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Frame stability implications: when the EOR specifies HSS vertical bracing slotted end connections

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Abstract
This research extends previous research by the authors concerning the use of HSS members for vertical bracing. In a typical slotted HSS brace connection which attaches to a gusset plate, the braces are slotted on each end for ease of erection so that they can be field welded to a gusset plate that has been shop welded to the column. In practice though, there are numerous situations where the slot length in the HSS members specified by the Engineer of Record (EOR) is not sufficient to erect the brace. This is most common when the braced bay has already been completed, and the HSS brace must be fit into the bay and slotted ends positioned over the gusset plates on either end. To resolve the issue, the Erector may extend the HSS slot without consulting the EOR. In these cases, there is an end length of the HSS that is 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, reduced buckling capacity of the gusset plate, and/or reduced buckling capacity of the brace member. This paper presents results from a parametric study using finite element models generated in ANSYS that are used to simulate the behavior of the HSS brace end connection configuration. Several of the following parameters are evaluated: HSS brace slot length beyond the edge of the gusset plate; HSS brace size and wall thickness; gusset plate geometry and properties; and overall frame height-to-width ratios. Results from a finite element parametric study are presented that demonstrate the effects of the various parameters described. The paper concludes with design specification recommendations for AISC 360 and AISC 341 as appropriate.

1. Introduction – Stability, Force Resistance, and Member Requirements

Common concentric braced framing configurations include cross-brace, inverted V-brace, and single diagonal brace systems. For heavy industrial type buildings, it is more common to use a single diagonal bracing member to provide stability against lateral forces associated with wind and seismic loading. The single diagonal brace creates a rigid, stable structure that resists racking and lateral movement. Unlike cross-bracing, which uses two intersecting members, this system relies on a single, strong diagonal member that must be designed to resist both tension and compression forces in the vertical plane of the frame. When a lateral force comes from one direction, the brace is in tension. When the force comes from the opposite direction, the same

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brace is in compression. Because it must handle both types of forces, this single diagonal member is typically a larger, heavier steel section comprised of two axes of symmetry. Also, this type of design has the flexibility to allow the placement of openings in the frame where the use of cross-bracing would obstruct openings, such as doors or windows. It is a simple and efficient way to transfer lateral forces through the structure. In this single diagonal braced frame system, one of the most versatile member types utilized is a Hollow Structural Section (HSS) member.

Once the type of bracing member is selected the next part of the design is to determine how it is to be connected to the frame. 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). One of the more common connection methods is the slotted HSS end plate connection, where a slot is cut in opposite HSS walls at each member end 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 typically 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.

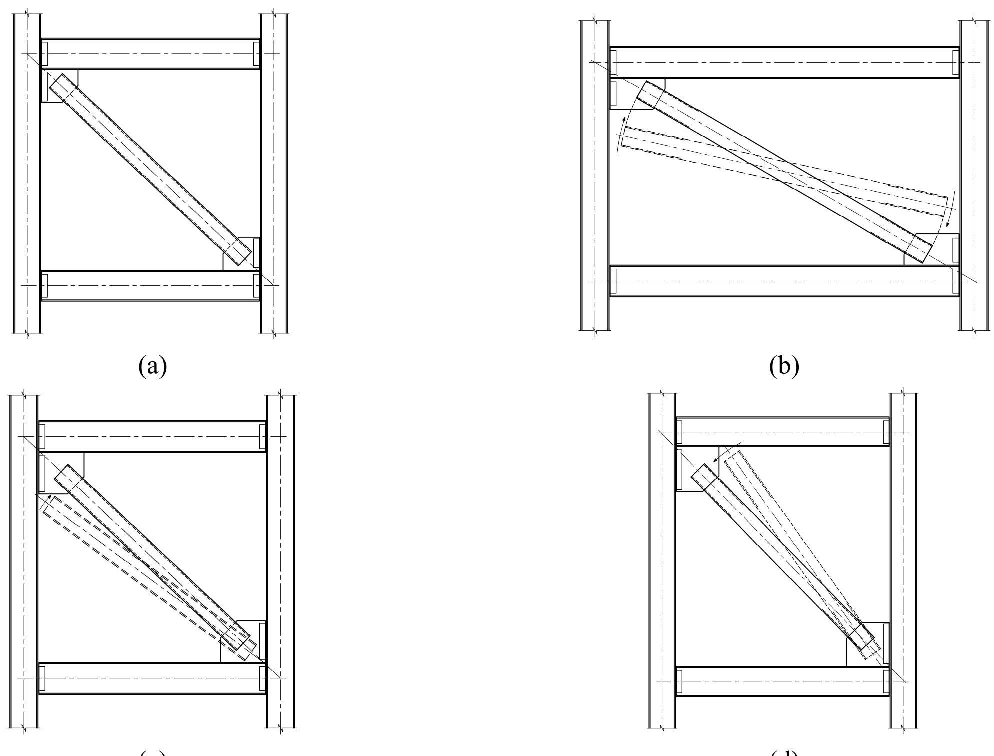


Figure 1: Erection Plans for Slotted Vertical HSS Braces (Dowswell and Lini 2023)

Slots are typically fabricated with a 1/2 to 1 in. (13 to 25 mm) minimum clearance beyond the edge of the gusset plate and even at that length the gap is usually inadequate for the required erection clearance; therefore, many fabricators prefer to use a clearance of 2 to 3 in. (51 to 76 mm). Even an extended slot length of 3 in. (76 mm) may not allow the brace to be maneuvered into its final position. Therefore, the slot length should be carefully evaluated by the detailer.

The concern with the use of HSS members for vertical bracing is the Erector's ability to install them. 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.

Thornton and Fortney (2012) noted that the required clearance dimension can be as high as 6 in. (152 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. 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.

This paper takes a detailed look at the various parameters associated with using slotted end HSS braces attached to gusset plates completing the braced bay of a structural steel frame. The following parameters are evaluated to determine which have a significant influence on the overall frame capacity based on linear elastic eigenvalue buckling results: gusset plate thickness, HSS brace size, HSS brace wall thickness, HSS member's slotted end length beyond the gusset plate, and structural steel frame geometry, i.e., beam-to-column aspect ratio. While the above parameters are varied the following have been held constant throughout all the analyses: Structural steel frame members' material properties, Structural steel frame members' beam-to-column relative flexural-to-axial stiffness ratio, beam, column, and HSS members' material properties, HSS brace member's buckling length, and HSS member's attachment length onto the gusset plate. Results from over two thousand five hundred finite element analyses will be presented that demonstrate the effects of various parameters described.

2. Background – Impetus for Continued Study

"Should the EOR be concerned when detailing and fabrication differ from analysis, modeling, and design?" was one of the overarching themes in previous research conducted by Reigles et al. (2025). Before trying to answer that question, three areas were examined starting with an informal history of braced frame design. Four distinct research periods were identified that examined vertical bracing systems, but not specifically HSS being used for the bracing members. They were: First Period, pre-1980; Second Period, 1980 to 1994 (pre-Northridge, CA earthquake); Third Period, 1994 (post-Northridge, CA earthquake) to 2010; and Fourth Period, 2011 to present.

If one is to study the behavior of HSS braced frames and their connections as outlined above, it is the authors' opinion that the following reference documents should be used as the basis for any ongoing research: **Hollow Structural Sections Connections Manual** (AISC 1997), AISC Steel Design Guide 24 (Packer and Olson 2024), and AISC Steel Design Guide 29 (Muir and Thornton 2014). The Connections Manual, when published, was considered a significant milestone in the design of HSS members. It showed the various ways these members could be connected to a structural steel frame. One type, the slotted end HSS bracing member/gusset plate connection, was illustrated for its versatility in both axial tension and compression, but the Connections Manual stops short of considering the potential exposed gap between the end of the gusset plate and the overall slot length provided in the HSS member.

The second and third areas that were previously examined looked at how the current overall work process is carried out from the Design Engineer, Detailer, Fabricator, to the Erector along with defining who has the ultimate overall Responsibility for Design. It should be clarified that the conclusions drawn from that previous research were based on a work process used by some design firms in the past and may not reflect the overall design engineering community.

The previous paper summarized what is found in ANSI/AISC 303-22, Code of Standard Practice (COSP) for Steel Buildings and Bridges (2022) and is reiterated here.

> "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."

Communication between all parties is paramount to ensuring that any method 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*.

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-up properly, and constructability is a consideration, the bracing elements have been altered or modified in the field without the knowledge or consent of the EOR. More succinctly, and the primary point focus of this research paper is to answer the following question: Is it acceptable for the Erector to increase the slot length provided by the Fabricator that is not sufficiently long enough to allow the rectangular HSS bracing members to be placed in a partially erected structural steel frame (Dowswell and Lini 2023)?

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 square 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 (Reigles et al. 2025).

In summary, ongoing research in the use of HSS continues in several areas from their general use in framed building systems (Packer 2006; Ziemian 2010), as HSS compression or bracing members (Saucedo 2007; Unterweger and Taras 2013), HSS to gusset plate bracing connections (Muir 2008; Roeder et al. 2011; Thornton 1991; Thornton and Lini 2011; Dranger and Thornton 2025), evaluation of effective length and shear lag factors (Dowswell 2012, 2021), and performance of HSS in braced frames under cyclic or seismic loading (Hsiao 2012; Sabelli et al. 2013; McCormick 2017; Thornton and Fortney 2012).

3. Finite Element Buckling Capacity Analysis of Braced Frame

To evaluate the compressive strength of a square HSS brace member in a single concentrically braced frame, and to quantify the effects of various parameters on the frame buckling capacity, a parametric finite element linear elastic eigenvalue buckling analysis approach was employed.

3.1 Finite Element Buckling Capacity Analysis Methodology

3.1.1 Finite Element Linear Elastic Eigenvalue Buckling Analysis

Linear elastic eigenvalue buckling analysis is a computational finite element analysis method that solves the eigenvalue problem used to predict the Euler theoretical critical buckling load that causes instability. The solution of the eigenvalue problem represents analysis of an ideal linear elastic structure under axial compressive, where the buckling load is treated as an eigenvalue, and the corresponding buckling modes are the eigenvectors of the system. The governing equation for the linear elastic buckling eigenvalue problem is shown in Eqn. 1.

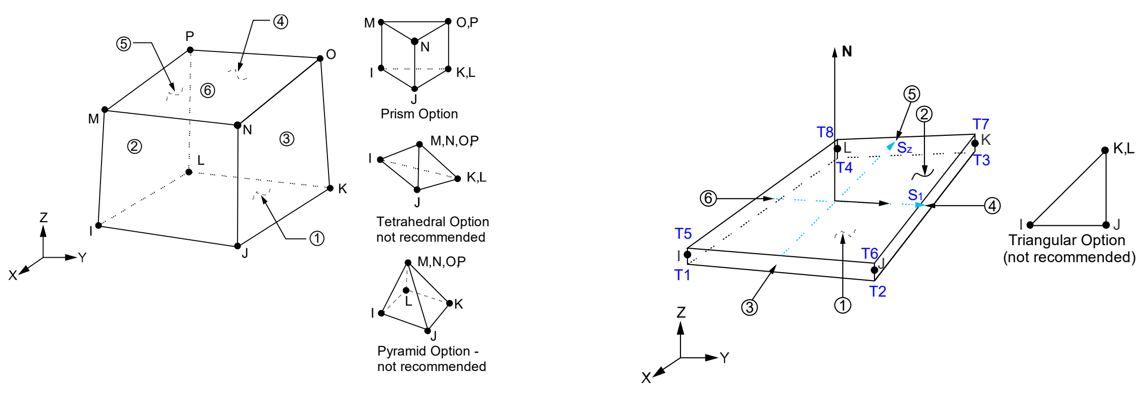
$$([\boldsymbol{K}] + \lambda_i[\mathrm{S}])\{\psi_i\} = \{0\} \tag{1}$$

where $\boldsymbol{K}$ is the structural stiffness matrix corresponding to the initial state, $\boldsymbol{S}$ is the additional stress-stiffening matrix (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 (BLF), λ_i, for the *i*-th mode (i.e., eigenvalues are the buckling load multipliers). The BLF of a given mode is the factor multiplied with the externally applied load that causes the frame to buckle. The mode shape from the eigenvalue analysis represents the displaced shape of the frame when the frame buckles at a corresponding load multiplier. The applied load is typically selected to have a unit value (e.g., 1 kip) such that the buckling load is greater in magnitude than the applied load, which results in a BLF greater than 1.0.

In this analysis, the structure is subjected to a combination of applied forces, and the BLF is determined by solving the linearized equilibrium equations. The critical buckling load (critical BLF) corresponds to the first eigenvalue of the system, with each subsequent eigenvalue representing higher-order buckling modes. This analysis approach is primarily applicable to thinwalled structures where the initial deformation due to buckling is assumed to be small relative to the structure's overall dimensions. Finite element model linear elastic eigenvalue buckling analysis is a linear analysis using linear elastic material properties that predicts idealized buckling of the modeled geometry. Thus, it provides an idealized, and often overly optimistic, approximation of the critical buckling load rather than a realistic prediction that includes nonlinear behavior. However, despite its limitations, it still serves as an essential tool for preliminary design and structural stability assessments.

3.1.2 Shell Element FE Model Used in ANSYS

The commercial finite element software ANSYS, Release 2022 R2 (ANSYS 2022) was employed to perform the finite element linear elastic eigenvalue buckling analysis of the single bay frames. ANSYS is a general-purpose FE software that has a broad range of capabilities to perform linear and nonlinear static and dynamic analysis in the time and frequency domains. ANSYS solid element SOLID185 was used to model the loading block, which is a 3-D, 8-node solid element. The loading block was used to apply distributed loading to the frame to avoid localized effects due to concentrated loading. The SOLID185 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 formulation supports plasticity, stress stiffening, large deflection, large strain, as well as other capabilities, which can be found in the ANSYS Element Library documentation. It allows for prism, tetrahedral, and pyramid degenerations when used in irregular geometry regions. Fig. 2(a) shows the geometry of the SOLID185 element used for the FE analysis.



(a) (b)

Figure 2: ANSYS Elements: (a) SOLID185 Element Geometry (b) SHELL181 Element Geometry

ANSYS shell element SHELL181 (see Fig. 2(b)) was used to model the steel wide flange shapes for the beams and columns, the HSS brace members, and gusset plates. The mesh for the structure is shown in Section 4.4. The mesh size of the shell elements is refined in regions 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, user defined material, stress stiffening, large deflection, large strain, as well as other capabilities, which can be found in the ANSYS Element Library documentation.

3.2 Finite Element Analysis Assumptions and Limitations

Model Parameters: The geometry of the structural frame that was modeled was assumed to be taken from one bay of a multilevel braced frame industrial type building, with sizes being based on a previously constructed project. The one bay frame of the building that was modeled consisted of two wide flange steel columns, two wide flange steel beams, and a single HSS diagonal brace element in between. The end connections of the HSS brace element were attached to two gusset plates, one being located at the upper left beam-to-column connection and the other at the lower right beam-to-column connection that made up the frame. The columns were assumed to be continuous, and the beams were framed into the columns. The columns were oriented so that the beams with their attached gusset plate were connected to the weak axis of the column. The configurations that were studied are shown in more detail in Figs. 3, 4, and 5. The angle of the brace with respect to the horizontal axis was selected at 30, 45 and 60 degrees to assess a lower bound, average, and upper bound on the brace angle to capture the effects of brace angle within typical ranges.

