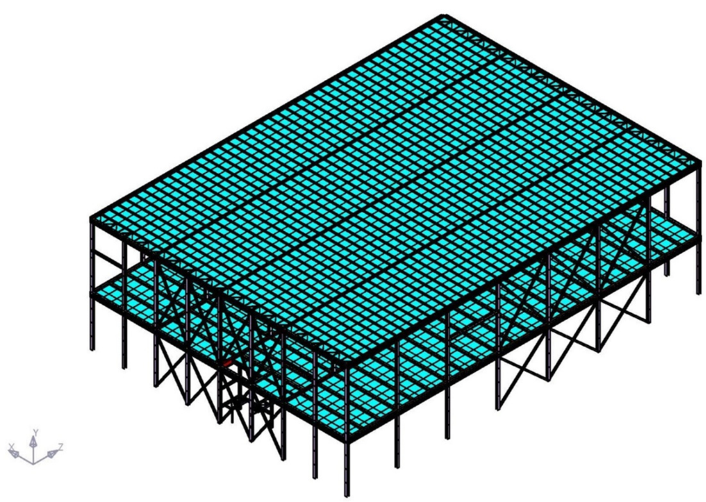


Figure 2: Isometric View of Industrial Building Used in Case Studies

4.2 Buckling Capacity Analysis Approaches

4.2.1 Finite Element Linear Eigenvalue Buckling Analysis

Linear eigenvalue buckling analysis is a computational method used to predict the theoretical critical buckling load and associated buckling mode shapes of an ideal linear elastic structure under axial compressive or lateral loads. Buckling failure is characterized by a loss of structural stiffness. Thus, in a finite element analysis the structure is evaluated using a finite element eigenvalue solution. This method assumes linear elastic material behavior and small deformations, with the system undergoing no nonlinearities in material or geometric response. The term *eigen* refers to the solution of the eigenvalue problem, where the buckling load is treated as an eigenvalue, and the corresponding buckling modes are the eigenvectors of the system. The governing equation for linear elastic buckling is shown in Eqn. 1.

$$([\boldsymbol{K}] + \lambda_i[\boldsymbol{S}])\{\psi_i\} = \{0\} \tag{1}$$

where $\boldsymbol{K}$ is the stiffness matrix corresponding to the base state, $\boldsymbol{S}$ is the additional stress-stiffening matrix or the geometric stiffness matrix due to stresses caused by the externally applied load, $\boldsymbol{F}$, and ψ_i is the buckling mode shape vector (eigenvectors) for the *i*-th mode. The outcome of interest from the analysis is the buckling load factor, λ_i, for the *i*-th mode (i.e., eigenvalues are the load multipliers). The load multiplier of a given mode is the multiplying factor to the applied load at which the structure buckles. The mode shape is the direction in which the structure buckles at a corresponding load multiplier. The applied load is typically selected so that the buckling load is greater in magnitude than the applied load.

In this analysis, the structure is subjected to a combination of applied forces, and the buckling load multiplier is determined by solving the linearized equilibrium equations. The critical load corresponds to the first eigenvalue of the system, with each subsequent eigenvalue representing higher-order buckling modes. This analysis is primarily applicable to thin-walled structures such as beams, columns, and plates, where the initial deformation due to buckling is assumed to be small relative to the structure's dimensions. Linear eigenvalue buckling analysis is a linear analysis with linear elastic material properties and utilizes small deformation theory. It idealizes force-displacement curve of members up to bifurcation point (see Fig. 3) and does not consider member imperfections, post-buckling behavior or large displacements. Thus, it provides an approximation of the critical load rather than a complete nonlinear response. However, it serves as an essential tool for preliminary design and structural stability assessments.

4.2.2 Nonlinear (2nd Order) Buckling Analysis

Nonlinear second-order buckling analysis, often referred to as geometric nonlinear buckling analysis, is a computational technique used to assess the stability of structures under load, accounting for both geometric and material nonlinearities. Unlike linear buckling analysis, which assumes small deformations and linear elastic material behavior, nonlinear second-order analysis considers the actual large displacements and the nonlinear response of the structure as it approaches or exceeds its critical buckling load (see Fig. 3 for force-displacement response). This analysis includes the effects of P-Δ (axial force-induced moments) and large displacement behavior, where deformations significantly influence the internal forces and the overall stability of the structure.

In this analysis, the governing equilibrium equations are derived based on the principles of finite deformation theory, accounting for the nonlinear response of the structural members under large displacements and are solved iteratively to capture the complex interactions between the applied loads and the resulting deformations. The second-order term arises from the consideration of the change in the structural geometry due to these large displacements, leading to a more accurate representation of the critical buckling load and post-buckling behavior. The governing system of equations is as follows:

$$[\boldsymbol{K}]\{\boldsymbol{u}\} = \{\boldsymbol{F}\} \tag{2}$$

where $\boldsymbol{K}$ is the stiffness matrix, $\boldsymbol{u}$ is the displacement vector, and $\boldsymbol{F}$ is the force vector. At buckling the reaction force term drops nearly to zero and force balance is not achieved (solution fails to converge). The analysis requires an iterative solution to obtain a result near the buckling load. The result is more accurate and can be used to predict both global and local buckling. Nonlinear second-order buckling analysis is particularly relevant for structures that experience significant deformations prior to buckling, such as thick-walled structural members, where the assumptions of linearity no longer hold. It provides a more accurate prediction of the critical load and post-buckling path, including bifurcations and possible snap-through or snap-back behaviors.

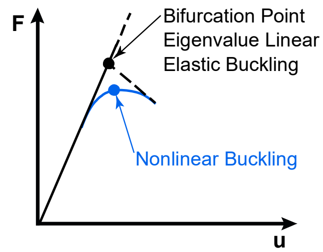


Figure 3: Force-Displacement Response during Nonlinear (2nd Order) Buckling Analysis

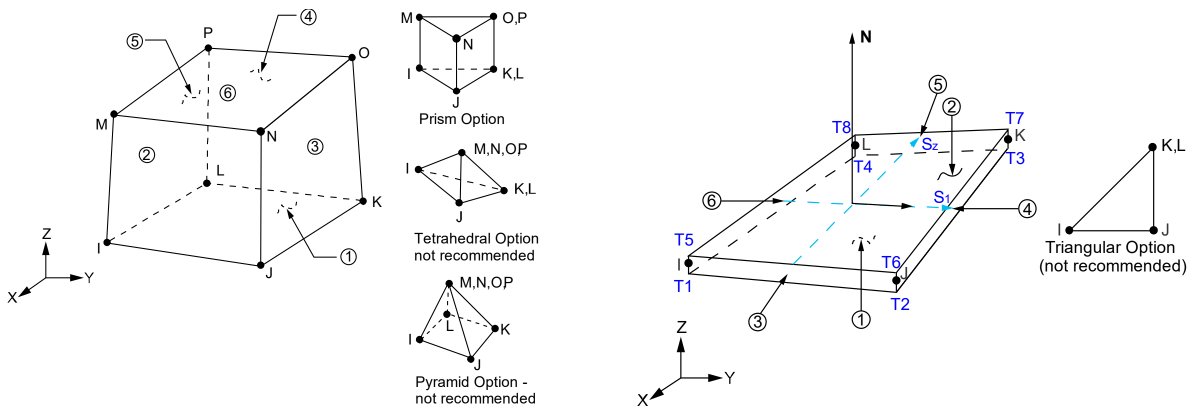
4.3 Finite Element Buckling Capacity Analysis Approach

4.3.1 Shell Element Model in ANSYS

The commercial finite element (FE) software ANSYS, Release 2022 R2 (ANSYS 2022) was employed to perform the finite element buckling analysis of the single bay frames evaluated in the case studies. ANSYS is a general-purpose FE software that has a broad range of capabilities to perform linear and nonlinear transient dynamic analysis in the time and frequency domains. ANSYS solid element SOLID185 was used to model the loading block, which is used to apply loading to the frame. SOLID185 is a 3-D, 8-node solid element. The element is defined by eight nodes having three degrees-of-freedom at each node: translations in the nodal X, Y, and Z directions. The element has plasticity, hyper-elasticity, stress stiffening, creep, large deflection, and large strain capabilities. It allows for prism, tetrahedral, and pyramid degenerations when used in irregular regions. Fig. 4(a) shows the geometry of the SOLID185 element used for the FE analysis.

