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**Smarter.
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333 Design of Shallow-Depth Floor Systems with
Asymmetric Steel I-Beams
Matthew T. Yarnold and Chase Ottmers

353 Evaluation of Seismic Design Parameters
for Metal Buildings with Heavy Wall and
Heavy Roof in High Seismic Zones
Mohammad T. Nikoukalam, Shahabeddin
Torabian, and Benjamin W. Schafer

377 Design Recommendation for Two-Sided
Bolted Rectangular HSS End-Plate
Connections in Axial Tension
Ian G. Kennedy, Jeffrey A. Packer, and
Kyle Tousignant

395 Shear Lag Effects in Tension Members with
Short Longitudinal Welds
Jackson T. Muncy and Mark D. Denavit

Steel Structures Research Update
411 High Strength Structural Steel in
Building Applications
Judy Liu

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Design of Shallow-Depth Floor Systems with Asymmetric Steel I-Beams

MATTHEW T. YARNOLD and CHASE OTTMERS

ABSTRACT

Shallow-depth composite floor systems aim to improve building economy, speed, and efficiency. Currently, an array of proprietary systems is available that comprises asymmetric steel sections (I-beam or trapezoidal box) with a relatively wide bottom flange. This wide bottom flange facilitates the placement of decking or precast/prefabricated panels. These systems are fast to assemble compared to cast-in-place flat slab construction. However, extensive fabrication is typically required for the current steel sections. As a result, this paper presents a design methodology for shallow-depth systems utilizing conventional asymmetric steel I-beams (AIBs) that can be hot-rolled or built-up. The design methodology covers the limit states critical for construction in the noncomposite state and in-service as a composite system. The composite design presented is a lower-bound approach with no mechanical connectors. The horizontal shear transfer is through steel-concrete bond and friction, resulting in a partial composite design. This design methodology was developed through an experimental testing program on shallow-depth floor systems. The testing reduced the uncertainty of several design parameters and validated certain assumptions. Three main phases of testing were (1) an initial large-scale concept test, (2) nine composite beam tests, and (3) a full-scale system validation test. The critical findings of each phase are provided. Finally, design aids are presented to equip future engineers with resources to efficiently design shallow-depth floor systems.

Keywords: structural steel, composite, asymmetric, shallow, and floor system.

INTRODUCTION

The type of floor system selected for multistory residential buildings can be driven by the depth (to optimize floor-to-floor height) and speed of construction, both of which affect cost. Flat slab floor systems are pervasive and comprise reinforced concrete or post-tensioned concrete. These systems yield a relatively shallow depth. However, they require extensive forming, shoring, and effort for reinforcement placement, making them relatively slow to construct. An alternative presented herein is shallow-depth steel-concrete composite floor systems, where stay-in-place formwork [either steel deep decking, precast hollow-core (HC) panels, or cross-laminated timber (CLT) panels] is installed on the bottom flange of steel beams (shown in Figure 1). To facilitate easy placement of decking or panels, asymmetric I-beams (AIBs), comprised of a narrow top flange, are utilized. Following formwork placement, a concrete slab is cast-in-place, and the concrete encases the top

flange and web of the steel beams. These steel AIBs can either be built-up sections or hot-rolled shapes.

Various proprietary built-up asymmetric steel beams exist in the marketplace, such as the Slim Floor system by ArcelorMittal (ArcelorMittal, 2017), the Girder-Slab floor system (Girder-Slab Technologies, 2016), and the DELTA-BEAM system (DELTABEAM Technical Manual, 2019). The beams used in these systems are different than AIBs because they require substantial fabrication with web openings intended for rebar to be placed perpendicular to the beam.

An alternative to the built-up sections just mentioned are hot-rolled asymmetric I-shapes (ASBs) produced by British Steel and Corus Construction (now part of TATA Steel). The ASBs have been rolled in Europe and used in a floor system called Slimdek® (Lawson and Brekelmans, 1999; Lawson et al., 1997, 1998; Rackham et al., 2006). Historically, these types of shapes were not rolled in the United States. However, several U.S. steel mills are considering hot-rolled AIBs, termed A-shapes. Specific A-shape cross sections were recently developed as part of an American Institute of Steel Construction (AISC) study (Yarnold et al., 2025), which were sized for multistory residential (and light commercial) shallow-depth composite floor systems. The motivation was to increase the speed and efficiency of floor systems. Table 1 provides the dimensions for each shape. Note that if A-shapes are unavailable, the dimensions shown are recommended for efficient built-up sections.