Material Properties: All the models evaluated used linear elastic stress-strain material properties. It is understood that linear elastic eigenvalue buckling analysis overpredicts buckling capacity, and that nonlinear buckling analysis that uses nonlinear material properties provides more realistic results. However, because of the computational effort of analyzing a more detailed shell element based FE model, linear material models and linear buckling analysis approaches were used so more configurations and parameters could be studied. The focus of this study was not so much the magnitude of buckling capacity for a particular configuration, but rather the change in buckling capacity for various configurations. The material models used for the wide flange shapes for the beams and columns are based on ASTM A992 (2022), which has a yield strength of 50 ksi and ultimate strength of 65 ksi. The gusset plates are modeled with material properties based on ASTM A572 (2023), Grade 50, which has a yield strength of 50 ksi and ultimate strength of 65 ksi. The HSS members are modeled with material properties of ASTM A500 (2023), Grade C, which has a yield strength of 50 ksi and ultimate strength of 62 ksi.

Boundary Conditions: Several different configurations for boundary conditions were considered. The final configuration selected consisted of boundary conditions being applied at both the bottom and top of the "idealized" frame. For all the configurations evaluated the "idealized" frame included the following features: 1) left and right columns both oriented in the weak direction, 2) top and bottom beams framed into the web of each column as a welded connection, and 3) for the three configurations of brace angle, i.e., TALL, SHORT, and SQUARE, the wide flange members for the beams and columns were selected so that the frames' beam-column flexural stiffness was kept approximately constant. More details on the various configurations are provided in Section 4. The bottom of the left column had fixed boundary conditions (translation and rotation are restrained in all three degrees-of-freedom). The bottom of the right column had a roller joint that allowed translation along the axial horizontal axis of the frame (colinear with direction of the applied force) and only allowed rotation about the horizontal axis perpendicular to the axial direction of the frame. Roller joint boundary conditions were also applied to the top of the left and right columns to prevent out-of-plane translation, which allowed the frames themselves to translate along the axial direction of the frame and aimed to prevent the frame from twisting or moving out-of-plane. These were considered to be a reasonable approximation of the installed condition where in-fill beams and diaphragm action of the floor/roof provide out-of-plane restraint.

4. Braced Frame Models – ANSYS Shell Model Test Matrix

Three braced frame configurations were considered that are labeled as "TALL", "SHORT" and "SQUARE" and are described in detail in Sections 4.1, 4.2, and 4.3, respectively.

4.1 TALL Braced Frame Configuration – Models 1, 4, 7; 34'-6" x 20'-0"

The TALL Braced Frame Configuration is comprised of W18x50 top/bottom beams and W14x145 left/right columns. The beams have a nominal length, L_b, 240 in., moment of inertia, I_x, 800 in.^{4}, and flexural stiffness, EI/L, 96,666.67 in.^{4}. The columns have a nominal length, L_c, 414 in., moment of inertia, I_y, 677 in.^{4}, and flexural stiffness, EI/L, 47,422.71 in.^{4}. Therefore, the frame beam-column flexural stiffness ratio is 2.04. Table 1 provides the relevant data for the various vertical bracing members and Fig. 3 shows the braced frame layout and gusset plate geometry used in the parametric studies to be conducted with this configuration.

Table 1: TALL Frame Model Configurations – Models 1, 4, and 7

Model No.	**Configuration Name**	**Frame/Bay Dimensions (H x L)**	**Brace Member**	**Brace Area** (in.^{2})	**Brace Length** (in.)	**Brace Axial Stiffness AE/L** (kip/in.)
Model 1	TALL	34'-6" x 20'-0"	HSS 8x8x5/8	16.40	382	1245.03
Model 1	TALL	34'-6" x 20'-0"	HSS 8x8x1/2	13.50	382	1024.87
Model 1	TALL	34'-6" x 20'-0"	HSS 8x8x3/8	10.40	382	789.53
Model 1	TALL	34'-6" x 20'-0"	HSS 8x8x3/16	5.37	382	407.67
Model 4	TALL	34'-6" x 20'-0"	HSS 6x6x5/8	16.40	382	1245.03
Model 4	TALL	34'-6" x 20'-0"	HSS 6x6x1/2	13.50	382	1024.87
Model 4	TALL	34'-6" x 20'-0"	HSS 6x6x3/8	10.40	382	789.53
Model 4	TALL	34'-6" x 20'-0"	HSS 6x6x3/16	5.37	382	407.67
Model 7	TALL	34'-6" x 20'-0"	HSS 10x10x5/8	16.40	382	1245.03
Model 7	TALL	34'-6" x 20'-0"	HSS 10x10x1/2	13.50	382	1024.87
Model 7	TALL	34'-6" x 20'-0"	HSS 10x10x3/8	10.40	382	789.53
Model 7	TALL	34'-6" x 20'-0"	HSS 10x10x3/16	5.37	382	407.67

4.2 SHORT Braced Frame Configuration – Models 2, 5, 8; 20'-0" x 34'-6"

The SHORT Braced Frame Configuration is comprised of W18x97 top/bottom beams and W14x120 left/right columns. The beams have a nominal length, L_b, 414 in., moment of inertia, I_x,1750 in.^{4}, and flexural stiffness, EI/L, 122,584.54 in.^{4}. The columns have a nominal length, L_c,240 in., moment of inertia, I_y, 495 in.^{4}, and flexural stiffness, EI/L, 59,812.50 in.^{4}. Therefore, the frame beam-column flexural stiffness ratio is 2.05. Table 2 provides the relevant data for the various vertical bracing members and Fig. 4 shows the braced frame layout and gusset plate geometry used in the parametric studies to be conducted with this configuration.

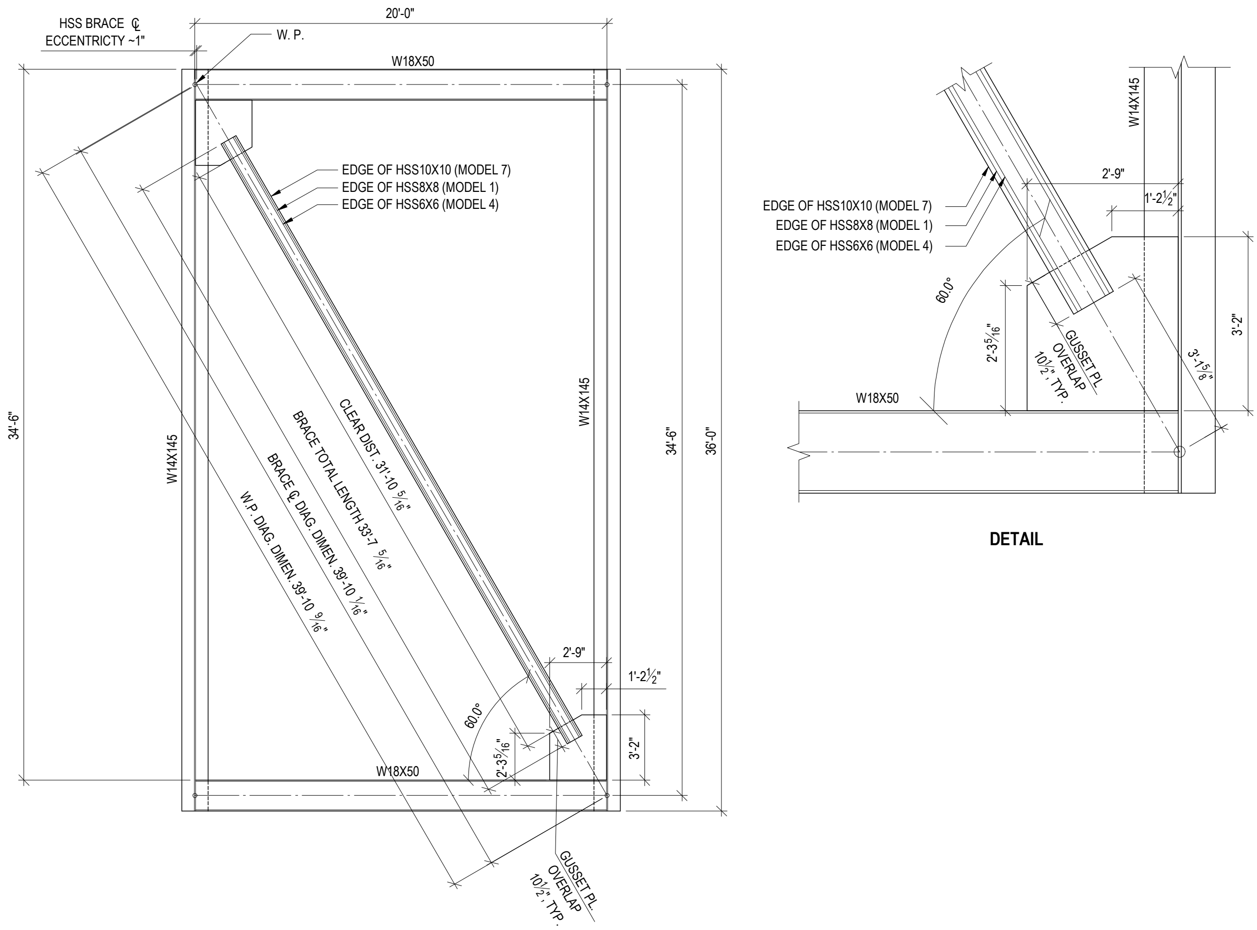


Figure 3: TALL Braced Frame and Gusset Plate Detailed Geometry

Table 2: SHORT Frame Model Configurations – Models 2, 5, and 8

Model No.	Configuration Name	Frame/Bay Dimensions (H x L)	Brace Member	Brace Area (in.^{2})	Brace Length (in.)	Brace Axial Stiffness AE/L (kip/in.)
Model 2	SHORT	20'-0" x 34'-6"	HSS 8x8x5/8	16.40	382	1245.03
Model 2	SHORT	20'-0" x 34'-6"	HSS 8x8x1/2	13.50	382	1024.87
Model 2	SHORT	20'-0" x 34'-6"	HSS 8x8x3/8	10.40	382	789.53
Model 2	SHORT	20'-0" x 34'-6"	HSS 8x8x3/16	5.37	382	407.67
Model 5	SHORT	20'-0" x 34'-6"	HSS 6x6x5/8	16.40	382	1245.03
Model 5	SHORT	20'-0" x 34'-6"	HSS 6x6x1/2	13.50	382	1024.87
Model 5	SHORT	20'-0" x 34'-6"	HSS 6x6x3/8	10.40	382	789.53
Model 5	SHORT	20'-0" x 34'-6"	HSS 6x6x3/16	5.37	382	407.67
Model 8	SHORT	20'-0" x 34'-6"	HSS 10x10x5/8	16.40	382	1245.03
Model 8	SHORT	20'-0" x 34'-6"	HSS 10x10x1/2	13.50	382	1024.87
Model 8	SHORT	20'-0" x 34'-6"	HSS 10x10x3/8	10.40	382	789.53
Model 8	SHORT	20'-0" x 34'-6"	HSS 10x10x3/16	5.37	382	407.67

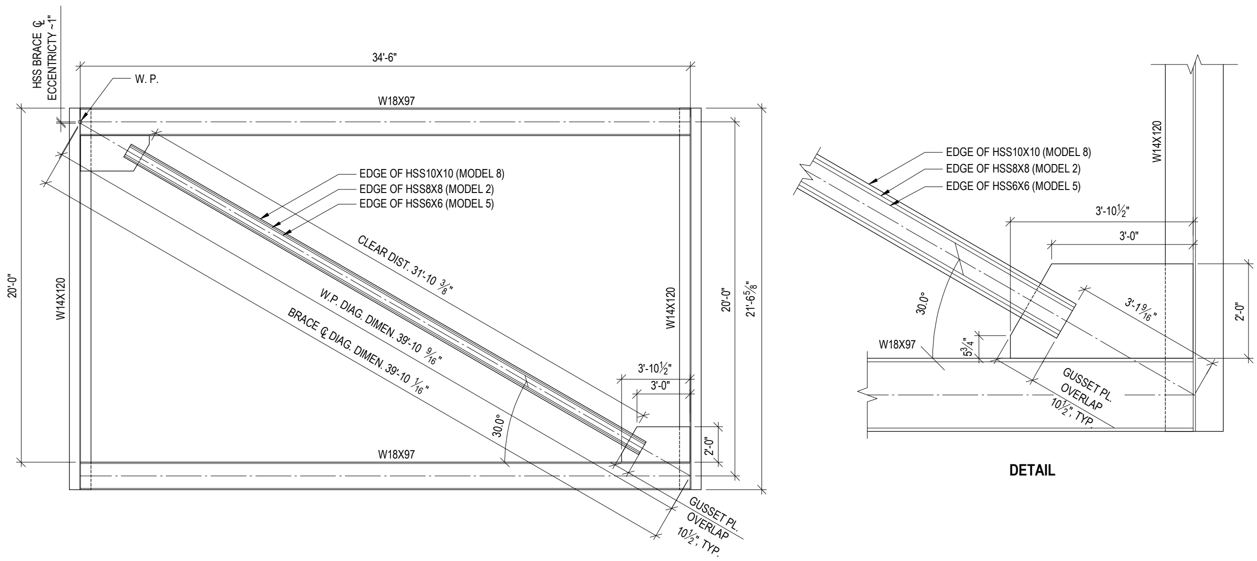


Figure 4: SHORT Braced Frame and Gusset Plate Detailed Geometry

4.3 SQUARE Braced Frame Configuration – Models 3, 6, 9; 28'-3" x 28'-3"

The SQUARE Braced Frame Configuration is comprised of W18x46 top/bottom beams and W14x90 left/right columns. The beams have a nominal length, L_b, 339 in., moment of inertia, I_x, 712 in.^{4}, and flexural stiffness, EI/L, 60,908.55 in.^{4}. The columns have a nominal length, L_c, 339 in., moment of inertia, I_y, 362 in.^{4}, and flexural stiffness, EI/L, 30,967.55 in.^{4}. Therefore, the frame beam-column flexural stiffness ratio is 1.97. Table 3 provides the relevant data for the various vertical bracing members and Fig. 5 shows the braced frame layout and gusset plate geometry used in the parametric studies to be conducted with this configuration.