ANSYS shell element SHELL181 (see Fig. 4(b)) was used to model the steel wide flange shapes for the beams and columns, the HSS brace members, and connecting plates (gusset plates and midspan connecting plate for the X-brace configuration). The mesh for the structure is shown in Fig. 6 for Case Study 1 (single HSS brace) and Fig. 12 for Case Study 2 (HSS X-brace system). The mesh of the shell elements is refined to be smaller in size where there are smaller features or where higher stress is anticipated. The element has six degrees of freedom at each node, specifically three translations and three rotations. The deformation shapes are linear in both in-plane directions. For the out-of-plane deformation, mixed interpolation is used. ANSYS SHELL181 has plasticity, hyper-elasticity, visco-elasticity, creep, swelling, user defined material, stress stiffening, large deflection, large strain, nonlinear stabilization, birth and death, adaptive descent and initial stress import capabilities. The ANSYS Element Library documentation provides more details.

The shell elements of the HSS brace members are connected to the shell elements of the gusset plates at each end of the brace members using contact elements, specifically ANSYS CONTA175 and TARGE170 elements. These contact elements connect the degrees of freedom of the HSS shell elements to the gusset plate shell elements. This approach models the HSS being welded along the axis of the HSS.

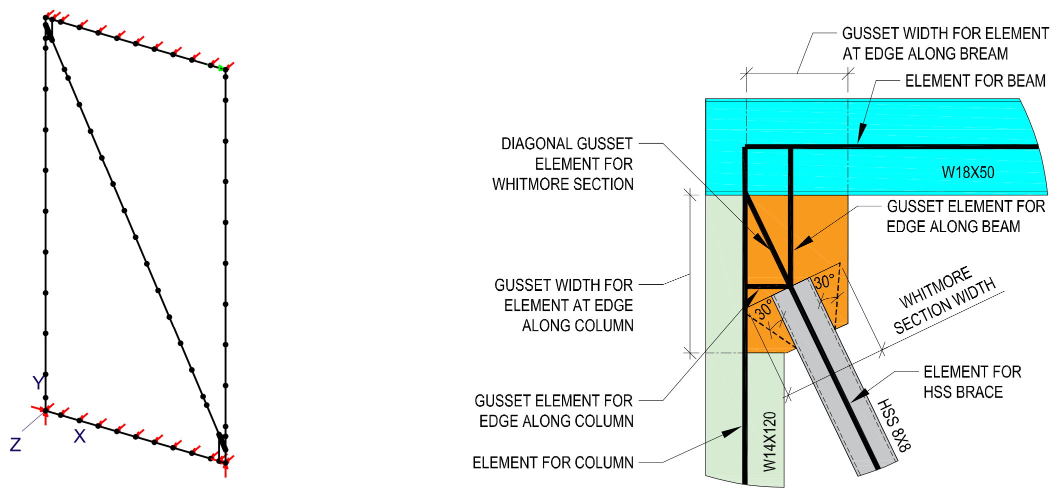


(a) (b)

Figure 4: ANSYS Elements: (a) SOLID185 Element Geometry (b) SHELL181 Element Geometry

4.3.2 Independent Beam Element Simplified Model in MASTAN2

Independent FE models were also created in MASTAN2 (2024) using beam element representations for the frame and brace members. MASTAN2 is an interactive structural analysis program that provides preprocessing, analysis, and postprocessing capabilities. The analysis routines provide 1st or 2nd order elastic or inelastic analysis of two or three-dimensional frames and trusses subjected to static loads. An isometric view of the MASTAN2 model is shown in Fig. 5(a). The gusset plate was captured using additional beam elements where the HSS brace members connect to the frame members as shown in Fig. 5(b). These independent MASTAN2 models were created for two reasons: 1) to verify reasonableness of the results from more complex ANSYS shell element models, and 2) to validate the use of simpler models.



(a) (b)

Figure 5: MASTAN2 Simplified FE Model: (a) Isometric View of Beam Elements Used to Model Single Brace Frame, and (b) Corner Detail Showing Beam Elements Used to Capture Gusset Plate Behavior

Modeling of the gusset plates in the beam element model in MASTAN2 required some simplifying assumptions. For the braced frame configuration used in the case studies, where the gusset plate is welded both to the column and beam, the brace gusset plate connections are neither pinned nor fixed joints. Also, these connections have a significant effect on the strength and stiffness of the overall braced frame system. Hence, accurate modeling of these connections is necessary to simulate overall braced frame behavior.

Previous researchers (e.g., Sabelli et al. 2013, Unterweger and Taras 2013, Hsaio 2012, Roeder et al. 2011, Saucedo 2007, Packer 2006, and Richard 1986) have considered various methods to model the connections of HSS to gusset plates. Sabelli et al. (2013) and Hsaio (2012) used a nonlinear out-of-plane rotational spring located at the physical end of the brace to simulate the out-of-plane rotational restraint of the gusset plate. The stiffness of this rotational spring was based upon the geometry and properties of the gusset plate. This approach is empirical with a basis in engineering mechanics. According to Hsaio, the out-of-plane rotational stiffness is effectively EI/L, or the out-of-plane rotational stiffness of a cantilever beam with the properties of the gusset. The stiffness is affected by the thickness and properties of the gusset. These researchers observed that the gusset plate sustains relatively minimal in-plane deformation. As a result, these researchers used rigid links to simulate this rigidity. Three rigid zones were used. The first extends from the work point of the connection to the physical end of the brace. The second extends from the work point to the physical end of the gusset along the length of the column. The third extends from the work point along the length of the beam; however, it only extends out to 75% of the gusset length on the beam. The rigid link at the ends of the brace makes the brace length identical to the actual length. Previous approaches have many desirable characteristics like the use of physical gusset parameters to model connection in-plane and out-of-plane stiffness, and the use of the physical brace length to model the brace element.

This paper sought to emulate those characteristics, but also wanted to capture gusset Whitmore section buckling, and avoid the use of rotational springs or rigid links. Therefore, beam elements were used to model the entirety of the gusset region. Three beam elements were used in total. One beam element extends from the beam-column joint to the physical end of the brace. A second element extends orthogonally from the physical end of the brace to the beam element. A third element extends orthogonally from the physical end of the brace to the column element. All three elements use properties of the gusset plate (Whitmore width for the diagonal element or actual width for the element connected to the beam and column, as well as the actual gusset plate thickness). These elements are identified in Fig. 5. This approach attempts to accurately simulate plate behavior with beam elements, thus allowing for the direct modeling of the stiffness and strength characteristics of the gusset connection. It is worth noting that the length of the three beam elements used to model the gusset plate include a portion of the column width, beam web, and column panel zone. Typically, this portion of the beam or column cross section is considered rigid for the purpose of gusset plate buckling evaluations and has been considered rigid by previous researchers. However, the physical structure (beam or column web) does have some flexibility associated with it. Additionally, the rotational support of the non-Whitmore portion of the gusset plate is modeled directly (that is, no reduced effective length factor, e.g., $K = 0.65$, is used). As a result, this modeling approach is capable of modeling both buckling of the gusset plate, rotational restraint provided to the brace, and in-plane stiffness of the joint.

The slotted portion of the HSS was modeled using a single beam element with cross-sectional properties associated with two C-shape cross sections. That is, the out-of-plane stiffness of this portion of the brace is reduced to account for the slot. This approach does not capture local buckling of the cross section.

4.3.3 Material Models Used in FE Models

The material models used for the wide flange shapes for the beams and columns were based on ASTM A992 (2011), which has a yield strength of 50 ksi and ultimate strength of 65 ksi. The HSS members were modeled with material properties of ASTM A500 (2023), Grade C, which has a yield strength of 50 ksi and ultimate strength of 62 ksi. Note that the MASTAN2 material properties do not use an input for ultimate strength.

5. Vertical Bracing Case Studies Results

5.1 Case Study 1: Single Brace Configuration

For Case Study 1 a linear eigenvalue buckling analysis was run for the single brace configuration. Various views of the FE model are shown in Fig. 6. The model consisted of approximately 30,300 elements and 30,900 nodes. Member sizes along with the significant parameters that were investigated are given in Tables 4 and 5, respectively. A total of 480 configurations were evaluated.

Table 4: Member Sizes

Member Description	Member Size
Beam Sections	Floor Beam: W33x118 | Roof Beam: W18x50
Column Section	W14x120
HSS Brace Size	8" x 8" x (varying thicknesses)

Table 5: Parameters Investigated

Parameter Description	Parameters Evaluated
HSS Brace Wall Thickness (in.)	1/8, 1/4, 3/8, 1/2
Slot Length[1] (in.)	0, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 6.0, 12.0, 18.0, 36.0
Gusset Plate Thickness (in.)	0.25, 0.375, 0.50, 0.625, 0,75, 0.875, 1.00, 1.25, 1.50, 2.00

[1] Inches from edge of gusset plate

An isometric view showing a contour plot of member deformations for linear eigenvalue analysis critical (Mode 1) buckling results from the ANSYS model (ANSYS 2022) for the frame configuration with a 1/2 in. thick gusset plate, HSS8x8x1/2 brace, and no HSS brace slot extension (i.e., the modeled slot extends only from the end of the HSS brace member to the edge of the gusset plate, where brace is welded to the gusset plate) is shown in Fig. 7(a). The deformation represents the mode shape, or eigenvectors, of the frame normalized to a value of unity. As shown in Fig. 7(a), the load multiplier obtained from the ANSYS shell element model was 80.3. Note that this and all of the other analysis results are based on an external load of 1.0 kip applied to the load plate shown in Fig. 6(d) that was oriented along the axial direction of the top wide flange beam. Gravity loading was excluded in all cases. Since the magnitude of the load applied to the frame was 1.0 kip, this means the linear eigenvalue analysis critical buckling load is 80.3 kip. This process was repeated for all 480 configurations, which are plotted in Fig. 10.