The overarching objective of this paper is to provide engineers with the design methodology for shallow-depth

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Table 1. Hot-Rolled A-Shape Recommended Dimensions (Yarnold et al., 2025)

Shape	Area	Depth	Web	Top Flange		Bottom Flange		Nominal Weight*
			Thickness	Width	Thickness	Width	Thickness	
	in. ²	in.	in.	in.	in.	in.	in.	lb/ft
A12×108	31.8	12.0	$\frac{9}{16}$	11.0	$1\frac{1}{8}$	16.0	$1\frac{3}{16}$	108
A12×85	25.0	12.0	$\frac{9}{16}$	8.0	$1\frac{1}{8}$	13.0	$\frac{3}{4}$	85
A10×88	25.9	10.0	$\frac{9}{16}$	9.0	$1\frac{1}{8}$	14.0	$\frac{3}{4}$	88
A10×65	19.1	10.0	$\frac{1}{2}$	7.0	1	12.0	$\frac{5}{8}$	65
A8×72	21.1	8.0	$\frac{5}{8}$	6.5	$1\frac{1}{4}$	11.5	$\frac{3}{4}$	72
A8×52	15.3	8.0	$\frac{1}{2}$	6.5	$\frac{7}{8}$	11.5	$\frac{1}{2}$	52
A6×43	12.3	6.0	$\frac{1}{2}$	5.0	1	10.0	$\frac{1}{2}$	43

* Includes additional weight for the fillets.

composite floor systems utilizing AIBs within a steel frame building. This design methodology was developed through an extensive experimental testing program. A summary of the testing program is also provided herein to support several assumptions. In addition, design aids are provided to further support the future design of shallow-depth floor systems.

DESIGN METHODOLOGY

A comprehensive design methodology was developed for shallow-depth floor systems utilizing AIBs (hot-rolled or built-up). The methodology first addresses the calculations during construction in the noncomposite state. The shallow-depth system may utilize panels (HC or CLT) or deep decking to support the concrete during construction. Next, the methodology addresses the calculations of the completed floor system in the composite state. The method presented is a lower-bound approach with no mechanical connectors for horizontal shear transfer (only bond and friction). Therefore, the calculations are for a partial composite design. If additional mechanisms are provided, this will only increase the performance. However, the speed of fabrication or assembly may be reduced.

Noncomposite Design—Construction

Two primary construction scenarios are analyzed for the noncomposite AIB: (a) concentric loading from the slab pour on adjacent bays and (b) eccentric loading from the slab pour of a single bay (inducing torsion). Figure 2 graphically illustrates these scenarios. This section examines the analysis required for the design of AIBs during construction.

Concentric Loading

The main concentric loading scenario is immediately after the concrete slab is placed over the entire tributary area of the AIB [Figure 2(a)]. The noncomposite section supports the entire dead load (if unshored) along with a construction live load, typically 20 psf (ASCE, 2014). The load combination for this analysis includes a load factor of 1.4 for variable construction dead load and a construction live load factor of 1.6 (ASCE, 2014).

According to the AISC *Specification for Structural Steel Buildings*, hereafter referred to as the AISC *Specification*, four flexural limit states and a shear limit state should be evaluated for this noncomposite concentric loading (AISC, 2022). The flexural limit states (for an asymmetric I-beam) are covered in the AISC *Specification* Section F4, which

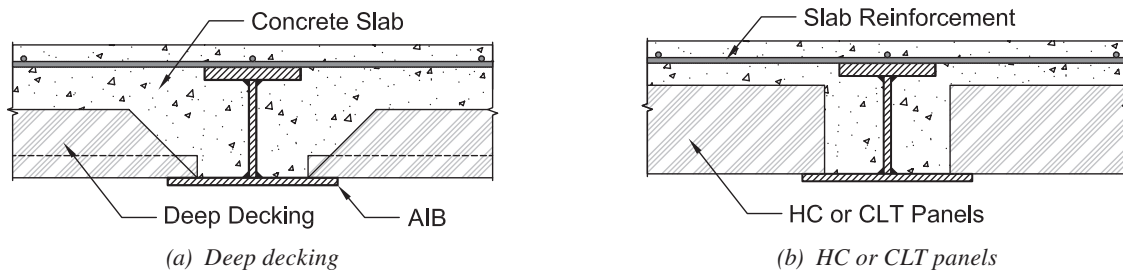


Fig. 1. Shallow-depth composite floor systems with AIBs.

includes compression flange yielding (CFY), tension flange yielding (TFY), compression flange local buckling (FLB), and lateral-torsional buckling (LTB). Treatment of CFY and TFY is relatively straightforward (refer to the AISC *Specification*). FLB is also relatively easy to apply; however, it is unlikely to control since AIB compression flanges are typically stocky and their width is narrow to facilitate panel or deep deck placement.