Table 3: SQUARE Frame Model Configurations – Models 3, 6, and 9

Model No.	**Configuration Name**	**Frame/Bay Dimensions (H x L)**	**Brace Member**	**Brace Area** (in.^{2})	**Brace Length** (in.)	**Brace Axial Stiffness AE/L** (kip/in.)
Model 3	SQUARE	28'-3" x 28'-3"	HSS 8x8x5/8	16.40	382	1245.03
Model 3	SQUARE	28'-3" x 28'-3"	HSS 8x8x1/2	13.50	382	1024.87
Model 3	SQUARE	28'-3" x 28'-3"	HSS 8x8x3/8	10.40	382	789.53
Model 3	SQUARE	28'-3" x 28'-3"	HSS 8x8x3/16	5.37	382	407.67
Model 6	SQUARE	28'-3" x 28'-3"	HSS 6x6x5/8	16.40	382	1245.03
Model 6	SQUARE	28'-3" x 28'-3"	HSS 6x6x1/2	13.50	382	1024.87
Model 6	SQUARE	28'-3" x 28'-3"	HSS 6x6x3/8	10.40	382	789.53
Model 6	SQUARE	28'-3" x 28'-3"	HSS 6x6x3/16	5.37	382	407.67
Model 9	SQUARE	28'-3" x 28'-3"	HSS 10x10x5/8	16.40	382	1245.03
Model 9	SQUARE	28'-3" x 28'-3"	HSS 10x10x1/2	13.50	382	1024.87
Model 9	SQUARE	28'-3" x 28'-3"	HSS 10x10x3/8	10.40	382	789.53
Model 9	SQUARE	28'-3" x 28'-3"	HSS 10x10x3/16	5.37	382	407.67

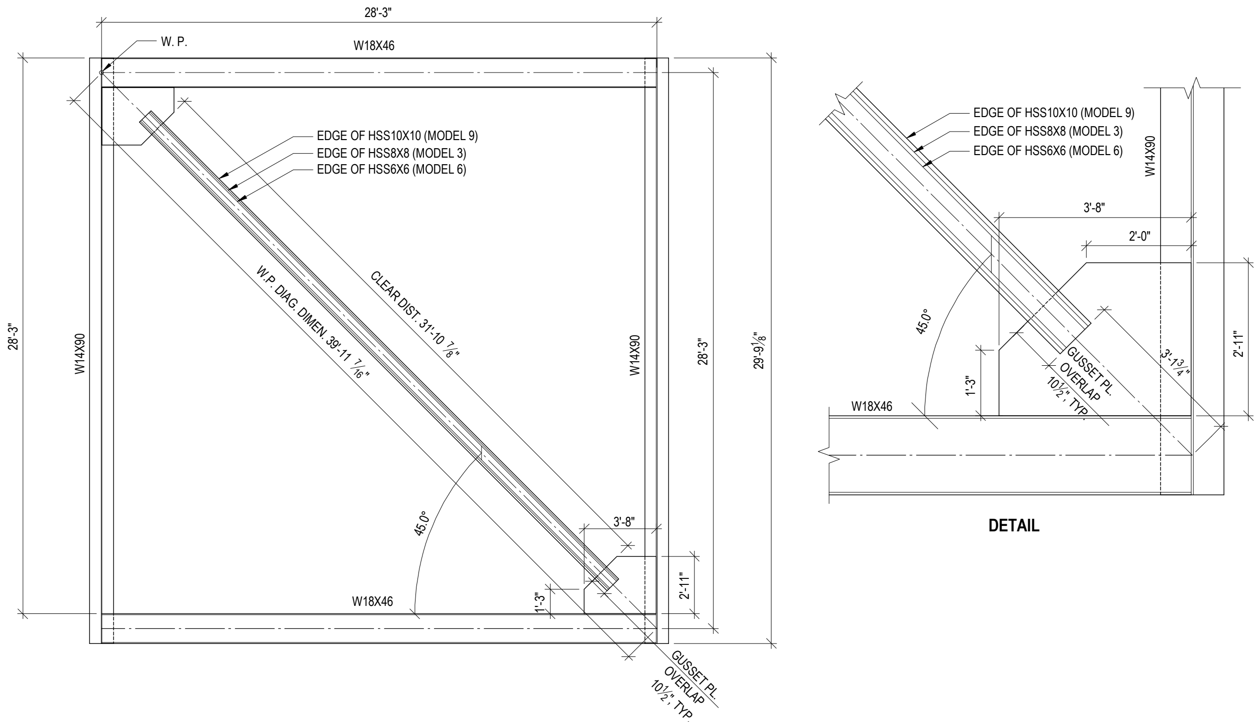


Figure 5: SQUARE Braced Frame and Gusset Plate Detailed Geometry

4.4 ANSYS Models - Descriptions

Figs. 6, 7, and 8 depict the finite element mesh configurations for the braced frames comprised of beams, columns, braces, and gusset plates shown in Figs. 3, 4, and 5, respectively. The mesh attributes for the 9 models are described in Table 4 for the TALL FE model configurations (Models 1, 4, and 7), SHORT FE model configurations (Models 2, 5, and 8), and SQUARE FE model configurations (Models 3, 6, and 9).

Table 4: Mesh Attributes for Models 1 through 9

Configuration Name	Model No.	Approximate No. Elements*	Approximate No. Nodes
TALL	Model 1	19538	20059
	Model 4	16688	17199
	Model 7	22289	22817
SHORT	Model 2	20308	20859
	Model 5	17524	18067
	Model 8	23647	23092
SQUARE	Model 3	19406	19919
	Model 6	16622	17127
	Model 9	22166	22684

*SHELL181 elements used for beams, columns, gusset plates and HSS brace members, SOLID185 elements used for load plate

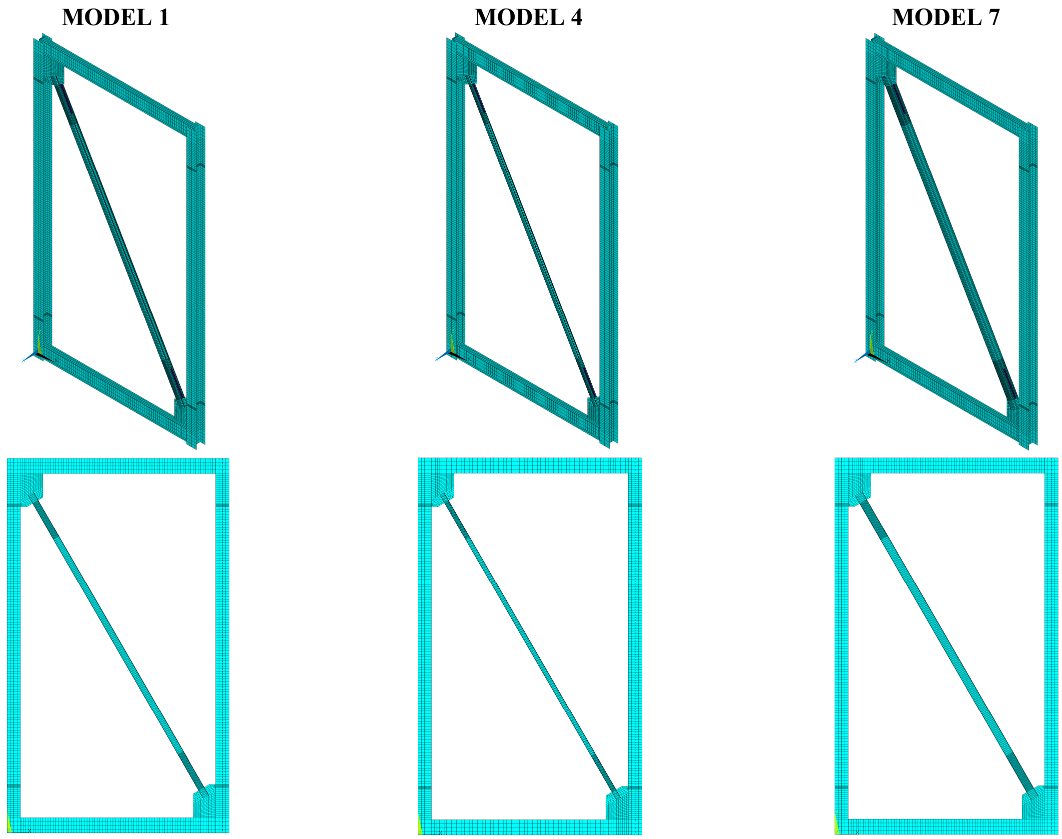


Figure 6: Mesh of TALL Frame Model Configurations – Models 1, 4, and 7

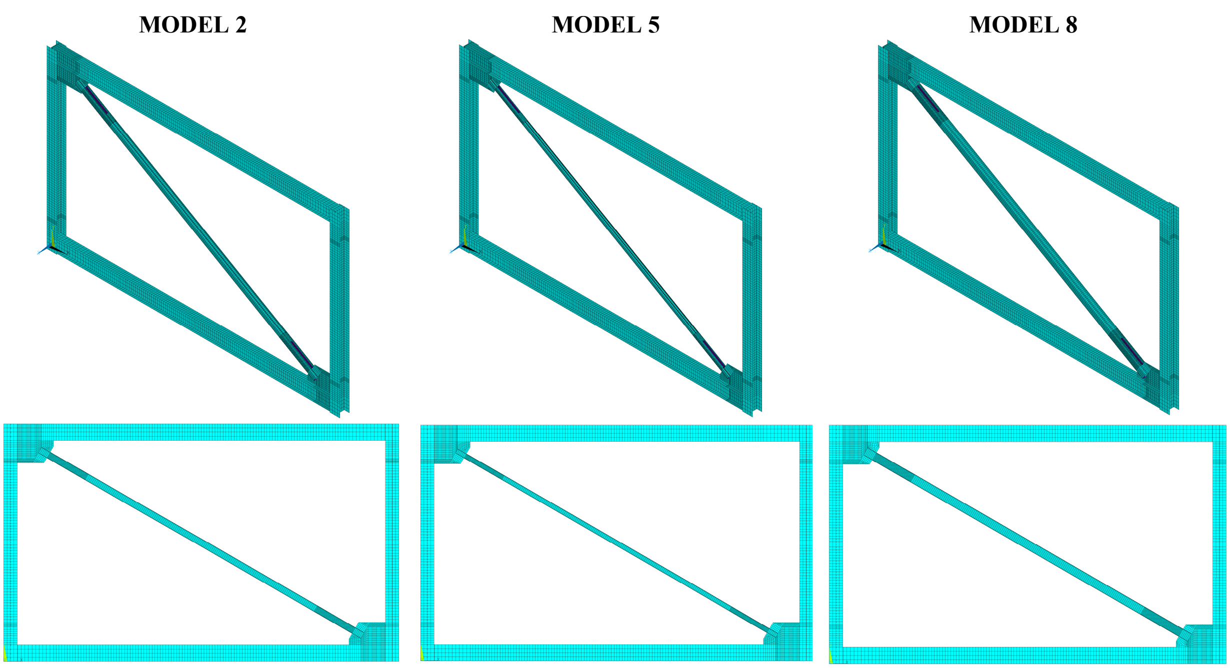


Figure 7: Mesh of SHORT Frame Model Configurations – Models 2, 5, and 8

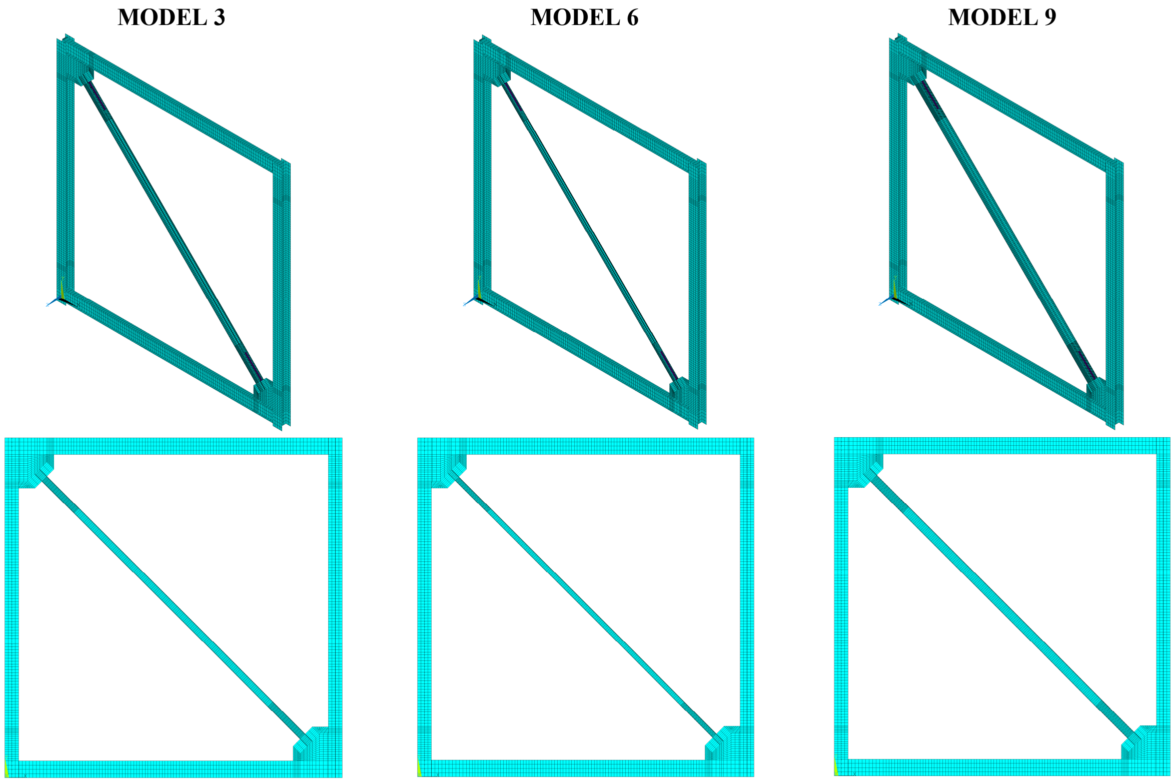


Figure 8: Mesh of SQUARE Frame Model Configurations – Models 3, 6, and 9

5. ANSYS Braced Frame Shell Element Model Results

For a bracing system to be effective all components need to be strong enough as well as stiff enough to ensure proper system behavior. What was found is that if the gusset plate is not stiff enough it will buckle prematurely, not allowing the HSS bracing member to act properly, i.e., buckle out-of-plane in the frame which it was placed. Gusset plate buckling occurs irrespective of the slotted-end gap provided. This condition was observed when the gusset plate geometry was reduced to 1/4 in. thickness. The 9 models included in the initial analytical test matrix that were founded on these thin gusset plates all showed similar responses and therefore were separated and removed from the rest of the braced frame model results described in Section 5.1. The result shown in Fig. 9 for Model 1 (TALL), with a HSS 8x8x1/2 brace member, a 1/4 in. thick gusset plate and 0 in. slot length has a Buckling Load Factor (BLF) = 21.385 kip. This deformation plot is typical for all cases when the bracing element is connected to a thin gusset plate that buckles.

In an effort to provide more context to these model braced frames with gusset plates (Thornton 1984, Dowswell 2006, Thornton and Lini 2011), their Whitmore section lengths, L1, L2, L3, and average effective lengths, L_{avg}, are given in Fig. 10(a) – (c) for the TALL, SHORT, and SQUARE structural frame geometries, respectively. The ASTM A572 Gr. 50 (2023) material properties and Whitmore Section geometries specifically for the 1/4 in. gusset plates used in the analyses can be found in Table 5. The nominal compressive strengths for these thin gusset plates are calculated for the TALL, SHORT, and SQUARE structural frame geometries and various sizes of HSS bracing members, respectively in Tables 6, 7, and 8. Two values are presented in each table based on whether the effective length factor, K, should be taken as K = 1.0 or

K = 0.65. Previous research has indicated that since the gusset plates in this study are connected to the beam as well as the web of the column that there is some minimal to significant restraint provided leading to the range of effective lengths. Nevertheless, the compressive strengths of these thin gusset plates are much lower than what would have been determined if the HSS bracing member had elastically buckled instead.