Model Information

Elements: ~30,300 Nodes: ~30,900

ANSYS Element Types:

SHELL181 for members SOLID185 for load plate TARGA170 and CONTA175 for connection of HSS to Gusset Plate

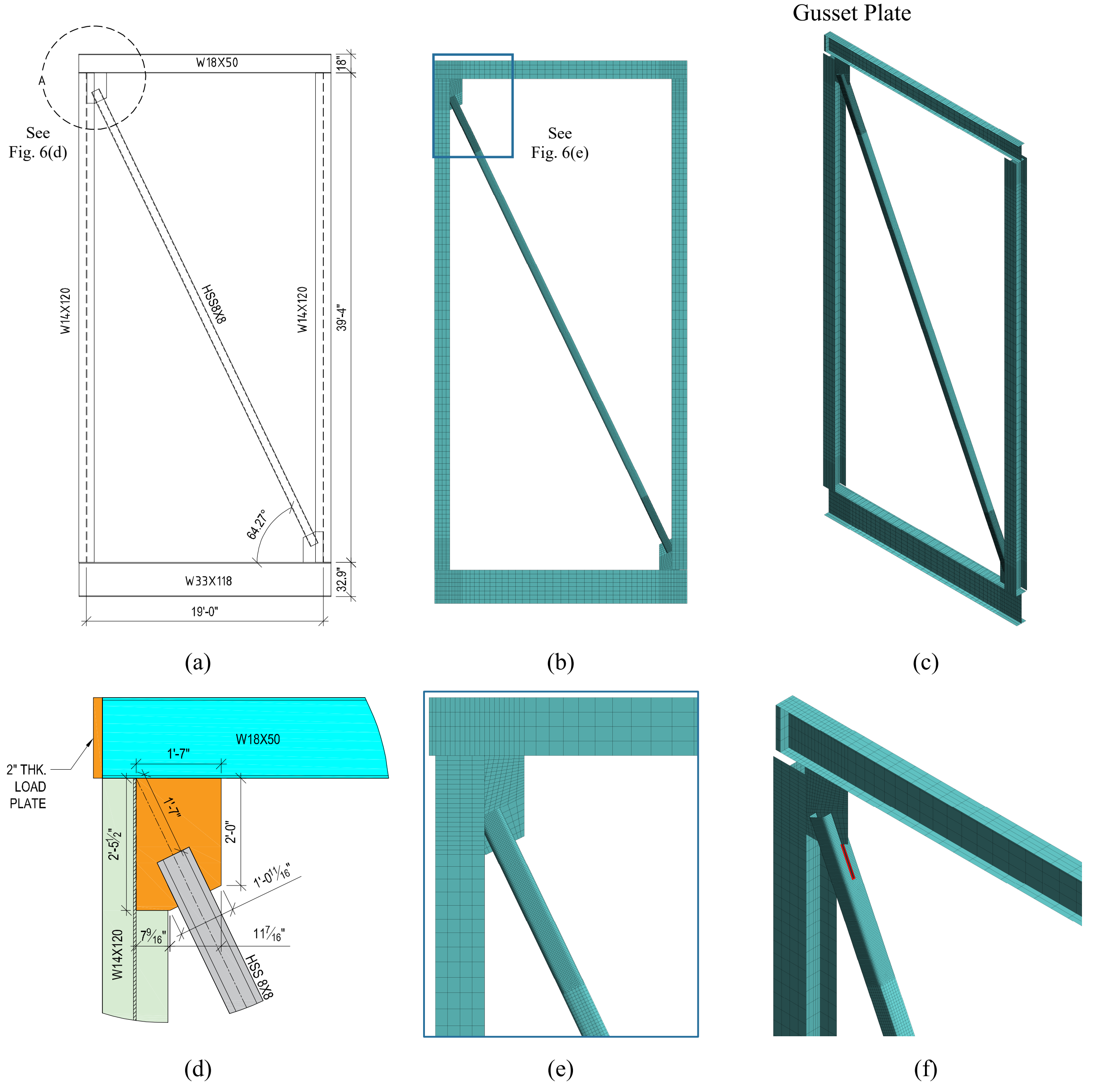
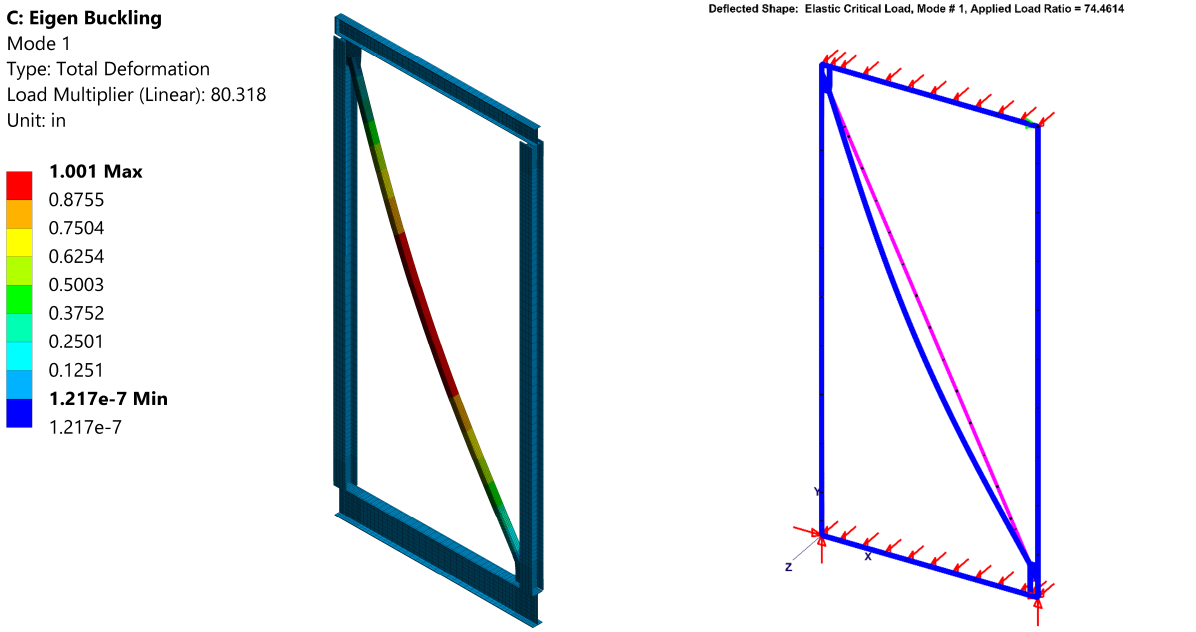


Figure 6: Model Geometry Used for Case Study 1: (a) Single Brace Elevation View; (b) FE Model Elevation View; (c) FE Model Isometric; (d) Detail A Dimensions at End Gusset Plate Location; (e) FE Model Zoomed View at Gusset Plate Location; and (f) FE Model Zoomed View Isometric Showing Location of Slot in HSS Brace

A comparison of linear eigenvalue analysis critical buckling results from MASTAN2 (2024) for the same single brace frame configuration is shown in Fig. 7(b). As shown in Fig. 7, the linear eigenvalue analysis critical buckling load from the ANSYS shell element model was 80.3 kip, compared with the MASTAN2 beam element model that had a critical buckling load of 74.5 kip. This equates to approximately 7.5% difference, which is reasonable given the considerably large computational expense of the much more complex shell element model.



(a) ANSYS Result - Mode 1 Buckling Load = 80.3 kip (b) MASTAN2 Result - Mode 1 Buckling Load = 74.5 kip

Figure 7: Comparison of Linear Eigenvalue Analysis Critical Buckling Results for Frame with 1/2" Thk. Gusset Plate, HSS8x8x1/2 Bracing Member, and No Slot Extension: (a) ANSYS Model, and (b) MASTAN2 Model

Additional comparisons between the ANSYS shell element model and MASTAN2 beam element model for configurations with a 1/2 in. thick gusset plate, HSS8x8x1/2 brace, and brace slot extension of 0 in., 6 in., and 12 in. are compared in Fig. 8. As shown in Fig. 8, the results are less than 8% over a range of slot lengths.