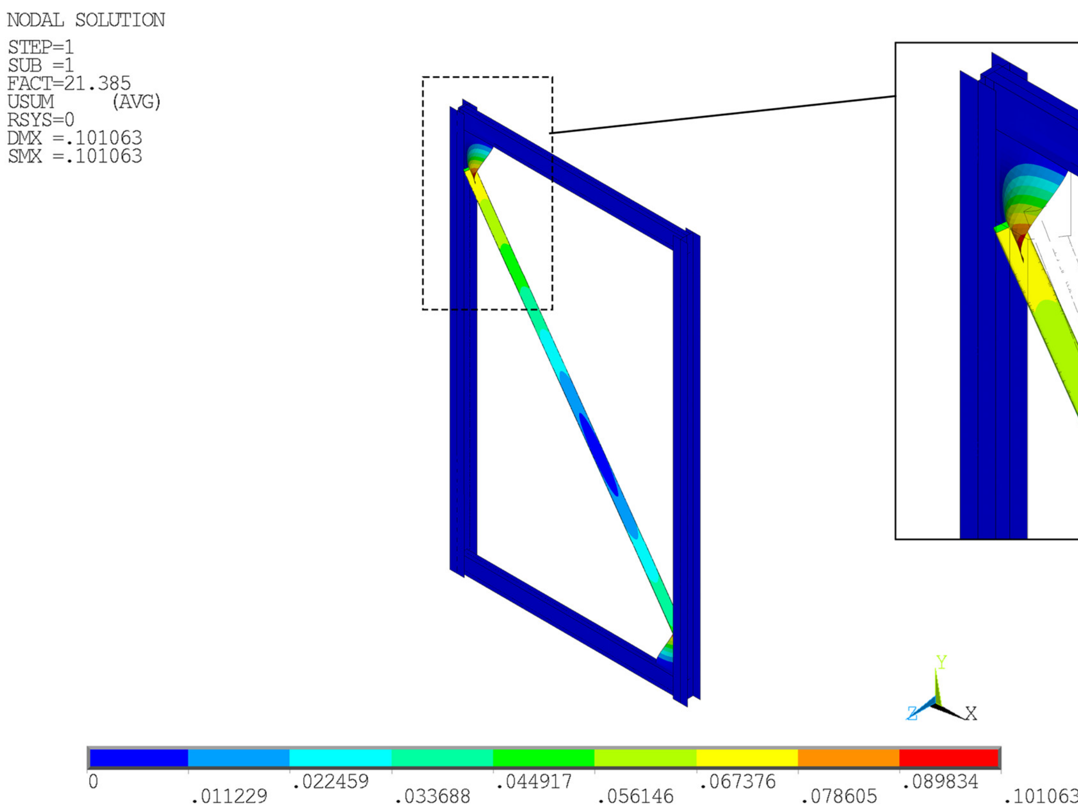
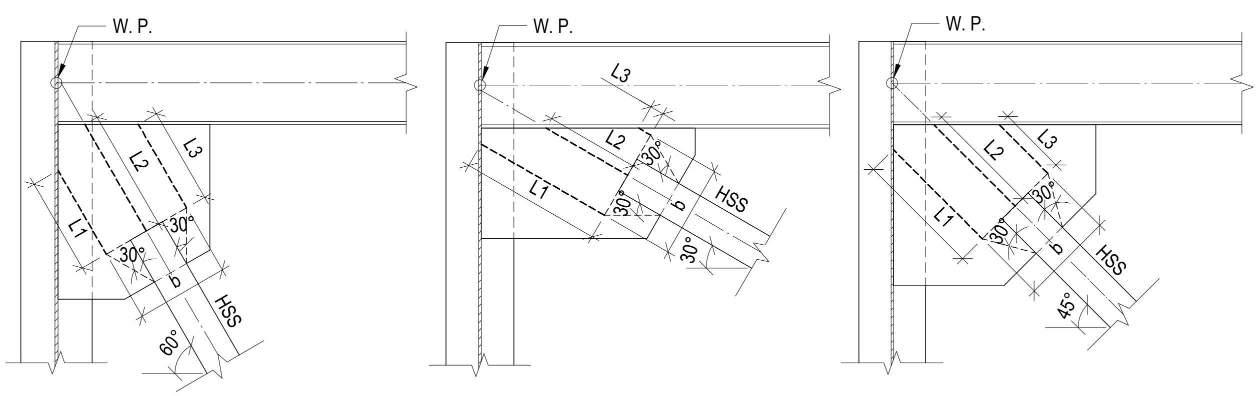


Figure 9: Thin Gusset Plate Buckling Before HSS Member



(a) TALL Frame: Models 1, 4, and 7

(b) SHORT Frame: Models 2, 5, and 8

(c) SQUARE Frame: Models 3, 6, and 9

Figure 10: Gusset Plate Whitmore Section Lengths for Various Model Configurations

Table 5: Common Gusset Plate Material and Whitmore Section Geometric Properties

HSS 8X8 Brace			HSS 6X6 Brace			HSS 10X10 Brace		
Gusset Plate Material Properties								
$E =$	29000	ksi	$E =$	29000	ksi	$E =$	29000	ksi
$F_y =$	50	ksi	$F_y =$	50	ksi	$F_y =$	50	ksi
$4.71(E/F_y)^{0.5} =$	113.43		$4.71(E/F_y)^{0.5} =$	113.43		$4.71(E/F_y)^{0.5} =$	113.43	
Gusset Plate Whitmore Section Geometric Properties								
$b =$	20.12	in.	$b =$	18.12	in.	$b =$	22.12	in.
$t =$	0.25	in.	$t =$	0.25	in.	$t =$	0.25	in.
$A =$	5.03	in.^{2}	$A =$	4.53	in.^{2}	$A =$	5.53	in.^{2}
$I =$	0.03	in.^{4}	$I =$	0.02	in.^{4}	$I =$	0.03	in.^{4}
$r =$	0.07	in.	$r =$	0.07	in.	$r =$	0.07	in.

Table 6: TALL Frame Whitmore Section Thin Gusset Plate Compressive Strength

HSS 8X8 Brace			HSS 6X6 Brace			HSS 10X10 Brace		
Whitmore Section Lengths (in.)								
L1	**L2**	**L3**	**L1**	**L2**	**L3**	**L1**	**L2**	**L3**
20.98	26.74	20.93	22.71	26.74	21.51	19.25	26.74	20.36
$L_{avg} = L_c =$	22.89	in.	$L_{avg} = L_c =$	23.66	in.	$L_{avg} = L_c =$	22.12	in.
Gusset Plate Whitmore Section Slenderness, $KL_{avg} = L_c$, where K = 1.0								
$L_c/r =$	317.17		$L_c/r =$	327.84		$L_c/r =$	306.50	
Gusset Plate Nominal Compressive Strength								
$F_e =$	2.85	ksi	$F_e =$	2.66	ksi	$F_e =$	3.05	ksi
$F_{cr} =$	2.50	ksi	$F_{cr} =$	2.34	ksi	$F_{cr} =$	2.67	ksi
$P_n =$	12.55	kip	$P_n =$	10.58	kip	$P_n =$	14.78	kip
Gusset Plate Whitmore Section Slenderness, $KL_{avg} = L_c$, where K = 0.65								
$L_c/r =$	206.16		$L_c/r =$	213.10		$L_c/r =$	199.23	
Gusset Plate Nominal Compressive Strength								
$F_e =$	6.73	ksi	$F_e =$	6.30	ksi	$F_e =$	7.21	ksi
$F_{cr} =$	5.91	ksi	$F_{cr} =$	5.53	ksi	$F_{cr} =$	6.32	ksi
$P_n =$	29.72	kip	$P_n =$	25.05	kip	$P_n =$	34.95	kip

Table 7: SHORT Frame Whitmore Section Thin Gusset Plate Compressive Strength

HSS 8X8 Brace			HSS 6X6 Brace			HSS 10X10 Brace		
Whitmore Section Lengths (in.)								
L1	**L2**	**L3**	**L1**	**L2**	**L3**	**L1**	**L2**	**L3**
30.93	20.44	3.01	31.51	20.44	4.74	30.36	20.44	1.28
$L_{avg} = L_c =$	18.13	in.	$L_{avg} = L_c =$	18.90	in.	$L_{avg} = L_c =$	17.36	in.
Gusset Plate Whitmore Section Slenderness, $KL_{avg} = L_c$, where K = 1.0								
$L_c/r =$	251.22		$L_c/r =$	261.89		$L_c/r =$	240.55	
Gusset Plate Nominal Compressive Strength								
$F_e =$	4.54	ksi	$F_e =$	4.17	ksi	$F_e =$	4.95	ksi
$F_{cr} =$	3.98	ksi	$F_{cr} =$	3.66	ksi	$F_{cr} =$	4.34	ksi
$P_n =$	20.01	kip	$P_n =$	16.58	kip	$P_n =$	23.99	kip
Gusset Plate Whitmore Section Slenderness, $KL_{avg} = L_c$, where K = 0.65								
$L_c/r =$	163.29		$L_c/r =$	170.23		$L_c/r =$	156.36	
Gusset Plate Nominal Compressive Strength								
$F_e =$	10.73	ksi	$F_e =$	9.88	ksi	$F_e =$	11.71	ksi
$F_{cr} =$	9.41	ksi	$F_{cr} =$	8.66	ksi	$F_{cr} =$	10.27	ksi
$P_n =$	47.33	kip	$P_n =$	39.23	kip	$P_n =$	56.79	kip

Table 8: SQUARE Frame Whitmore Section Thin Gusset Plate Compressive Strength

HSS 8X8 Brace			HSS 6X6 Brace			HSS 10X10 Brace		
Whitmore Section Lengths (in.)								
L1	**L2**	**L3**	**L1**	**L2**	**L3**	**L1**	**L2**	**L3**
27.40	24.98	14.91	28.40	24.98	15.91	26.40	24.98	13.91
$L_{avg} = L_c =$	22.43	in.	$L_{avg} = L_c =$	23.10	in.	$L_{avg} = L_c =$	21.76	in.
Gusset Plate Whitmore Section Slenderness, $KL_{avg} = L_c$, where K = 1.0								
$L_c/r =$ 310.80			$L_c/r =$ 320.08			$L_c/r =$ 301.52		
Gusset Plate Nominal Compressive Strength								
$F_e =$	2.96	ksi	$F_e =$	2.79	ksi	$F_e =$	3.15	ksi
$F_{cr} =$	2.60	ksi	$F_{cr} =$	2.45	ksi	$F_{cr} =$	2.76	ksi
$P_n =$	13.07	kip	$P_n =$	11.10	kip	$P_n =$	15.27	kip
Gusset Plate Whitmore Section Slenderness, $KL_{avg} = L_c$, where K = 0.65								
$L_c/r =$ 202.02			$L_c/r =$ 208.05			$L_c/r =$ 195.99		
Gusset Plate Nominal Compressive Strength								
$F_e =$	7.01	ksi	$F_e =$	6.61	ksi	$F_e =$	7.45	ksi
$F_{cr} =$	6.15	ksi	$F_{cr} =$	5.80	ksi	$F_{cr} =$	6.54	ksi
$P_n =$	30.93	kip	$P_n =$	26.27	kip	$P_n =$	36.17	kip

5.1 General Observations

5.1.1 Observations from TALL Models 1, 4, 7

The critical linear elastic eigenvalue buckling load of frames for the TALL configuration Models 1, 4, and 7 with the varying HSS wall thicknesses are summarized in Fig. 11. When comparing the 12 plots the following observations can be made:

The range of gusset plate thickness for all plots was 0.5 in. to 1.50 in. The range of slot length from edge of gusset plate for all plots was 0.0 in. to 36 in.

- Fig. 11(a) – (d) covered bracing members HSS8x8x3/16 to HSS8x8x5/8:
  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased almost linearly from 53 kip to 97 kip (t = 3/16 in.); 89 kip to 141 kip (t = 3/8 in.); 110 kip to 166 kip (t = 1/2 in.); and 125 kip to 185 kip (t = 5/8 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was reduced with respect to the 0.0 in. slot length results almost linearly from 52 kip to 91 kip (t = 3/16 in.); 87 kip to 132 kip (t = 3/8 in.); 107 kip to 155 kip (t = 1/2 in.); and 122 kip to 173 kip (t = 5/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from less than 2% to greater than 6%.
- Fig. 11(e) – (h) covered bracing members HSS6x6x3/16 to HSS6x6x5/8:
  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased almost linearly from 25 kip to 55 kip (t = 3/16 in.); 38 kip to 78 kip (t = 3/8 in.); 46 kip to 89 kip (t = 1/2 in.); and 51 kip to 96 kip (t = 5/8 in.) when a 0.0 in. slot length was maintained;

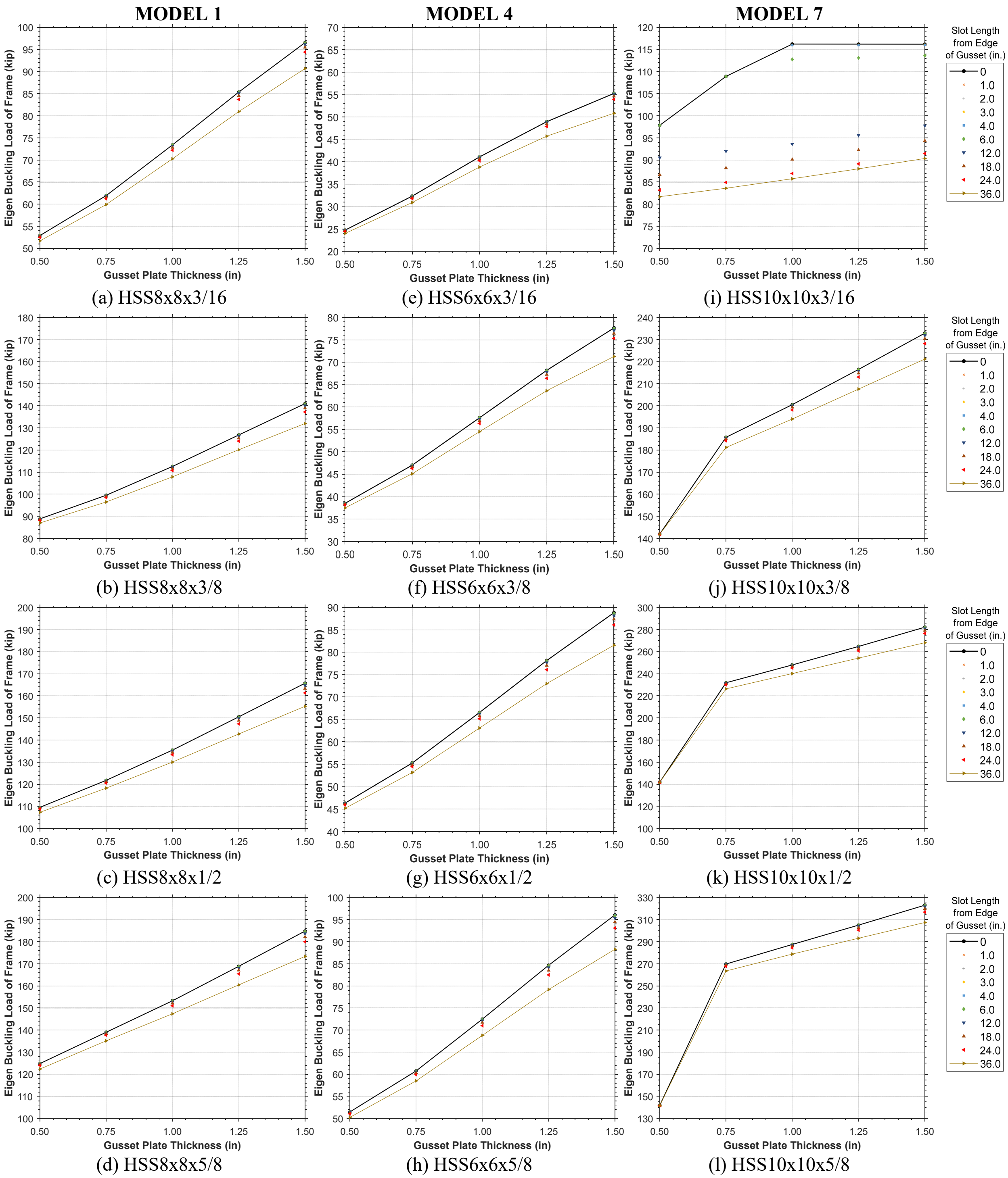


Figure 11: Linear Elastic Eigenvalue Buckling Results for Models 1, 4, and 7 with Varying Wall Thickness

- As the slot length was increased over the given range, the buckling load of the frame geometry was reduced with respect to the 0.0 in. slot length results almost linearly from 24 kip to 51 kip (t = 3/16 in.); 37 kip to 71 kip (t = 3/8 in.); 45 kip to 82 kip (t = 1/2 in.); and 50 kip to 80 kip (t = 5/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from less than 3% to greater than 8%.

- Fig. 11(i) – (l) covered bracing members HSS10x10x3/16 to HSS10x10x5/8 and will be divided into two parts: HSS10x10x3/16 and HSS10x10x3/8 to HSS10x10x5/8 since their behaviors are very different than the preceding two bracing member sizes:

  For bracing member HSS10x10x3/16

  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry no longer increased linearly as it went from 98 kip to 116 kip (t = 3/16 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was drastically reduced, though increased linearly from 82 kip to 90 kip (t = 3/16 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from 16% to 22%.

  For bracing members HSS10x10x3/8 to HSS10x10x5/8

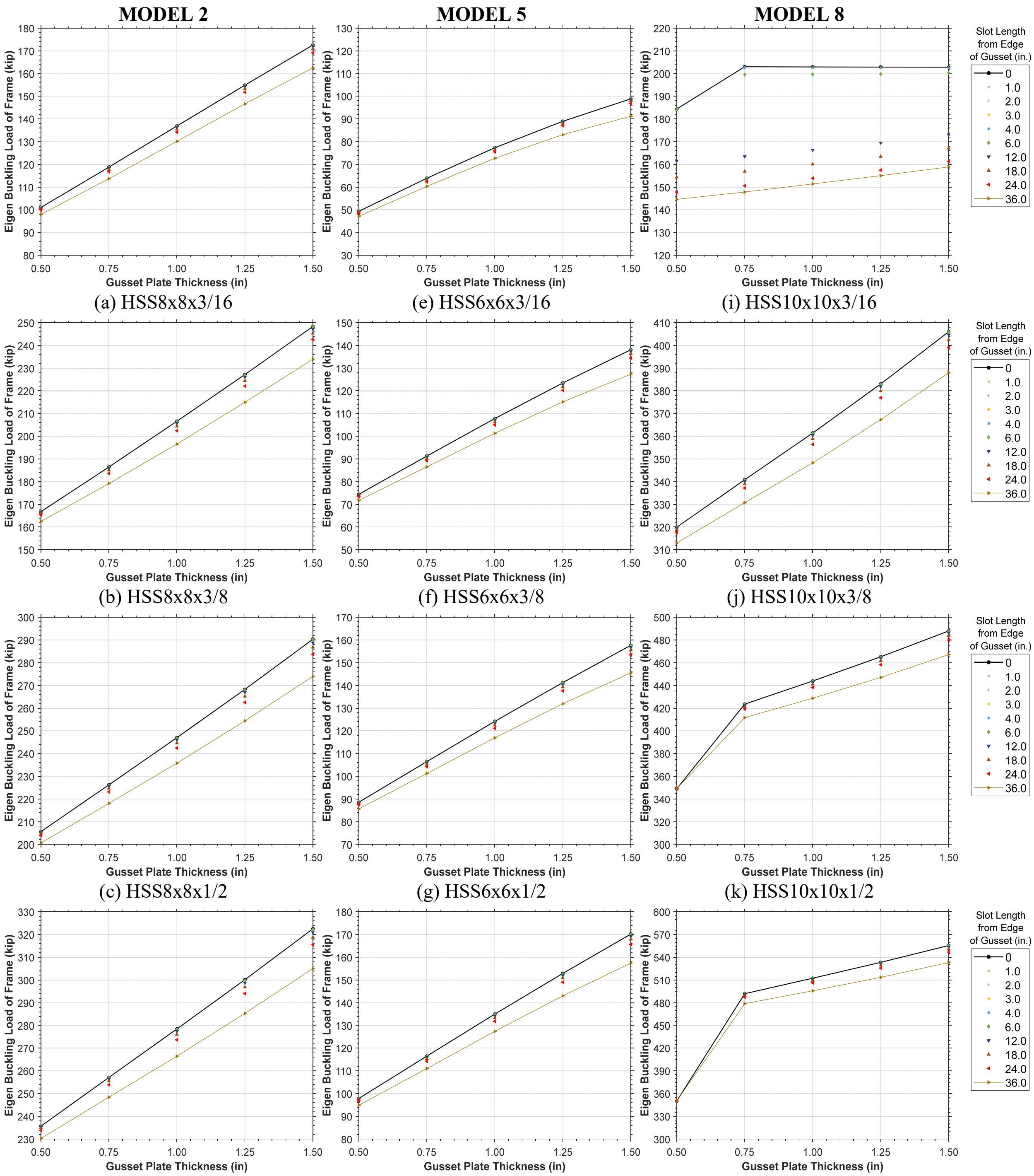
  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in. The buckling load was 142 kip to 233 kip (t = 3/8 in.); 142 kip to 282 kip (t = 1/2 in.); and 142 kip to 323 kip (t = 5/8 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was reduced, still increased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in. The buckling loads were 142 kip to 221 kip (t = 3/8 in.); 142 kip to 268 kip (t = 1/2 in.); and 142 kip to 307 kip (t = 5/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from 0% to greater than 5%.

5.1.2 Observations from SHORT Models 2, 5, 8

The critical linear elastic eigenvalue buckling load of frames for the SHORT configuration Models 2, 5, and 8 with the varying HSS wall thicknesses are summarized in Fig. 12. When comparing the 12 plots the following observations can be made:

The range of gusset plate thickness for all plots was 0.5 in. to 1.50 in. The range of slot length from edge of gusset plate for all plots was 0.0 in. to 36 in.

- Fig. 12(a) – (d) covered bracing members HSS8x8x3/16 to HSS8x8x5/8:
  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased almost linearly from 101 kip to 173 kip (t = 3/16 in.); 167 kip to 248 kip (t = 3/8 in.); 206 kip to 290 kip (t = 1/2 in.); and 236 kip to 323 kip (t = 5/8 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was reduced with respect to the 0.0 in. slot length results almost linearly from 98 kip to 162 kip (t = 3/16 in.); 162 kip to 234 kip (t = 3/8 in.); 201 kip to 274 kip (t = 1/2 in.); and 230 kip to 305 kip (t = 5/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from less than 3% to greater than 5%.



(a) HSS8x8x3/16 (e) HSS6x6x3/16 (i) HSS10x10x3/16

(b) HSS8x8x3/8 (f) HSS6x6x3/8 (j) HSS10x10x3/8

(c) HSS8x8x1/2 (g) HSS6x6x1/2 (k) HSS10x10x1/2

(d) HSS8x8x5/8 (h) HSS6x6x5/8 (l) HSS10x10x5/8

Figure 12: Linear Elastic Eigenvalue Buckling Results for Models 2, 5, and 8 with Varying Wall Thickness

- Fig. 12(e) – (h) covered bracing members HSS6x6x3/16 to HSS6x6x5/8:
  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased almost linearly from 49 kip to 99 kip (t = 3/16 in.); 74 kip to

138 kip (t = 3/8 in.); 89 kip to 158 kip (t = 1/2 in.); and 98 kip to 170 kip (t = 5/8 in.) when a 0.0 in. slot length was maintained;

  - As the slot length was increased over the given range, the buckling load of the frame geometry was reduced with respect to the 0.0 in. slot length results almost linearly from 47 kip to 91 kip (t = 3/16 in.); 72 kip to 127 kip (t = 3/8 in.); 86 kip to 146 kip (t = 1/2 in.); and 95 kip to 157 kip (t = 5/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from approximately 3% to greater than 7%.

- Fig. 12(i) – (l) covered bracing members HSS10x10x3/16 to HSS10x10x5/8 and will be divided into three parts: HSS10x10x3/16, HSS10x10x3/8, and HSS10x10x1/2 to HSS10x10x5/8 since their behaviors are very different than the preceding two bracing member sizes:

  For bracing member HSS10x10x3/16

  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry no longer increased linearly as it went from 184 kip to 203 kip (t = 3/16 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was drastically reduced, though increased linearly from 145 kip to 159 kip (t = 3/16 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction of approximately 22%.

  For bracing member HSS10x10x3/8

  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased linearly as it went from 320 kip to 406 kip (t = 3/8 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was reduced, though increased linearly from 313 kip to 388 kip (t = 3/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from approximately 2% to greater than 4%.

  For bracing members HSS10x10x1/2 to HSS10x10x5/8

  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in. The buckling load was 349 kip to 488 kip (t = 1/2 in.) and 350 kip to 556 kip (t = 5/8 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was reduced with respect to the 0.0 in. slot length results, still increased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in. The buckling loads were 349 kip to 467 kip (t = 1/2 in.); and 350 kip to 533 kip (t = 5/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from 0% to greater than 4%.

5.1.3 Observations from SQUARE Models 3, 6, 9

The critical linear elastic eigenvalue buckling load of frames for the SQUARE configuration Models 3, 6, and 9 with the varying HSS wall thicknesses are summarized in Fig. 13. When comparing the 12 plots the following observations can be made:

The range of gusset plate thickness for all plots was 0.5 in. to 1.50 in. The range of slot length from edge of gusset plate for all plots was 0.0 in. to 36 in.

- Fig. 13(a) – (d) covered bracing members HSS8x8x3/16 to HSS8x8x5/8 and will be divided into two parts: HSS8x8x3/16 to HSS8x8x3/8 and HSS8x8x1/2 to HSS8x8x5/8 since their behaviors are very different than the smallest bracing member size:

  For bracing members HSS8x8x3/16 to HSS8x8x3/8

  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased almost linearly from 69 kip to 105 kip (t = 3/16 in.) and 113 kip to 156 kip (t = 3/8 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was reduced with respect to the 0.0 in. slot length results almost linearly from 68 kip to 100 kip (t = 3/16 in.) and 112 kip to 149 kip (t = 3/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from less than 2% to greater than 4%.

  For bracing members HSS8x8x1/2 to HSS8x8x5/8

  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in. The buckling load was 134 kip to 185 kip (t = 1/2 in.); and 144 kip to 208 kip (t = 5/8 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was reduced with respect to the 0.0 in. slot length results, still increased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in. The buckling loads were 132 kip to 177 kip (t = 1/2 in.); and 143 kip to 199 kip (t = 5/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from less than 1% to greater than 4%.

- Fig. 13(e) – (h) covered bracing members HSS6x6x3/16 to HSS6x6x5/8:

  - As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased almost linearly from 32 kip to 61 kip (t = 3/16 in.); 50 kip to 85 kip (t = 3/8 in.); 61 kip to 97 kip (t = 1/2 in.); and 68 kip to 105 kip (t = 5/8 in.) when a 0.0 in. slot length was maintained;
  - As the slot length was increased over the given range, the buckling load of the frame geometry was reduced almost linearly from 31 kip to 57 kip (t = 3/16 in.); 49 kip to 79 kip (t = 3/8 in.); 60 kip to 91 kip (t = 1/2 in.); and 67 kip to 98 kip (t = 5/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from less than 2% to approximately 7%.

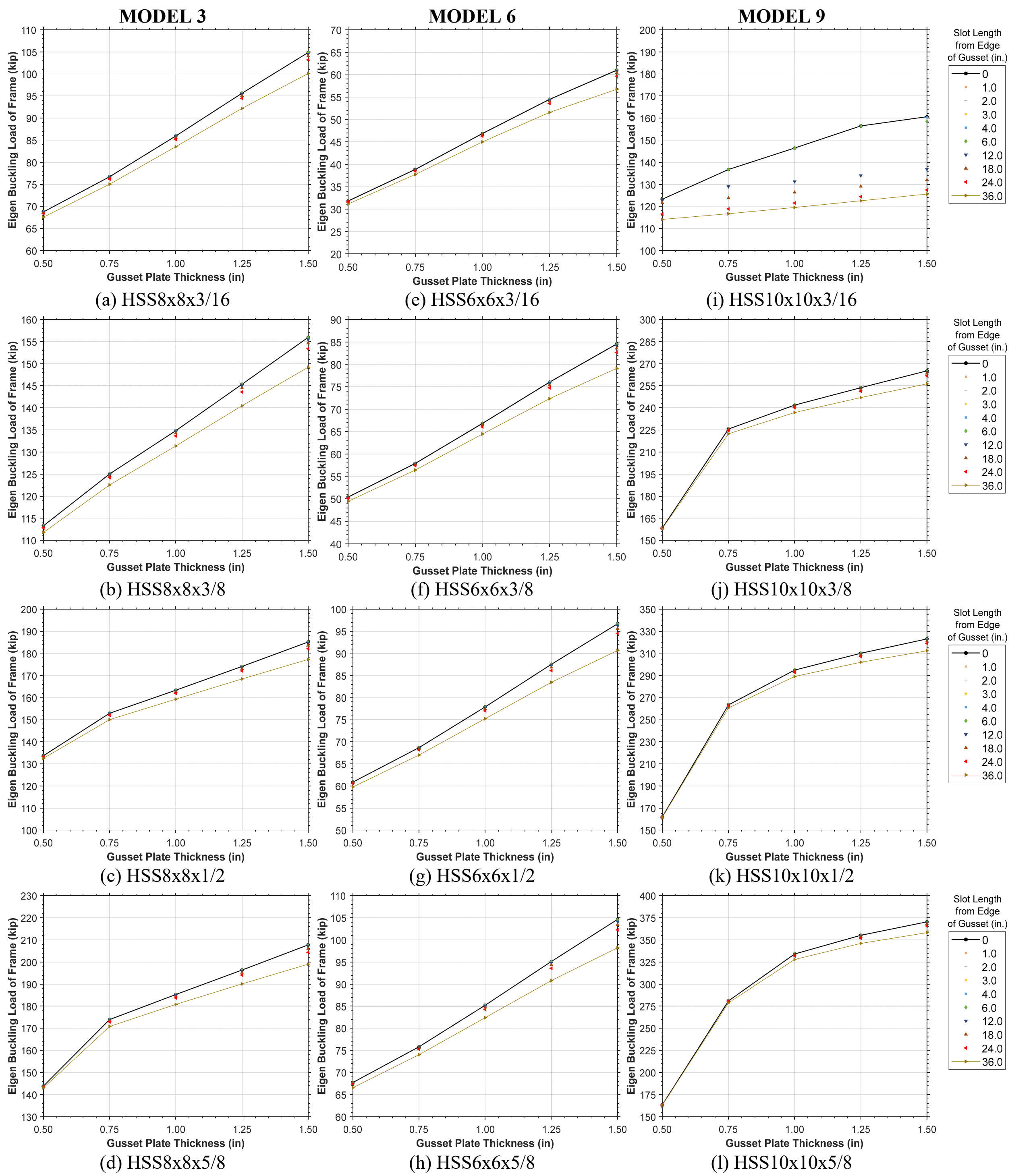


Figure 13: Linear Elastic Eigenvalue Buckling Results for Models 3, 6, and 9 with Varying Wall Thickness

- Fig. 13(i) – (l) covered bracing members HSS10x10x3/16 to HSS10x10x5/8 and will be divided into two parts: HSS10x10x3/16 and HSS10x10x3/8 to HSS10x10x5/8 since their behaviors are very different than the preceding two bracing member sizes:

For bracing member HSS10x10x3/16

- As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry no longer increased linearly as it went from 123 kip to 161 kip (t = 3/16 in.) when a 0.0 in. slot length was maintained;
- As the slot length was increased over the given range, the buckling load of the frame geometry was drastically reduced with respect to the 0.0 in. slot length results, though increased linearly from 114 kip to 126 kip (t = 3/16 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from less than 8% to less than 22%.