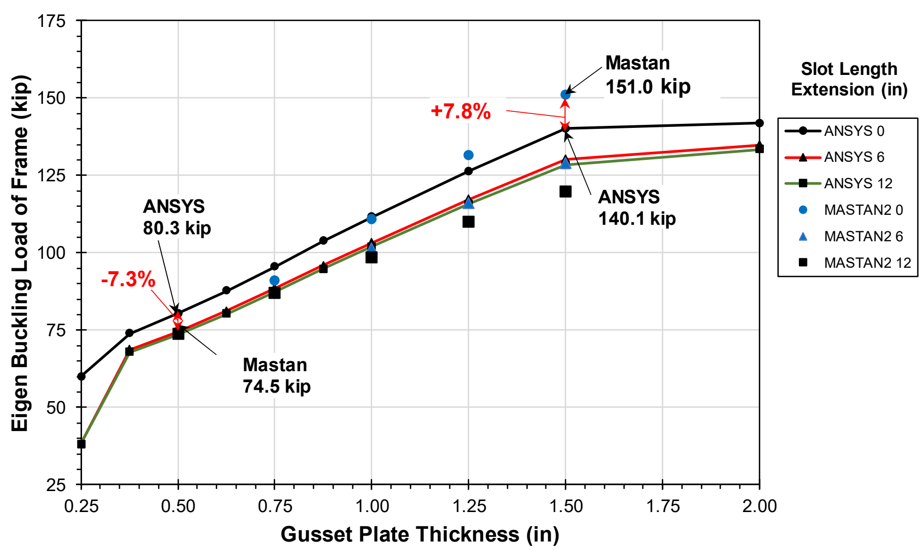


Figure 8: Single Brace Linear Eigenvalue Analysis Critical Buckling Results Comparison between ANSYS Shell Element Models and MASTAN2 Beam Element Models for HSS8x8x1/2 and 1/2 in. Thk. Gusset

Fig. 9 shows the linear eigenvalue analysis critical buckling load results from the ANSYS shell element model for the single brace frame configurations with no HSS brace slot extension (i.e., the modeled slot extends only from the end of the HSS brace member to the edge of the gusset plate, where brace is welded to the gusset plate) for the four different HSS wall thicknesses evaluated. The results in Fig. 9 show the effect of only changing the gusset plate thickness and without having a slot extension in the HSS brace. These results show there is a linear increase in buckling capacity of the frame as the gusset plate thickness is increased up to a plate thickness of approximately 1.5 in., after which the buckling capacity remains nearly constant.

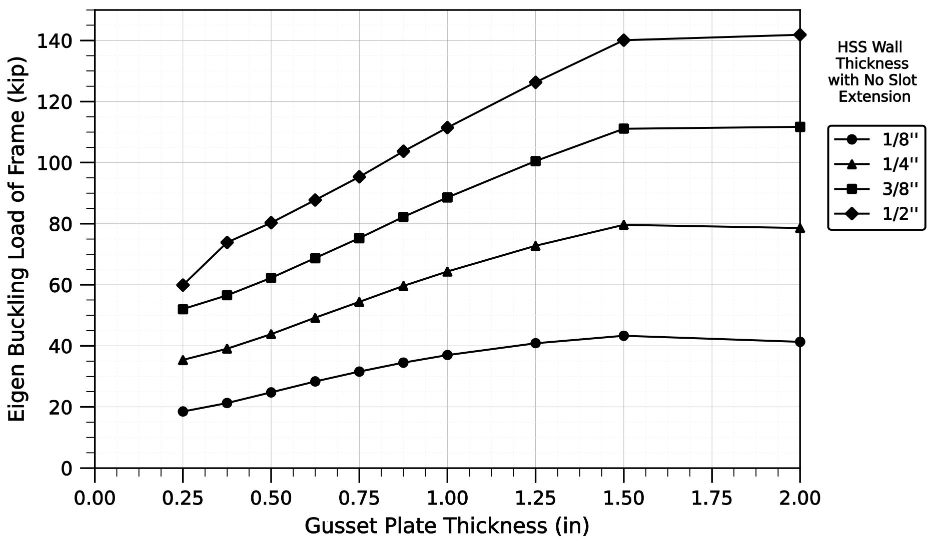
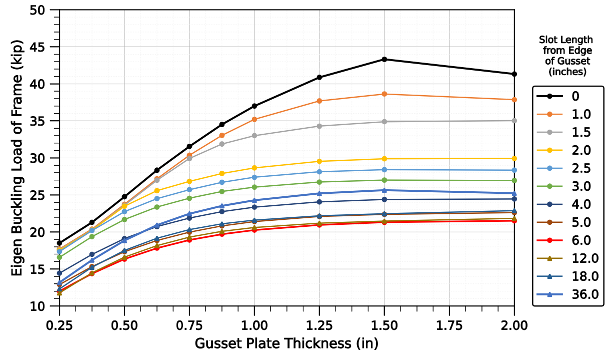


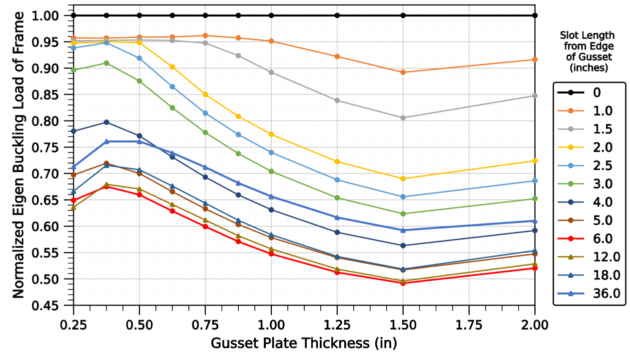
Figure 9: Single Brace Linear Eigenvalue Critical Buckling Results for Varying HSS Wall Thicknesses and Gusset Plate Thickness without Slot Extensions

The ANSYS shell element model linear eigenvalue analysis critical buckling results from all 480 configurations evaluated are plotted in Fig. 10. The results in Figs. 10(a, c, e, and g) are the raw results for critical buckling (Mode 1 from eigenvalue analysis). To compare the effect of extending the slot in the HSS brace, the results are normalized with respect to the case of no slot extension in the HSS brace and plotted in Figs. 10(b, d, f, and h). Observations and commentary on the plots are provided in Section 5.3.

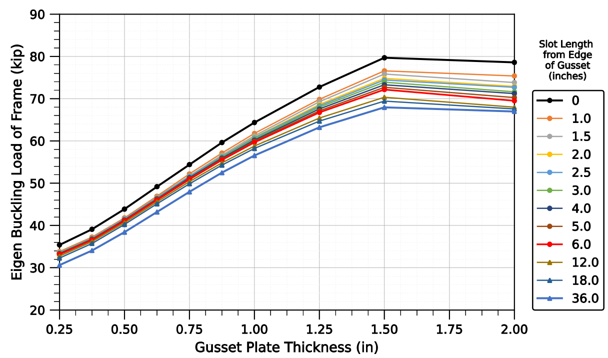
For some of the frame configurations, a 2nd order nonlinear buckling analysis was also performed. A comparison of results from the ANSYS shell element model and MASTAN2 beam element model is shown in Fig. 11. The nonlinear buckling results shown are for a frame with a 1/2" thick gusset plate, HSS8x8x1/2 brace, and a brace slot extension of 0 in. The nonlinear buckling load from the ANSYS shell model was 70.7 kip, compared to the 65.5 kip from the MASTAN2 beam element model, which is approximately 7.6% difference. Thus, the beam element modeling approach in MASTAN2 is also reasonably accurate in capturing the nonlinear buckling capacity of the frame.



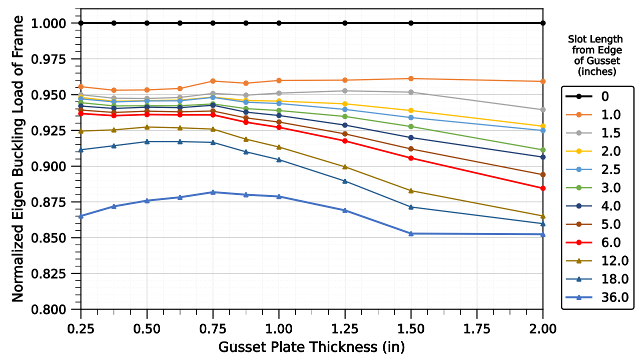
(a) Mode 1 Results HSS 1/8" Wall Thk.