For bracing members HSS10x10x3/8 to HSS10x10x5/8

- As the gusset plate thickness was increased over the given range, the buckling load of the frame geometry increased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in. The buckling load was 158 kip to 265 kip (t = 3/8 in.); 162 kip to 323 kip (t = 1/2 in.); and 163 kip to 371 kip (t = 5/8 in.) when a 0.0 in. slot length was maintained;
- As the slot length was increased over the given range, the buckling load of the frame geometry was reduced with respect to the 0.0 in. slot length results, still increased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in. The buckling loads were 158 kip to 256 kip (t = 3/8 in.); 162 kip to 312 kip (t = 1/2 in.); and 163 kip to 358 kip (t = 5/8 in.) when a 36.0 in. slot length was introduced; or in terms of percentage reduction from 0% to greater than 3%.

5.2 Equivalent Effective Length Factor K_{eq}

The frame buckling capacity results presented in Figs. 11, 12, 13 show how the in-plane horizontal buckling capacity of the frame system changes for various parameters. In design, the effective length factor, *K*, is often taken as 1.0 (effective length of brace equals the physical length of the brace). However, for braced frames encountered in industrial buildings where floor-to-floor heights and lateral loading demand can both be larger in magnitude as compared to a typical commercial building, the member sizes required are often much larger (dimensions and web/flange thickness). These larger member sizes mean that the braced frame for some configurations start to behave more like a moment frame than a braced frame. To assess how the end conditions of the brace (beam, column and gusset plate stiffness) may influence the effective length of the brace, an expression for an equivalent effective length factor, K_{eq}, was developed as shown in Eqns. 5 and 6.

To develop an expression for an equivalent effective length factor, K_{eq}, first start with the equations for critical stress of a compression member in accordance with AISC 360 Specification (AISC 2022), which are shown in Eqns. 2 and 3.

$$\mathrm{F_{cr}} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y \quad \text{when } \frac{\mathrm{L_c}}{\mathrm{r}} \leq 4.71\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}} \quad \textit{(AISC Spec. Eqn E3-2)} \tag{2}$$

$$\mathrm{F_{cr}} = 0.877F_e \quad \text{when } \frac{\mathrm{L_c}}{\mathrm{r}} > 4.71\sqrt{\frac{\mathrm{E}}{\mathrm{F_y}}} \quad \textit{(AISC Spec. Eqn E3-3)} \tag{3}$$

where F_e is the Euler elastic buckling stress as shown in Eqn. 4.

$$F_e = \frac{\pi^2 E}{\left(\frac{K\,L_c}{r}\right)^2} \tag{4}$$

Assuming the effective length factor, *K*, is equal to 1.0, Eqns. 2 and 3 above can be rearranged and solved for *K*, which one can call the equivalent effective length factor, K_{eq}. This results in the expressions shown in Eqns. 5 and 6.

$$K_{eq} = \frac{\pi\, r}{L_c}\sqrt{\frac{E}{F_y}\left(\frac{\ln\left(\frac{F_{cr}}{F_y}\right)}{\ln(0.658)}\right)} \quad \text{when } \frac{L_c}{r} \leq 4.71\sqrt{\frac{E}{F_y}} \tag{5}$$

$$K_{eq} = \frac{\pi\, r}{L_c}\sqrt{\frac{0.877E}{F_{cr}}} \quad \text{when } \frac{L_c}{r} > 4.71\sqrt{\frac{E}{F_y}} \tag{6}$$

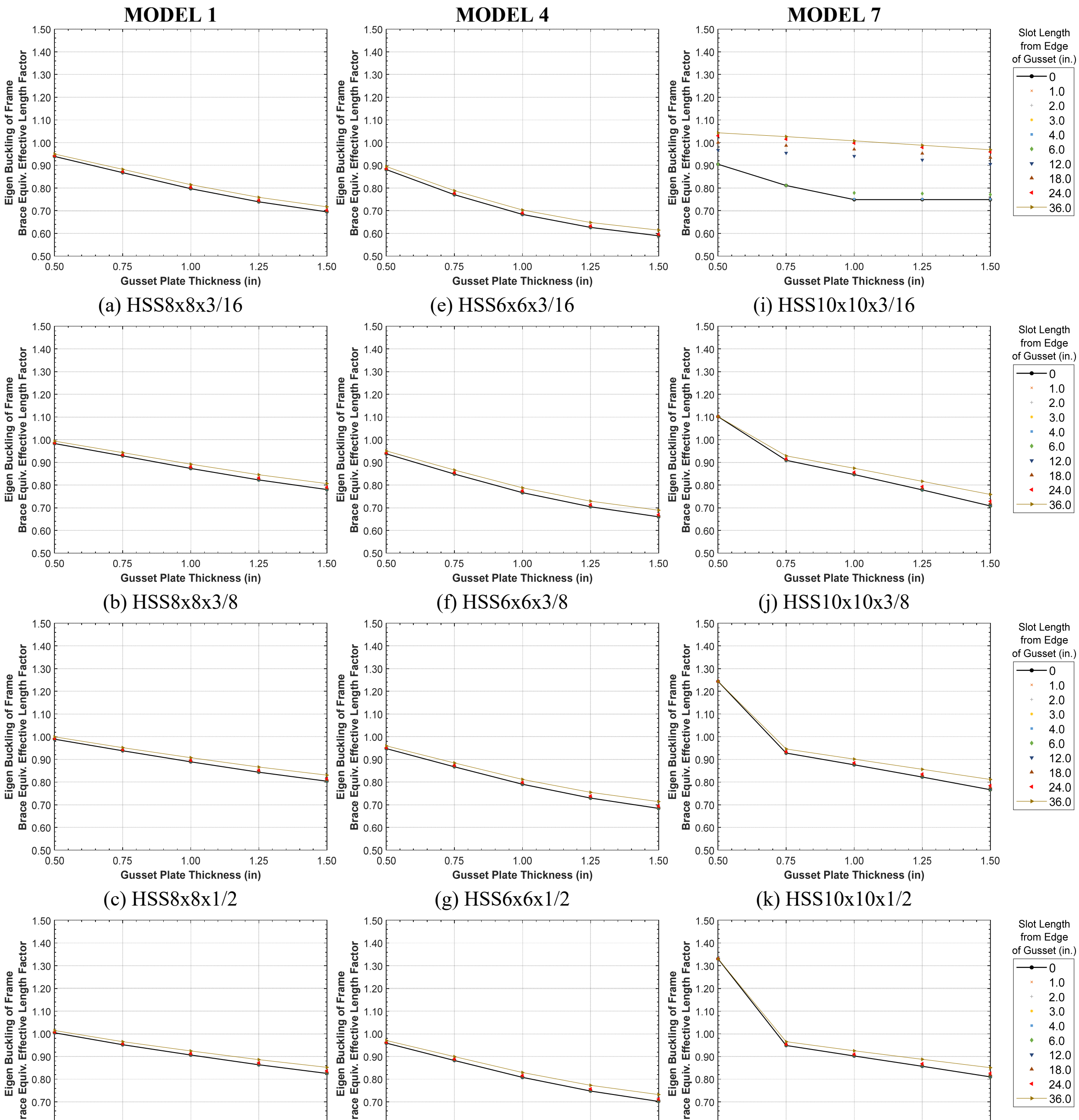
The results in Figs. 11, 12, and 13 represent the lateral force along the axial direction of the top beam. The compressive load along the axial direction of the HSS brace member is determined by dividing the frame critical buckling load by the cosine of the corresponding brace angle. Then that HSS brace axial force can be divided by the HSS brace cross-sectional area to determine the critical stress, F_{cr}. Then, substituting the critical stress into either Eqns. 5 or 6, as appropriate, the equivalent effective length factor of the HSS brace for each configuration can be plotted.

5.2.1 Equivalent Effective Length Factors, K_{eq}, for TALL Models 1, 4, 7

The equivalent effective length factors, K_{eq}, of brace members for the TALL configuration Models 1, 4, and 7 with the varying HSS wall thicknesses are summarized in Fig. 14. When comparing the 12 plots the following observations can be made:

The range of gusset plate thickness for all plots was 0.5 in. to 1.50 in. The range of slot length from edge of gusset plate for all plots was 0.0 in. to 36 in.

- Fig. 14(a) – (d) covered bracing members HSS8x8x3/16 to HSS8x8x5/8 and Fig. 14(e) – (h) covered bracing members HSS6x6x3/16 to HSS6x6x5/8:
    - As the gusset plate thickness was increased over the given range, K_{eq}, for the HSS bracing member decreased almost linearly;
    - Model 1 K_{eq} showed a decrease from 0.94 to 0.69 (t = 3/16 in.); 0.98 to 0.78 (t = 3/8 in.); 0.99 to 0.80 (t = 1/2 in.); and 1.00 to 0.83 (t = 5/8 in.) when a 0.0 in. slot length was maintained. An almost insignificant increase in K_{eq} was observed, 0.95 to 0.72 (t = 3/16 in.); 0.99 to 0.81 (t = 3/8 in.); 1.00 to 0.83 (t = 1/2 in.); and 1.01 to 0.85 (t = 5/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 4%;



(a) HSS8x8x3/16 (e) HSS6x6x3/16 (i) HSS10x10x3/16

(b) HSS8x8x3/8 (f) HSS6x6x3/8 (j) HSS10x10x3/8

(c) HSS8x8x1/2 (g) HSS6x6x1/2 (k) HSS10x10x1/2

(d) HSS8x8x5/8 (h) HSS6x6x5/8 (l) HSS10x10x5/8

Figure 14: Equivalent Effective Length Factors, K_{eq}, Results for Models 1, 4, and 7 with Varying Wall Thickness

- Model 4 K_{eq} showed a decrease from 0.88 to 0.59 (t = 3/16 in.); 0.94 to 0.66 (t = 3/8 in.); 0.95 to 0.68 (t = 1/2 in.); and 0.96 to 0.70 (t = 5/8 in.) when a 0.0 in. slot length was maintained. An almost insignificant increase in K_{eq} was observed, 0.89 to 0.61 (t = 3/16 in.); 0.95 to 0.69 (t = 3/8 in.); 0.96 to 0.71 (t = 1/2 in.); and 0.97 to 0.73 (t = 5/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 5%.

- Fig. 14(i) – (l) covered bracing members HSS10x10x3/16 to HSS10x10x5/8 and will be divided into two parts: HSS10x10x3/16 and HSS10x10x3/8 to HSS10x10x5/8 since their behaviors are very different than the preceding two bracing member sizes:

  For bracing member HSS10x10x3/16

  - As the gusset plate thickness was increased over the given range, K_{eq} for the HSS bracing member no longer decreased linearly when a 0.0 in. slot length was maintained;
  - Model 7 K_{eq} showed a decrease from 0.90 to 0.75 (t = 3/16 in.);
  - As the slot length was increased over the given range, K_{eq} for the HSS bracing member was significantly raised, though decreased linearly from 1.04 to 0.97 (t = 3/16 in.) when a 36.0 in. slot length was introduced; or in terms of percentage, from 15% to 29%.

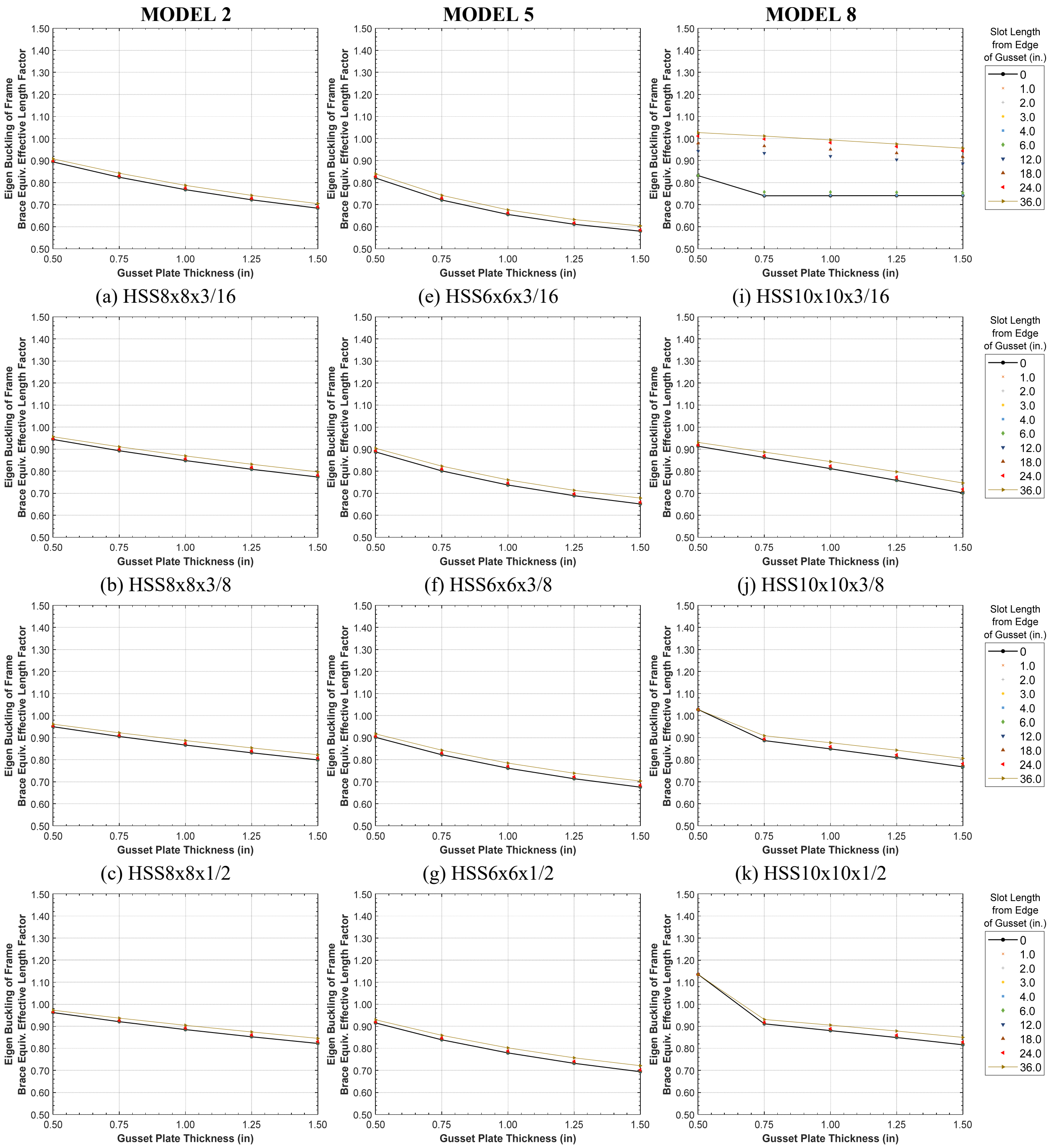
  For bracing members HSS10x10x3/8 to HSS10x10x5/8

  - As the gusset plate thickness was increased over the given range, K_{eq} for the HSS bracing member no longer decreased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in;
  - Model 7 K_{eq} showed a decrease from 1.10 to 0.71 (t = 3/8 in.); 1.24 to 0.77 (t = 1/2 in.); and 1.33 to 0.81 (t = 5/8 in.) when a 0.0 in. slot length was maintained. An almost insignificant increase in K_{eq} was observed, 1.10 to 0.76 (t = 3/8 in.); 1.24 to 0.81 (t = 1/2 in.); and 1.33 to 0.85 (t = 5/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 5%.