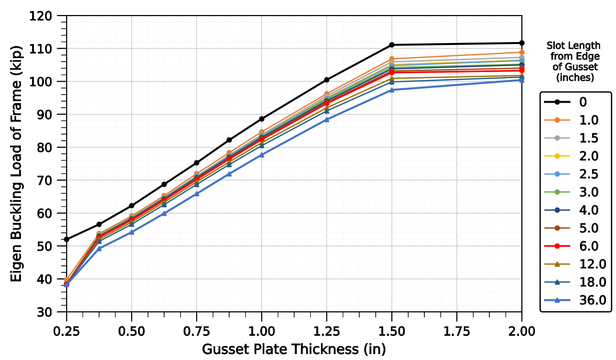
(b) Normalized Mode 1 Results HSS 1/8" Wall Thk.



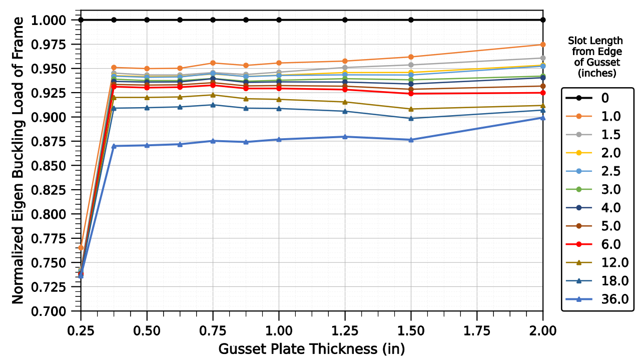
(c) Mode 1 Results HSS 1/4" Wall Thk.



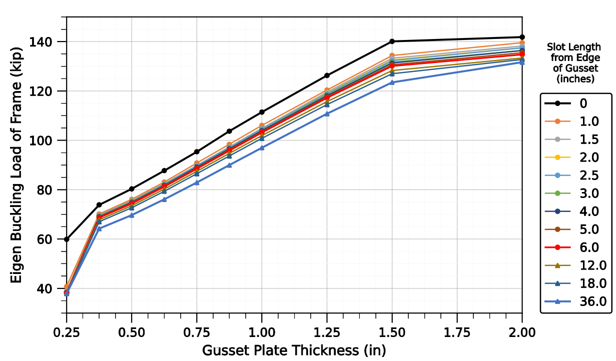
(d) Normalized Mode 1 Results HSS 1/4" Wall Thk.



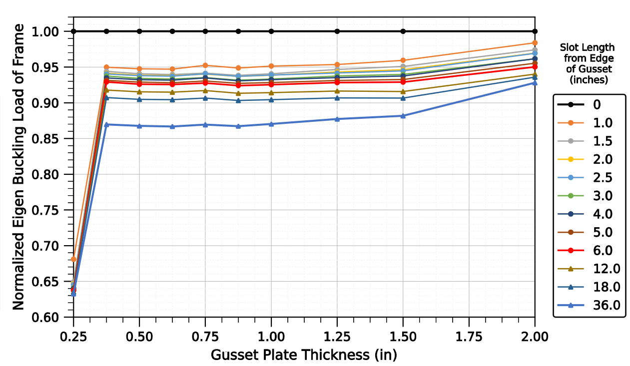
(e) Mode 1 Results HSS 3/8" Wall Thk.



(f) Normalized Mode 1 Results HSS 3/8" Wall Thk.

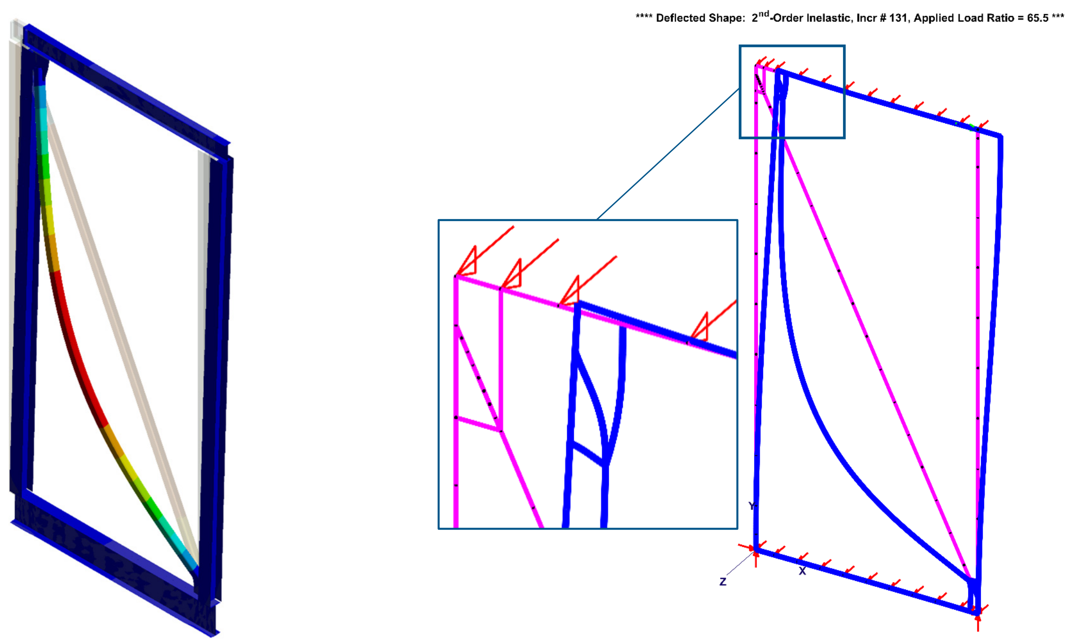


(g) Mode 1 Results HSS 1/2" Wall Thk.



(h) Normalized Mode 1 Results HSS 1/2" Wall Thk.

Figure 10: Single Brace Linear Eigenvalue Analysis Critical Buckling Results for Varying HSS Wall Thicknesses and Slot Lengths



(a) Nonlinear Buckling Load = 70.7 kip (b) Nonlinear Buckling Load = 65.5 kip

Figure 11: Comparison of 2nd Order Nonlinear Buckling Results for Frame Configuration with 1/2" Thk. Gusset Plate and HSS8x8x1/2 Bracing Member: (a) ANSYS Model, and (b) MASTAN2 Model

5.2 Case Study 2: X-Brace Configuration

For Case Study 2 a linear eigenvalue buckling analysis was run for an X-brace configuration. Various views of the FE model are shown in Fig. 12. Member sizes along with the significant parameters that were investigated are given in Tables 6 and 7, respectively. The model consisted of approximately 55,093 elements and 56,120 nodes. A total of 384 configurations were evaluated.

Table 6: Member Sizes

Member Description	Member Size
Beam Sections	Floor Beam: W33x118 | Roof Beam: W18x50
Column Section	W14x120
HSS Brace Size	8" x8" x (varying thicknesses)

Table 7: Parameters Investigated

Parameter Description	Parameters Evaluated
HSS Brace Wall Thickness (in.)	1/8, 1/4, 3/8, 1/2
Slot Length[1] (in.)	0, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 6.0, 12.0, 18.0, 36.0
Gusset Plate Thickness (in.)	0.50, 0.625, 0,75, 0.875, 1.00, 1.25, 1.50, 2.00

[1] Inches from edge of gusset plate

An isometric view showing a contour plot of member deformations for linear eigenvalue analysis critical (Mode 1) buckling results from the ANSYS model for the frame configuration with a 3/4 in. thick gusset plate, HSS8x8x1/2 brace, and no HSS brace slot extension is shown in Fig. 13(a). The deformation represents the mode shape, or eigenvectors, of the frame normalized

Model Information

Elements: ~55,093 Nodes: ~56,120

ANSYS Element Types:

SHELL181 for members SOLID185 for load plate CONTACT175 for connection of HSS to Gusset Plate and HSS Connection Plates at Midspan Intersection