5.2.2 Equivalent Effective Length Factors, K_{eq}, for SHORT Models 2, 5, 8

The equivalent effective length factors, K_{eq}, of brace members for the TALL configuration Models 2, 5, and 8 with the varying HSS wall thicknesses are summarized in Fig. 15. When comparing the 12 plots the following observations can be made:

- The range of gusset plate thickness for all plots was 0.5 in. to 1.50 in. The range of slot length from edge of gusset plate for all plots was 0.0 in. to 36 in;
- Fig. 15(a) – (d) covered bracing members HSS8x8x3/16 to HSS8x8x5/8 and Fig. 15(e) – (h) covered bracing members HSS6x6x3/16 to HSS6x6x5/8:
  - As the gusset plate thickness was increased over the given range, K_{eq}, for the HSS bracing member decreased almost linearly;
  - Model 2 decrease was from 0.89 to 0.68 (t = 3/16 in.); 0.94 to 0.77 (t = 3/8 in.); 0.95 to 0.80 (t = 1/2 in.); and 0.96 to 0.82 (t = 5/8 in.) when a 0.0 in. slot length was maintained. An almost insignificant increase in K_{eq} was observed, 0.91 to 0.70 (t = 3/16 in.); 0.96 to 0.80 (t = 3/8 in.); 0.96 to 0.82 (t = 1/2 in.); and 0.97 to 0.85 (t = 5/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 3%;
  - Model 5 decrease was from 0.82 to 0.58 (t = 3/16 in.); 0.89 to 0.65 (t = 3/8 in.); 0.90 to 0.68 (t = 1/2 in.); and 0.91 to 0.69 (t = 5/8 in.) when a 0.0 in. slot length was maintained. An almost insignificant increase in K_{eq} was observed, 0.84 to 0.60 (t = 3/16 in.); 0.90 to 0.68 (t = 3/8 in.); 0.92 to 0.70 (t = 1/2 in.); and 0.93 to 0.72 (t = 5/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 5%.



(a) HSS8x8x3/16 (e) HSS6x6x3/16 (i) HSS10x10x3/16

(b) HSS8x8x3/8 (f) HSS6x6x3/8 (j) HSS10x10x3/8

(c) HSS8x8x1/2 (g) HSS6x6x1/2 (k) HSS10x10x1/2

(d) HSS8x8x5/8 (h) HSS6x6x5/8 (l) HSS10x10x5/8

Figure 15: Equivalent Effective Length Factors, K_{eq}, Results for Models 2, 5, and 8 with Varying Wall Thickness

- Fig. 15(i) – (l) covered bracing members HSS10x10x3/16 to HSS10x10x5/8 and will be divided into three parts: HSS10x10x3/16, HSS10x10x3/8, and HSS10x10x1/2 to HSS10x10x5/8 since their behaviors are very different than the preceding two bracing member sizes:

For bracing member HSS10x10x3/16

- o As the gusset plate thickness was increased over the given range, K_{eq} for the HSS bracing member no longer decreased linearly when a 0.0 in. slot length was maintained;
- o Model 8 K_{eq} showed a decrease from 0.83 to 0.74 (t = 3/16 in.);
- o As the slot length was increased over the given range, K_{eq} for the HSS bracing member was significantly raised, though decreased linearly from 1.03 to 0.96 (t = 3/16 in.) when a 36.0 in. slot length was introduced; or in terms of percentage, from 23% to 29%.

For bracing member HSS10x10x3/8

- o As the gusset plate thickness was increased over the given range, K_{eq} for the HSS bracing member decreased linearly when a 0.0 in. slot length was maintained;
- o Model 8 K_{eq} showed a decrease from 0.91 to 0.70 (t = 3/8 in.);
- o An almost insignificant increase in K_{eq} was observed, 0.93 to 0.75 (t = 3/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 7%.

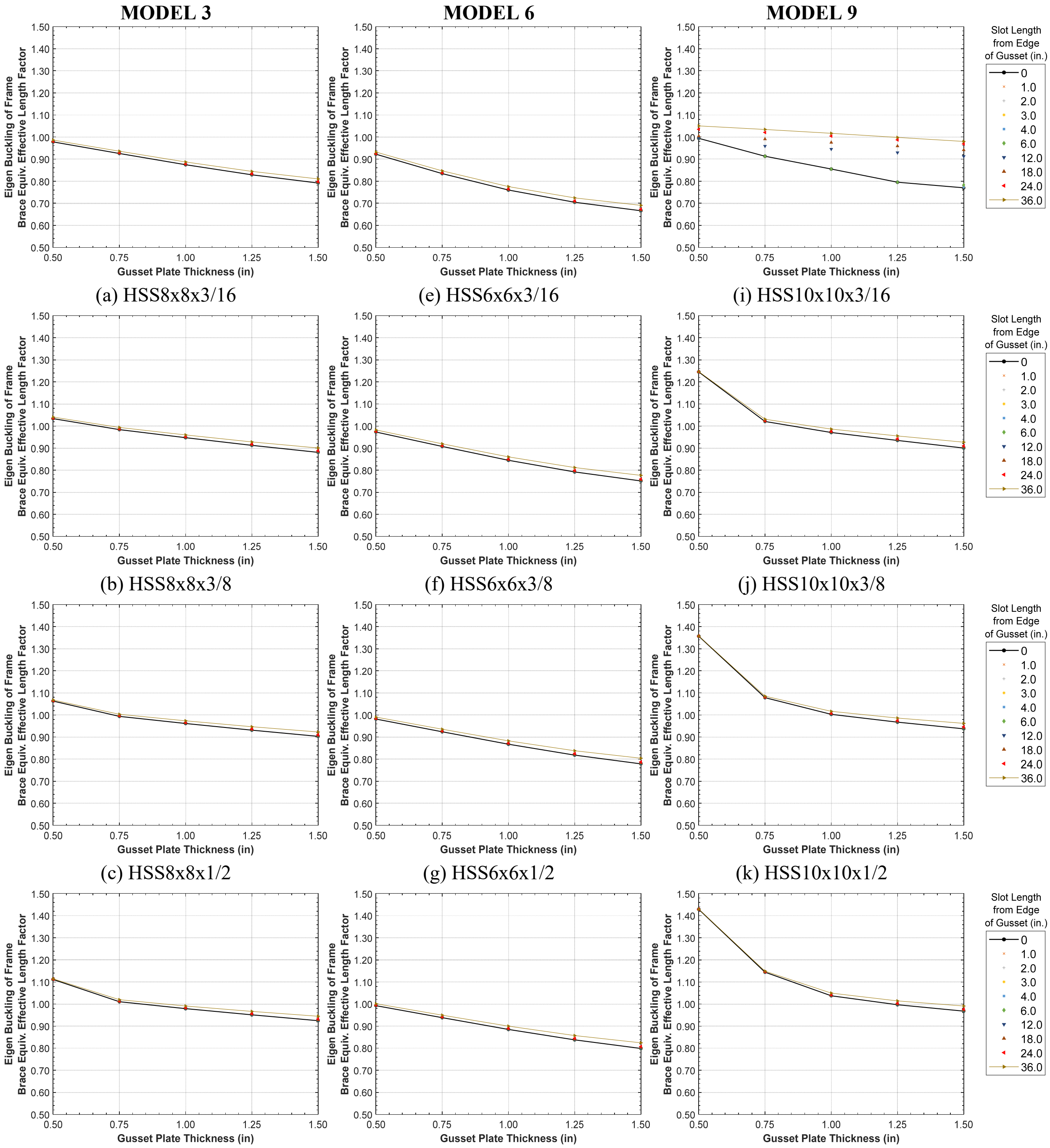
For bracing members HSS10x10x1/2 to HSS10x10x5/8

- o As the gusset plate thickness was increased over the given range, K_{eq} for the HSS bracing member no longer decreased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in;
- o Model 8 K_{eq} showed a decrease from 1.03 to 0.77 (t = 1/2 in.); and 1.14 to 0.82 (t = 5/8 in.) when a 0.0 in. slot length was maintained. An almost insignificant increase in K_{eq} was observed, 1.03 to 0.81 (t = 1/2 in.); and 1.13 to 0.85 (t = 5/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 5%.

5.2.3 Equivalent Effective Length Factors, K_{eq}, for SQUARE Models 3, 6, 9

The equivalent effective length factors, K_{eq}, of brace members for the TALL configuration Models 3, 6, and 9 with the varying HSS wall thicknesses are summarized in Fig. 16. When comparing the 12 plots the following observations can be made:

- The range of gusset plate thickness for all plots was 0.5 in. to 1.50 in. The range of slot length from edge of gusset plate for all plots was 0.0 in. to 36 in.
- Fig. 16(a) – (d) covered bracing members HSS8x8x3/16 to HSS8x8x5/8 and Fig. 16(e) – (h) covered bracing members HSS6x6x3/16 to HSS6x6x5/8:
  - o As the gusset plate thickness was increased over the given range, K_{eq}, for the HSS bracing member decreased almost linearly;
  - o Model 3 decrease was from 0.98 to 0.79 (t = 3/16 in.); 1.03 to 0.88 (t = 3/8 in.); 1.06 to 0.90 (t = 1/2 in.); and 1.11 to 0.92 (t = 5/8 in.) when a 0.0 in. slot length was maintained. An almost insignificant increase in K_{eq} was observed, 0.99 to 0.81 (t = 3/16 in.); 1.04 to 0.90 (t = 3/8 in.); 1.07 to 0.92 (t = 1/2 in.); and 1.11 to 0.94 (t = 5/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 3%;
  - o Model 6 decrease was from 0.92 to 0.67 (t = 3/16 in.); 0.97 to 0.75 (t = 3/8 in.); 0.98 to 0.78 (t = 1/2 in.); and 0.99 to 0.80 (t = 5/8 in.) when a 0.0 in. slot length was maintained.



(a) HSS8x8x3/16 (e) HSS6x6x3/16 (i) HSS10x10x3/16

(b) HSS8x8x3/8 (f) HSS6x6x3/8 (j) HSS10x10x3/8

(c) HSS8x8x1/2 (g) HSS6x6x1/2 (k) HSS10x10x1/2

(d) HSS8x8x5/8 (h) HSS6x6x5/8 (l) HSS10x10x5/8

Figure 16: Equivalent Effective Length Factors, K_{eq}, Results for Models 3, 6, and 9 with Varying Wall Thickness

An almost insignificant increase in K_{eq} was observed, 0.93 to 0.69 (t = 3/16 in.); 0.98 to 0.78 (t = 3/8 in.); 0.99 to 0.80 (t = 1/2 in.); and 1.00 to 0.82 (t = 5/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 4%.

- Fig. 16(i) – (l) covered bracing members HSS10x10x3/16 to HSS10x10x5/8 and will be divided into two parts: HSS10x10x3/16 and HSS10x10x3/8 to HSS10x10x5/8 since their behaviors are very different than the preceding two bracing member sizes:

For bracing member HSS10x10x3/16

- As the gusset plate thickness was increased over the given range, K_{eq} for the HSS bracing member no longer decreased linearly when a 0.0 in. slot length was maintained;
- Model 9 K_{eq} showed a decrease from 0.99 to 0.77 (t = 3/16 in.);
- As the slot length was increased over the given range, K_{eq} for the HSS bracing member was significantly raised, though decreased linearly from 1.05 to 0.98 (t = 3/16 in.) when a 36.0 in. slot length was introduced; or in terms of percentage, from 5% to 27%.

For bracing members HSS10x10x3/8 to HSS10x10x5/8

- As the gusset plate thickness was increased over the given range, K_{eq} for the HSS bracing member no longer decreased linearly, but had two distinct slopes that occurred at a gusset plate thickness of 0.75 in;
- Model 9 K_{eq} showed a decrease from 1.24 to 0.90 (t = 3/8 in.); 1.36 to 0.94 (t = 1/2 in.); and 1.43 to 0.97 (t = 5/8 in.) when a 0.0 in. slot length was maintained. An almost insignificant increase in K_{eq} was observed, 1.25 to 0.93 (t = 3/8 in.); 1.36 to 0.96 (t = 1/2 in.); and 1.43 to 0.99 (t = 5/8 in.), when a 36.0 in. slot length was introduced; or in terms of percentage, less than 3%.

5.3 Effects of Field Modifications

Any field modification to a bracing system that has been previously analyzed, designed, fabricated and erected may affect the frame's overall structural stability. Without knowing to what degree that modification was should be a concern to all parties involved in a building's construction and most importantly to the Engineer of Record. One common case where this has been observed is the modification to the slotted end length of HSS vertical bracing members that will be welded to previously installed gusset plate end connections. These gusset plates are typically bolted or welded to the frame members prior to the HSS element being placed. Since the beams and columns are already erected, i.e., the frame is already in a stable configuration, it is incumbent on the Erector to best determine how the vertical braces are installed without having to dismantle a portion of the frame. The HSS vertical bracing member must be fit over the gusset plates and to accomplish this, they must now be rotated into position. To ease construction or eliminate any fit-up issues that may occur in the field the slotted end length of the member will be extended. The responsible parties are supposed to be notified as dictated by the AISC 303 Code of Standard Practice (2022), but this may or may not occur depending on the type of project. If an evaluation is not conducted, it may never be known if the structural capacity of the frame has been compromised, and if it has been, to what degree.

6. Discussion, Summary, and Conclusions

The analytical study started out with a narrow focus to determine whether increasing the end slot length on HSS bracing members to ease construction installation would be detrimental to the overall stability of a braced frame. A limited scope of work was conducted that tried to answer this, but the results were inconclusive (Reigles et al. 2025). Therefore, a broader range of factors needed to better understand the buckling capacity and overall performance of end slotted HSS braced frames was undertaken. It was hoped that this more comprehensive framework would accomplish this and lead to more versatile and optimized braced frame design utilizing HSS members. The study parameters were expanded as follows:

1) Three bay sizes, i.e., structural frame geometries, were studied while keeping the overall HSS brace length constant, L_{BRACE} = ~32 ft. The braced frame configurations were shown in detail in Figs. 3, 4, and 5 and labeled TALL = 34'-6" x 20'-0, SHORT = 20'-0 x34'-6", and SQUARE = 28'-3" x 28'-3", respectively;
2) The three structural frame geometries were comprised of wide-flange beams and columns whose members were selected so that the ratio of their beam-weak axis column flexural stiffness would be approximately 2.0 (TALL = 2.04, SHORT = 2.05, and SQUARE = 1.97);
3) A range of HSS bracing member sizes was selected that would encompass their typical use in commercial or industrial building types being studied. The HSS members selected were: HSS6x6, HSS8x8, and HSS10x10;
4) A range of HSS bracing member wall thicknesses was selected that would encompass their typical use in commercial or industrial building types being studied. The HSS members wall thicknesses selected were: t_{HSS} = 3/16 in., 3/8 in., 1/2 in., and 5/8 in.;
5) A range of HSS bracing member end slot lengths as measured from the edge of the gusset plate was selected that would preclude possible field fit up to an excessive slot length that should not occur, but would give an upper bound to any effects the slot would have on the buckling capacity of the brace. The end slot lengths chosen were: L_{SLOT} = 0, 1.0 in., 2.0 in., 3.0 in., 4.0 in., 6.0 in., 12.0 in., 18.0 in., 24.0 in., and 36.0 in.;
6) Unique gusset plate geometries were established for the TALL, SHORT, and SQUARE structural frames. It was determined that the influence of those various geometries would be minimal to the overall buckling capacity of the braced frames as the in-contact length between the HSS member end slot and the gusset plate was kept constant at 10.5 in.;
7) The range of gusset plate thicknesses was selected that would encompass their typical use in commercial or industrial building types being studied. The thicknesses that were selected were: $t_{GUSSET\ PLATE}$ = 0.25 in., 0.50 in., 0.75 in., 1.00 in., 1.25 in., and 1.50 in.

The following limitations from the previous study (Reigles et al. 2025) were maintained in this current study:

1) ASTM A500 Gr. C, ASTM A572 Gr. 50, and ASTM A992 material properties were used for the HSS members, gusset plates, and wide flange shapes, respectively;
2) Initial imperfections were not applied to any of the structural members in the ANSYS finite element models;
3) No residual stress patterns were applied to any of the structural members in the ANSYS finite element models.

The first significant result that was observed happens when the gusset plate provided is too thin. In the current study, regardless of the size or thickness of the HSS bracing member, gusset plates with a thickness equal to 0.25 in. (or less than 0.50 in.) were observed to buckle out-of-plane as illustrated in Fig. 9 while the bracing element "went along for the ride" and did not buckle. Fig. 10 provides the gusset plate Whitmore section lengths for all the finite element models and Tables 6, 7, and 8 tabulate the gusset plate nominal compressive strengths for the TALL (Models 1, 4, and 7); SHORT (Models 2, 5, and 8); and SQUARE (Models 3, 6, and 9) based on AISC 360 (2022) Chapter E, respectively. Two bounding values are provided for each gusset plate, KL = 1.0 and KL = 0.65, along with each HSS member size. The two effective lengths are a reasonable attempt to simulate a pin-ended and a partially restrained boundary condition for the gusset plate as it has been modeled attached on two sides, one being to the flange of the beam and the other to web of the column. It should be noted that the gusset plate nominal compressive strength values provided are significantly smaller than those determined for the TALL (Models 1, 4, 7); SHORT (Models 2, 5, and 8); or SQUARE (Models 3, 6, and 9) braced frames when the gusset plate thickness is 0.50 in. or greater.

The eigenvalue buckling loads of the braced frames are illustrated in Figs. 11, 12, and 13. Models 4, 5, and 6 have HSS6x6 bracing members, Models 1, 2, and 3 have HSS8x8 bracing members, and Models 7, 8, and 9 have HSS10x10 bracing members. Only general trends can be observed over the range of gusset plate thickness from 0.50 in. to 1.50 in. for a majority of the results. In general, the elastic buckling loads of the structural frames with the smallest and midsize braces show their load capacity increases nearly linearly with increasing gusset plate thickness.

The following figures show that as the gusset plate thickness increases from 0.50 in. to 1.50 in., the buckling capacity increases nearly linearly, as follows when the slot length from the edge of the gusset plate is 0.0 in.:

Fig. 11(a)-(d), HSS8x8 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.83, 1.58, 1.51, 1.48;
Fig. 11(e)-(h), HSS6x6 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 2.20, 2.05, 1.93, 1.88;
Fig. 12(a)-(d), HSS8x8 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.71, 1.49, 1.41, 1.37;
Fig. 12(e)-(h), HSS6x6 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 2.02, 1.86, 1.78, 1.73;
Fig. 13(a)-(d), HSS8x8 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.52, 1.38, 1.38, 1.44;
Fig. 13(e)-(h), HSS6x6 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.91, 1.70, 1.59, 1.54.

The same figures show that there is a slight reduction in load ratios of these frames when the slot length gets extended from 0.0 in. to an extreme value of 36.0 in., as follows:

Fig. 11(a)-(d), HSS8x8 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.75, 1.52, 1.45, 1.42;
Fig. 11(e)-(h), HSS6x6 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 2.13, 1.92, 1.82, 1.60;
Fig. 12(a)-(d), HSS8x8 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.65, 1.44, 1.36, 1.33;
Fig. 12(e)-(h), HSS6x6 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.94, 1.76, 1.70, 1.65;
Fig. 13(a)-(d), HSS8x8 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.47, 1.33, 1.34, 1.39;
Fig. 13(e)-(h), HSS6x6 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.84, 1.61, 1.52, 1.46.

Therefore, in general, based on the results presented in these figures, it may or may not be necessary that a reduced load capacity be considered back into the design when the slot at the end of the HSS brace is extended beyond a reasonable length for constructability.

A much different behavior, i.e., result has been observed in the following figures when the slot length from the edge of the gusset plate is 0.0 in.:

Fig. 11(i)-(l), HSS10x10 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.18, 1.64, 1.99, 2.27;
Fig. 12(i)-(l), HSS10x10 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.10, 1.27, 1.40, 1.59;
Fig. 13(i)-(l), HSS10x10 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.31, 1.68, 1.99, 2.28.

The same figures show that there is a slight reduction in load ratios of these frames when the slot length gets extended from 0.0 in. to an extreme value of 36.0 in., as follows:

Fig. 11(i)-(l), HSS10x10 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.10, 1.56, 1.89, 2.16;
Fig. 12(i)-(l), HSS10x10 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.10, 1.24, 1.34, 1.52;
Fig. 13(i)-(l), HSS10x10 (w/ t_{WALL} = 3/16, 3/8, 1/2, 5/8) load ratios: 1.11, 1.62, 1.93, 2.20.

Further study into the behavior of these finite element models is warranted, but is outside the scope of this research. However, based on these results, the design information provided in the AISC Steel Construction Manual (2023), Part 13 Design of Bracing Connections and Truss Connections, needs to be expanded as it currently only focuses on the gusset plate portion of the bracing connection. The bracing element itself framing into the connection region needs to be considered. It is recommended that new rules or guidelines be developed based on this research or subsequent study that will address, 1) gusset plate design in more detail including material, size, shape, thickness, provided connection length, and available Whitmore section; and 2) HSS vertical bracing design that will include material, size, wall thickness, overall or clear length, and provided end slot length.

Finally, it should be noted that while the frame geometries established in this study provided a good upper and lower bound for a majority of bay dimensions in commercial or industrial facilities, at least two other limitations exist. The first was the HSS vertical bracing length that was kept constant setting the height and width dimensions of the structural frames. The second was the beam-to-column flexural stiffness ratio that was kept constant. The design implication(s) of a shorter HSS vertical bracing length, e.g., by 25% (or approximately 24 ft.) has yet to be evaluated. The design implication(s) of a higher beam-to-column flexural stiffness ratio of the frame members, e.g., by 50% (or approximately 3.0) has yet to be ascertained.

In order to accomplish the analytical work in a timely manner it was decided that all the finite element studies would only be performed using linear elastic material properties. Going forward, the following should be considered:

- Any future work should look at a subset of the critical parameters explored and include the effects of nonlinear material properties as well as geometric imperfections;
- Any future work should look into the relationship between the frame geometry, the gusset plate, its Whitmore section buckling capacity, and the HSS member size, wall thickness, and brace length need further study as the brace length was kept constant in this research for the various frame geometries that were evaluated;
- Any future work should look at using reduced effective lengths, i.e., $K < 1.0$, for the design of the bracing elements for seismic $R = 3$ type systems which are common for industrial facilities. For wind analysis, where we don't typically rely on inelasticity, it may be worth exploring the use of reduced effective lengths;
- The authors feel that in any future work 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 that HSS bracing members with extended end slots have on seismic performance. Such studies are essential for determining the suitability of extended slots being provided in seismic design and their impact on the energy dissipation and resilience of bracing systems.

References

AISC (2014). **Design Guide 29: Vertical Bracing Connections – Analysis and Design**, by Larry S. Muir and William A. Thornton, American Institute of Steel Construction, Chicago, IL.

AISC (2022). ANSI/AISC 303-22. An American National Standard. "Code of Standard Practice for Steel Buildings and Bridges." American Institute of Steel Construction, Chicago, IL.

AISC (2022). ANSI/AISC 341-22. An American National Standard. "Seismic Provisions for Structural Steel Buildings." American Institute of Steel Construction, Chicago, IL.

AISC (2022). ANSI/AISC 360-22. An American National Standard. "Specification for Structural Steel Buildings." American Institute of Steel Construction, Chicago, IL.

AISC (2023). **Seismic Design Manual**. Fourth Edition, American Institute of Steel Construction, Chicago, IL.

AISC (2023). **Steel Construction Manual**. Sixteenth Edition, American Institute of Steel Construction, Chicago, IL.

AISC (2024). **Design Guide 24: Hollow Structural Section Connections (Second Edition)**, by Jeffrey A. Packer and Kimberley Olson, American Institute of Steel Construction, Chicago, IL.

ANSYS (2022), Release 2022 R2, "Element Reference," July 2022, ANSYS, Inc.

ASTM A500/A500M-23 (2023). "Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes." ASTM International, West Conshohocken, PA.

ASTM A572/A572M-23 (2023). "Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel." ASTM International, West Conshohocken, PA.

ASTM A992/A992M-22 (2022). "Standard Specification for Structural Steel Shapes." ASTM International, West Conshohocken, PA.

AWS D1.1/D1.1M:2015 (2015). "Structural Welding Code-Steel." American Welding Society, Miami, FL.

Dowswell, B. (2006). "Effective Length Factors for Gusset Plate Buckling," ***Engineering Journal***, AISC, Vol. 43, No. 2, Second Quarter, pp 91 – 101.

Dowswell, B. (2021). "Analysis of the Shear Lag Factor for Slotted Rectangular HSS Members," ***Engineering Journal***, AISC, Vol. 58, No. 3, Third Quarter, pp 155 – 164.

Dowswell, B. and Lini, C. (2023). "Drop It in the Slot," ***Modern Steel Construction***, American Institute of Steel Construction, September, pp. 16 - 18.

Dranger, T.S. and Thornton, W.A (2025), "A Derivation of the Uniform Force Method for Analysis and Design of Gusset Plate Connections for Vertical Diagonal Bracing," ***Engineering Journal*,** AISC, Vol. 62, No. 2, Second Quarter, pp 55 – 58.

Hsiao, P-C. (2012). "Seismic Performance Evaluation of Concentrically Braced Frames." PhD Dissertation, Department of Civil and Environmental Engineering, University of Washington.

MASTAN2 (2024), Version 3.5, retrieved from http://www.mastan2.com/about.html

McCormick, J. (2017). "Considerations for Use of HSS in Seismic Frame Systems." Steel Tube Institute (STI). Chicago, IL.

Muir, Larry S. (2008) "Designing Compact Gussets with the Uniform Force Method," ***Engineering Journal***, AISC, Vol. 45, No. 1, First Quarter, pp 13 – 20.

Nair, R. Shankar (2009) "A Model Specification for Stability Design by Direct Analysis," ***Engineering Journal***, AISC, Vol. 46, No. 1, First Quarter, pp 29 – 38.

Packer, J.A. (2006). Edited, Tubular Structures XI, 11th International Symposium and IIW International Conference on Tubular Structures, 1st Edition, contains Houdremont Lecture by Jeffrey A. Packer, "Tubular brace member connections in braced steel frames."

Reigles, D.G., Green, P.S., and Brown, H.J. (2025). "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." *Annual Stability Conference Proceedings*, Louisville, KY, Structural Stability Research Council (SSRC), April 1-4.

Roeder, C.W., Lehman, D.E., Lumpkin, E., Hsiao, P-C, and Palmer K. (2011). "SCBF Gusset Plate Design," T.R. Higgins Lecture, AISC North American Steel Construction Conference, Pittsburgh, PA, May 11-14.

Sabelli, R., Roeder, C.W., and Hajjar, J.F. (2013). "Seismic Design of Steel Special Concentrically Braced Frame Systems: A Guide for Practicing Engineers," NIST GCR 13-917-24, US Dept. of Commerce, National Institute of Standards and Technology, Gaithersburg, MD.

Saucedo, G.M. (2007). "Slotted End Connections to Hollow Sections." PhD Dissertation, Department of Civil Engineering, University of Toronto.

STI (2016), **HSS Design Manual, Volume 4: Truss and Bracing Connections**, Steel Tube Institute, Chicago, IL.

Thornton, W.A. (1984). "Bracing Connections for Heavy Construction." ***Engineering Journal***, AISC, Vol. 21, No. 2, Second Quarter, pp 139 - 148.

Thornton, W.A. (1991). "On the Analysis and Design of Bracing Connections." *Proceedings*, AISC National Steel Construction Conference, Washington, DC, pp 26-1 through 26-33.

Thornton, W.A. and Lini, C. (2011). "The Whitmore Section," ***Modern Steel Construction***, American Institute of Steel Construction, July, 5 pp.

Thornton, W.A. and Fortney, P.J. (2012). "Satisfying Inelastic Rotation Requirements for In-Plane Critical Axis Brace Buckling for High Seismic Design," *Engineering Journal*, American Institute of Steel Construction, Third Quarter, pp. 99-108.

Unterweger H., Taras, A. (2013). "Compression Members with Hollow Sections and Concentric Slotted Gusset Plates – Behavior and Recommended Design Model," *Annual Stability Conference Proceedings*, St. Louis, MO, Structural Stability Research Council (SSRC), April 16-20.

Ziemian, R. D., Editor (2010). **Guide to Stability Design Criteria for Metal Structures**. Sixth Edition, John Wiley & Sons Inc., Hoboken, NJ.