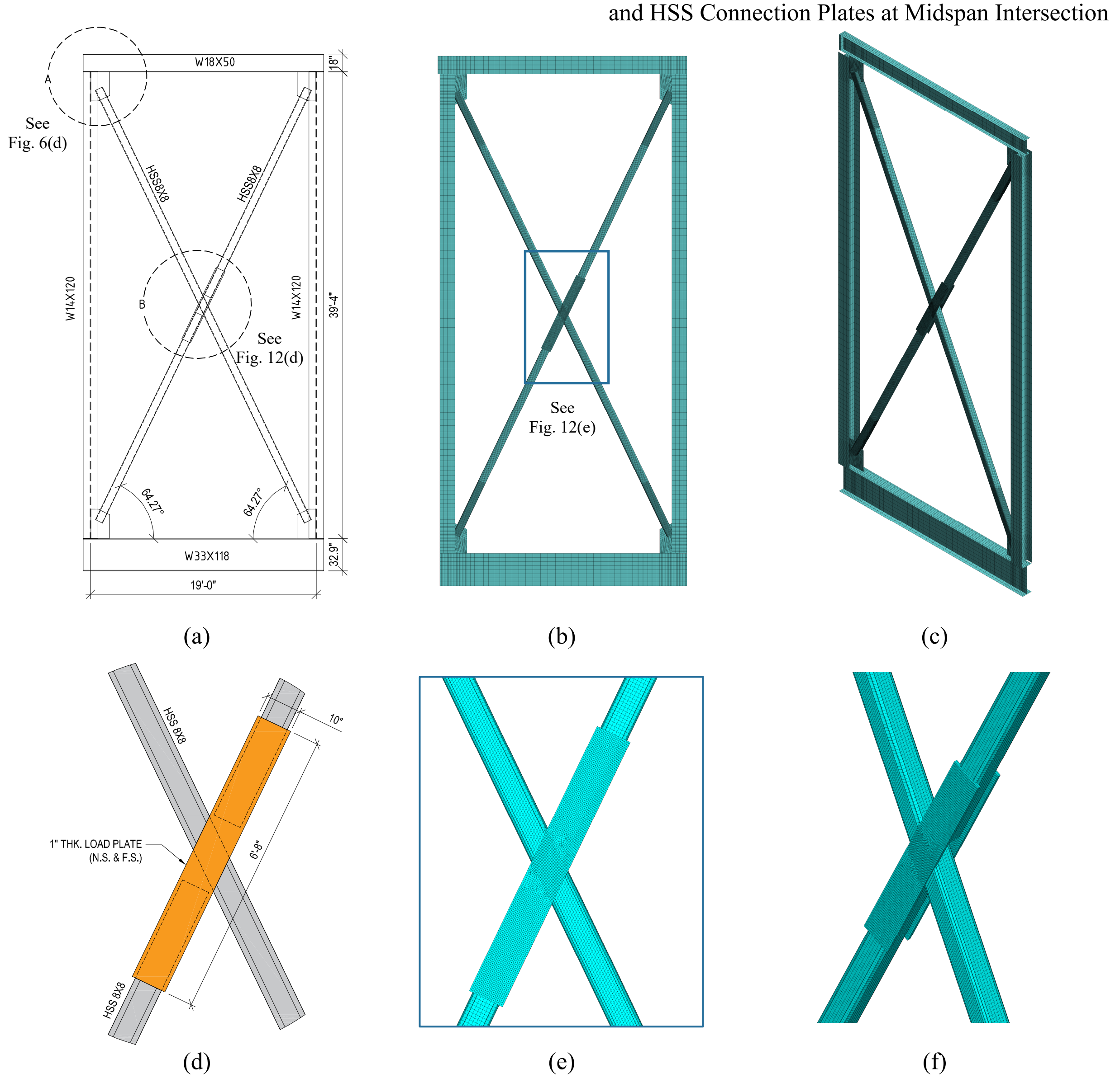
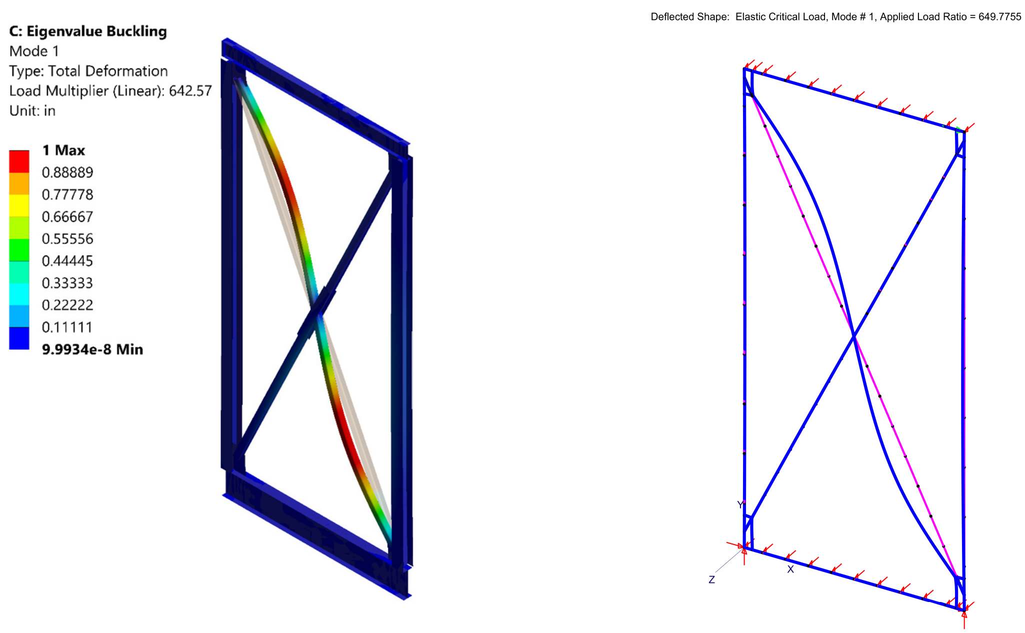


Figure 12: Model Geometry Used for Case Study 2: (a) X-Brace Elevation View; (b) FE Model Elevation View; (c) FE Model Isometric; (d) Details at X-Brace Midspan Connection; (e) FE Model Zoomed View at Midspan Connection Location; and (f) FE Model Zoomed View Isometric Showing Midspan Connection Details

to a value of unity. As shown in Fig. 13(a), the load multiplier obtained from the ANSYS shell element model was 642.57. Note that, similar to the single brace evaluated in Case Study 1, this and all of the other analysis results for the X-brace configurations are based on an external load of 1.0 kip applied to the load plate shown in Fig. 6(d) that was oriented along the axial direction of the top wide flange beam and, as with the previous analyses gravity loading was excluded. Since the magnitude of the load applied to the frame was 1.0 kip, this means the linear eigenvalue analysis critical buckling load is 642.6 kip. This process was repeated for all 384 configurations, which are plotted in Fig. 14.

A comparison of linear eigenvalue analysis critical buckling results from MASTAN2 for the same X-brace frame configuration is shown in Fig. 13(b). As shown in Fig. 13, the linear eigenvalue analysis critical buckling load from the ANSYS shell element model was 642.6 kip, compared with the MASTAN2 beam element model that had a critical buckling load of 649.8 kip. This equates to approximately 1.1% difference, which is reasonable given the considerably large computational expense of the much more complex shell element model.

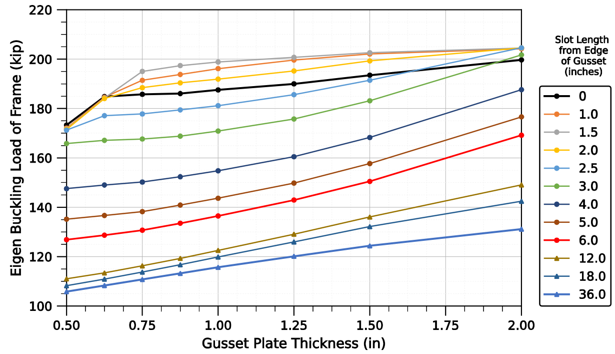


(a) ANSYS Result - Mode 1 Buckling Load = 642.6 kip (b) MASTAN2 Result - Mode 1 Buckling Load = 649.8 kip

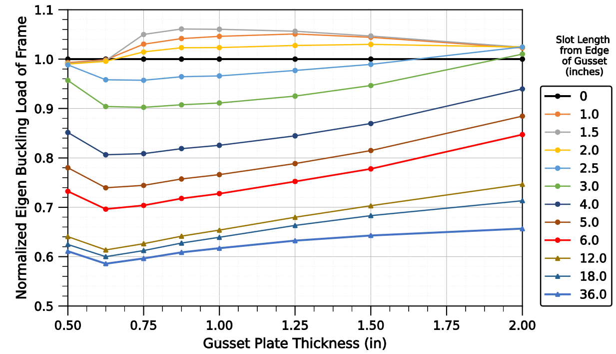
Figure 13: Comparison of Eigen Linear Elastic Critical Buckling Results for Frame Configuration with 3/4" Thk. Gusset Plate, HSS8x8x1/2 Bracing Member, and No Slot Extension: (a) ANSYS Model, and (b) MASTAN2 Model

The ANSYS shell element model linear eigenvalue analysis critical buckling results from all 384 configurations evaluated are plotted in Fig. 14. The results in Figs. 14(a, c, e, and g) are the raw results for critical buckling (Mode 1 from eigenvalue analysis). To compare the effect of extending the slot in the HSS brace, the results are normalized with respect to the case of no slot extension in the HSS brace and plotted in Figs. 14(b, d, f, and h). Observations and commentary on the plots are provided in Section 5.3.

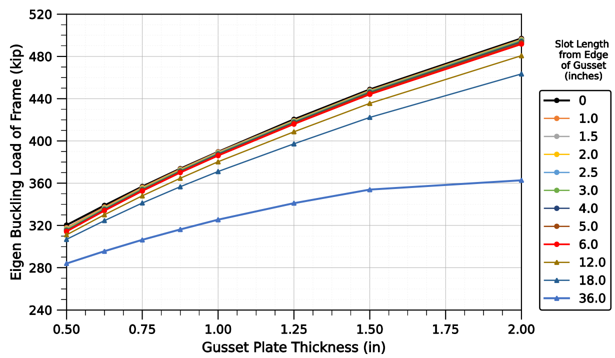
Fig. 15 shows the linear eigenvalue analysis critical buckling load results from the ANSYS shell element model for X-brace frame configurations with no HSS brace slot extension (i.e., the modeled slot extends only from the end of the HSS brace member to the edge of the gusset plate, where brace is welded to the gusset plate) for the four different HSS wall thicknesses evaluated. The results in Fig. 15 show the effect of only changing the gusset plate thickness and without having a slot extension in the HSS brace. These results show there is a linear increase in buckling capacity of the frame for HSS wall thickness values of 1/4 in., 3/8 in., and 1/2 in. as the gusset plate thickness is increased. However, for the case of HSS brace with 1/8 in. wall thickness the



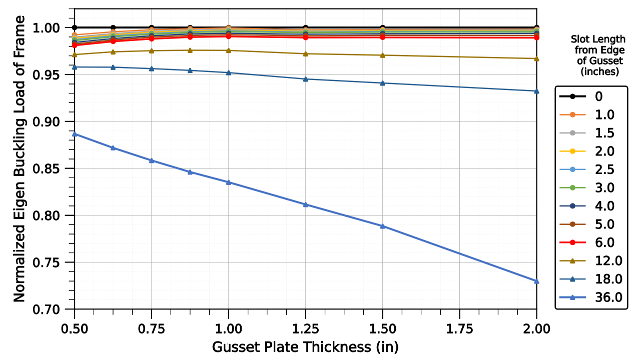
(a) Mode 1 Results HSS 1/8" Wall Thk.



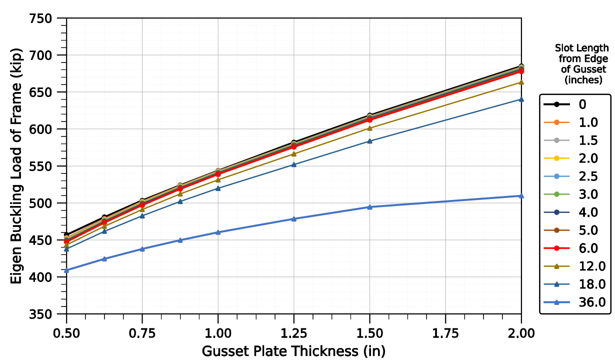
(b) Normalized Mode 1 Results HSS 1/8" Wall Thk.



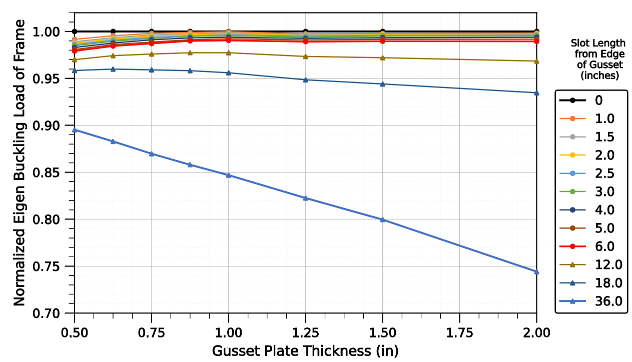
(c) Mode 1 Results HSS 1/4" Wall Thk.



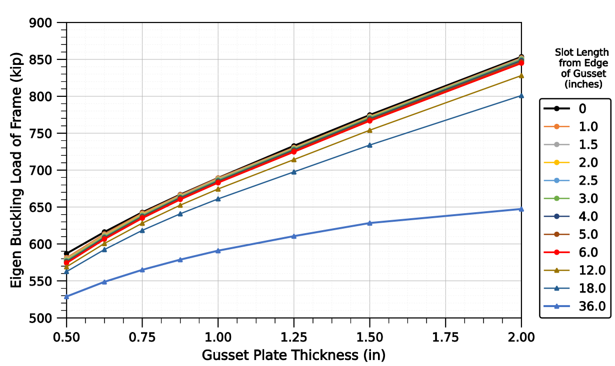
(d) Normalized Mode 1 Results HSS 1/4" Wall Thk.



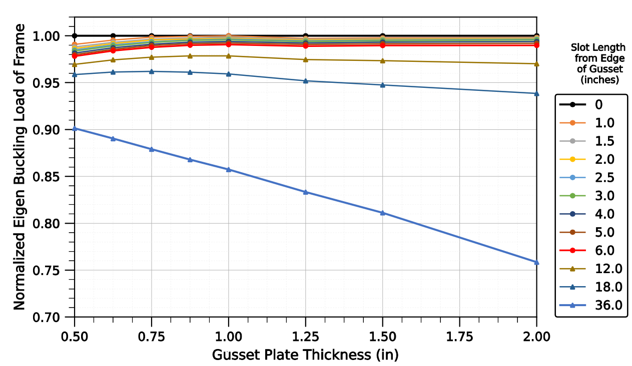
(e) Mode 1 Results HSS 3/8" Wall Thk.



(f) Normalized Mode 1 Results HSS 3/8" Wall Thk.



(g) Mode 1 Results HSS 1/2" Wall Thk.



(h) Normalized Mode 1 Results HSS 1/2" Wall Thk.

Figure 14: X-Brace Linear Eigen Buckling Analysis Critical Buckling Results for Varying HSS Wall Thicknesses and Slot Lengths

buckling capacity remains nearly constant beyond a plate thickness of approximately 5/8 in. This suggests that the HSS brace with a wall thickness of 1/8 in. is the controlling factor for buckling capacity, provided the gusset plate thickness is at least 5/8 in.

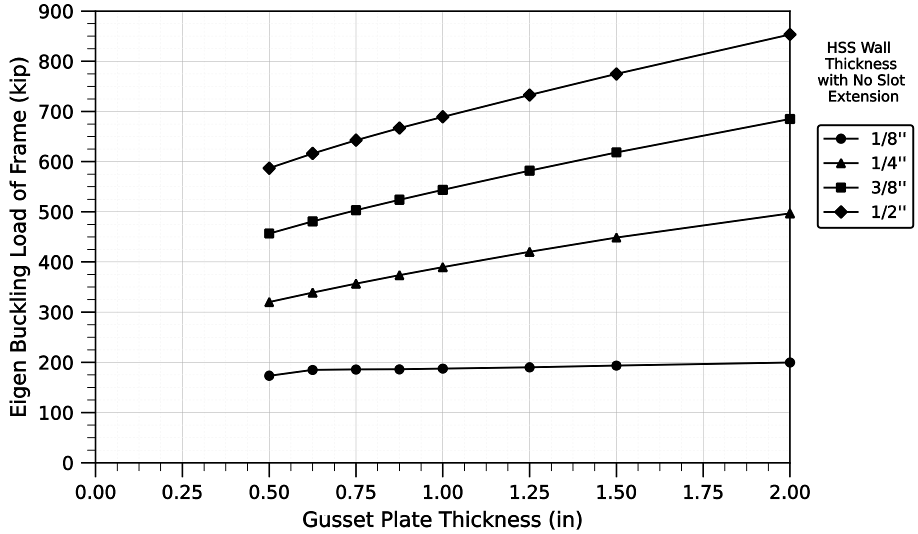


Figure 15: X-Brace Linear Eigenvalue Critical Buckling Results for Varying HSS Wall Thicknesses and Gusset Plate Thickness without Slot Extensions

5.3 Observations from Case Studies

5.3.1 Observations from Case Study 1

The gusset plate thickness was found to have a large effect on the overall braced frame lateral buckling capacity. The influence of gusset plate thickness on the lateral capacity of the frame is believed to be primarily attributed to the end rotational restraint provided by thicker plates, which enhances brace stability and lateral load resistance. However, this effect diminishes beyond a threshold thickness, which is likely because additional end restraint no longer improves brace buckling capacity. For braces with thinner HSS wall thicknesses, such as the 1/8 in. HSS wall thickness case studied, the influence of gusset plate thickness plays a reduced role. This is likely due to local buckling of the brace wall emerging as the dominant failure mode at larger gusset plate thicknesses. For thicker HSS walls in the range of 1/4 in. to 1/2 in. and gusset plate thicknesses between 1/2 in. and 1.0 in., buckling capacity exhibited a maximum variation of approximately 9% variation for slot lengths under 12 inches (see normalized results in Figs. 10(d, f, and h)).

Regarding the effects of HSS brace slot length extension overall, it was found that as the HSS wall thickness is increased, the effects of the slot length is reduced (i.e., HSS brace slot length effects are more pronounced for thinner HSS wall thicknesses). For the 1/8 in. HSS wall thickness case, results varied much greater than for the other cases studied. This is likely due to local buckling of the HSS cross-sectional wall for this case. Note that for this case, the width-to-thickness ratio (b/t) of the outstanding flange is (4 in. - 3/8 in.) / (1/8 in.) = 29, whereas the limiting b/t ratio provided by Table B4.1a of AISC 360 (2022) is $\boldsymbol{0.56}\sqrt{\boldsymbol{E}/\boldsymbol{F_y}}$ = 13.5. Also, for the 1/8 in. HSS wall thickness case, it appears that the slot length of 6.0 inches produced unexpected worst-case performance, potentially due to interactions between local and global buckling, suggesting a need for further investigation. For thicker HSS walls (3/8 in. to 1/2 in.), the longer brace slot lengths resulted in reduced brace capacity, likely due to the reduced cross-sectional properties of the slotted portion of the brace, resulting in a reduced overall buckling capacity. In effect, the brace becomes a stepped cross section with larger properties near midspan and reduced cross-sectional properties in slotted regions, resulting in weakened ends.

Preliminary validation of numerical models confirmed that a simplified beam element model in MASTAN2 can capture similar behavior and produce relatively similar results as from the more complicated ANSYS shell element model, particularly for gusset plate buckling, a critical aspect of braced frame performance. While promising, additional validation is required to fully establish the accuracy of beam element models in programs such as MASTAN2 for this application, which is the subject of future research.

5.3.2 Observations from Case Study 2

The X-brace configuration linear eigenvalue buckling capacity is more than four times the associated single brace linear eigenvalue buckling capacity. This is due to both the reduced brace length, as well as the additional rotational restraint provided by the midspan connection including heavy plate material. In the range of HSS wall thickness of 1/4 in. to 1/2 in., gusset plate thickness of 1/2 in. to 1.0 in., and slot lengths less than 12 in. the results have a maximum variation of approximately 3% (see normalized results in Figs. 14(d, f, and h)). This behavior, less pronounced than in single-brace systems, is attributable to the X-brace configuration, where slots are located only at beam-column connections, reducing their influence on buckling capacity. The midspan connection's stabilizing effect further mitigates the impact of slot dimensions. Therefore, the relative impact of the slot on the overall braced frame capacity is reduced. The 1.0 in. and 1.5 in. slot length cases exceed the 0 in. slot length case capacity for several gusset plate thicknesses. Although this is considered within the margin of error, it should be studied further.

5.3.3 General Modeling Observations from Both Case Studies 1 and 2

Based on Case Studies 1 and 2, the ANSYS model indicates that the gusset plate provides significantly more rotational stiffness than the Whitmore section alone. To replicate this behavior, the MASTAN2 model utilized an assembly of three beams to represent the gusset's rotational stiffness, aligning with the results observed in the ANSYS simulations. There are a number of rational approaches that may be used to approximate the gusset plate behavior. One potential practical approach for engineers involves reducing the brace length in the analysis model to reflect the physical member length, excluding the Whitmore section and beam-column panel zone. Additionally, the effective length factor (i.e., **K** Factor) can be adjusted by incorporating the flexural stiffness (**EI**/**L**), where **E** is the elastic modulus, **I** is the moment of inertia of the cross section, and **L** is the member length, of the Whitmore section to better approximate rotational stiffness.

When comparing results from the ANSYS model and hand methods, Case 1 (single diagonal) demonstrates that the nominal strength, $\mathbf{P_n}$, of the gusset plate is influenced by a 30-degree spread, with **L** defined as the distance from the brace end to the beam/column face. The ANSYS model provides an equivalent **K Factor** derived from eigenvalue buckling results, which can be compared against effective length nomographs using the rotational stiffness of the Whitmore section. These comparisons offer valuable insights into accurately modeling gusset plate behavior under buckling conditions. Example results comparing equivalent **K Factor** derived from the ANSYS models with manual methods is summarized in Table 8. These results are for the case of 1/2 in. HSS wall thickness, with 0 in. slot length extension and varying gusset plate thicknesses.

Table 8: Comparison of ANSYS Results with Manual Calculation Methods Using Nomographs

Gusset Plate Thickness (in.)	ANSYS Brace Axial Load (kip)	Equivalent K Factor from ANSYS*	Equivalent K Factor from Nomographs**
0.250	135.4	1.06	1.00
0.375	166.9	0.95	0.99
0.500	181.5	0.91	0.97
0.625	198.3	0.87	0.94
0.750	215.5	0.84	0.91
0.875	234.4	0.80	0.87
1.000	251.9	0.77	0.82
1.250	285.4	0.73	0.74
1.500	316.6	0.69	0.67

*Equivalent K Factor from ANSYS value is effective length to achieve the ANSYS brace load value
**EI / L girder is based on gusset Whitmore section with L = 0.5 (Whitmore length); EI / L column using brace properties
Results are based on elastic (Euler) buckling capacity

Key observations highlight that if the gusset plate is adequately sized, further increases in rotational stiffness provide limited benefits. However, based on results from the single brace case study evaluated in this paper, it appears that as the ratio of the gusset cross-sectional out-of-plane rotational stiffness to the brace out-of-plane rotational stiffness, $\mathbf{(EI / L)_{gusset} / (EI / L)_{brace}}$, falls below approximately 60, the likelihood of gusset plate buckling becomes a critical design concern. This underscores the importance of balancing gusset plate dimensions to ensure sufficient stability without introducing buckling vulnerabilities.

6. Conclusions and Future Work

This paper provided an overview of common industrial building lateral frame systems with a focus on HSS braces with welded slotted gusset plate end connections. Additionally, it provided common modifications to ease construction or fit-up issues that may occur in the field. One such common field change is to extend the HSS slot length to fit the HSS brace within the building frame. This paper also provided an overview of design processes, with a focus on the interaction between the Engineer of Record, the Fabricator, and the Erector. It also provided Code of Standard Practice requirements for detailing which sets a framework for the contractual requirements between each of these parties.

The paper also performed analytical studies of an example industrial building braced frame which used a slotted HSS brace connection in its lateral system where the Erector extended the HSS slot length to allow for fit-up. Two models were studied: one single HSS brace configuration, and a X-brace HSS brace configuration. Both an ANSYS (2022) shell element model and a MASTAN2 (2024) beam element model were developed to study the impacts of the extended HSS slot length on the overall braced frame behavior.

The studies suggest that the extended HSS slot length does impact overall braced frame behavior by reducing capacity on the order of 5% for common brace sizes (HSS 8x8 with 1/4 in. to 1/2 in. wall thickness) and gusset sizes (1/2 in. to 1.0 in. thick). Additionally, for thinner HSS wall sizes (1/8 in. in the study), the overall braced frame capacity is significantly reduced due to local buckling of the cross section at the extended slot portion. The studies also found that the gusset plate thickness provides a substantial impact to the overall braced frame capacity due to the rotational restraint provided by the gusset plate end connection. Finally, it was determined that simpler beam models can capture comparable results to more complex models if they can properly account for end conditions.

Future research should explore a broader range of braced frame geometries to enhance understanding of factors affecting buckling capacity and overall performance. Key areas for investigation include variations in member sizes, bay width and height, and connection geometries. By examining these parameters, a more comprehensive framework for evaluating diverse structural configurations can be established, contributing to more versatile and optimized design strategies.

The scope of nonlinear analysis should also be expanded to investigate the load-deflection behavior of braced frames with extended HSS slots. Particular emphasis should be placed on cyclic loading to assess the seismic implications of these configurations. Comparing the results with design requirements in AISC 341 (2022), which prioritizes tensile rupture of the brace, will provide critical insights into the influence of extended slots on seismic performance. Such studies are essential for determining the suitability of extended slots in seismic design and their impact on the energy dissipation and resilience of bracing systems.

Additionally, the impact of extended HSS slots on gusset plate buckling behavior warrants further investigation, particularly concerning potential reductions in the Whitmore section buckling capacity. Evaluating this relationship will refine design criteria for gusset plates, ensuring adequate resistance to buckling in systems with extended slots. Finally, the validity of using an effective brace length shorter than the physical length in non-seismic applications, such as wind design, should be assessed. This approach, while potentially cost-effective, must be carefully evaluated to confirm its impact on frame stability and overall safety in non-cyclic loading scenarios.

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