LTB is a critical limit state for AIBs in shallow-depth floor systems because the top flange is unbraced for the entire span before concrete curing. The AISC *Specification* LTB provisions can be strictly followed for AIBs, but this would be highly conservative. The authors recommend modification of these provisions to account for the load height effect.

The AISC *Specification* LTB provisions are derived for a load application at the beam centroid. For many situations, the applied load is at the top of the beam; however, the beam is usually braced with metal decking in typical composite floor systems. If the load is applied to the top flange with significant unbraced lengths, as the beam begins to buckle out-of-plane, a secondary moment is developed due to the rotation of the beam, resulting in an additional torsional moment in the direction of buckling [illustrated in Figure 3(a)]. However, if the load is applied to the bottom flange, a restoring torsional moment is formed in the opposite direction, which is shown in Figure 3(b).

To account for this load height effect, an adjustment is made to the LTB modification factor, C_b (AISC, 2022), through the load height modification factor, C_b^* (Helwig et al., 1997). This factor is a function of the warping stiffness of the cross section, W_s , and the type of loading. The warping stiffness parameter is evaluated in Equation 1 using the weak-axis moment of inertia, I_y ; the distance between flange centroids, h_o ; the shear modulus, G ; the modulus of elasticity of steel, E_s ; and the torsional constant of the section, J . Note that the warping stiffness, W_s , calculation in Equation 1 is for doubly symmetric I-shapes. This decision was made due to the recommendation by Helwig et al. (1997).

The load height modification factor, C_b^* , with load applied on the bottom flange (resultant centered) for a uniformly distributed load is expressed in Equation 2. C_b^* increases C_b from 10 to 35% (depending on the geometry), which directly increases the LTB strength.

$$W_s = \frac{\pi}{L_b} \sqrt{\frac{E_s I_y h_o^2}{4GJ}} \quad (1)$$

$$C_b^* = C_b (1 - 0.154W_s^2 + 0.535W_s) \quad (2)$$

The shear limit state is addressed in AISC *Specification* Section G2. The nominal shear strength considers the shear area of the web, the yield stress of the steel, and the web shear strength coefficient. The vertical shear strength

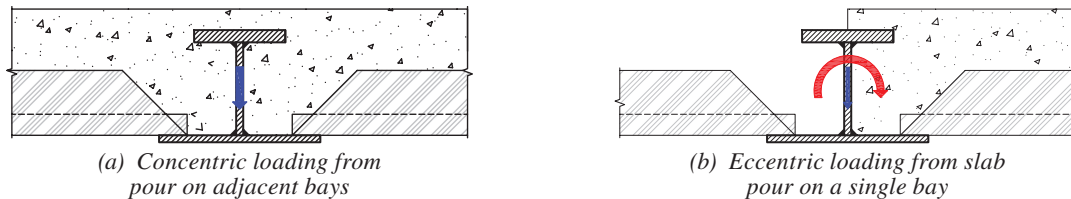


Fig. 2. Primary construction scenarios.

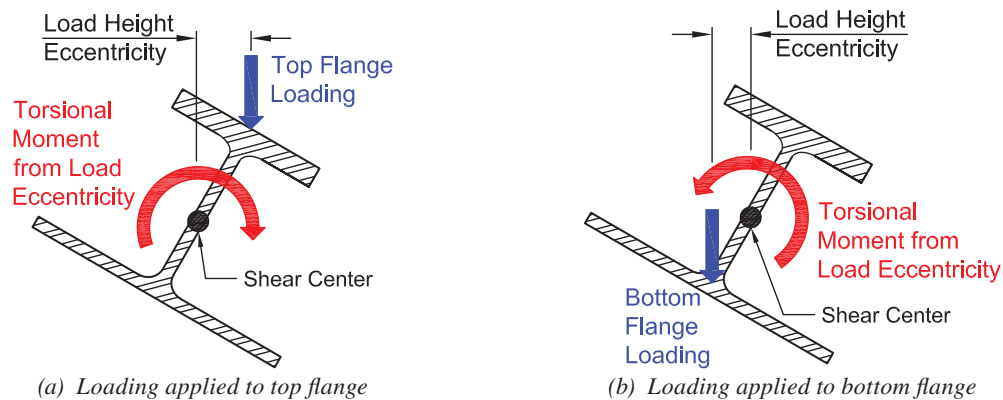


Fig. 3. Load height effect.

of AIBs is typically not a concern for A-shapes due to the web thicknesses. For built-up AIBs, the web thickness must be selected to satisfy AISC *Specification* Section G2 provisions.

Eccentric Loading

The primary eccentric loading scenarios (for unshored construction) are when panels (HC or CLT) are placed entirely on one side of the AIB or when the concrete slab is poured over a single bay [Figure 2(b)]. These scenarios induce normal stresses and shear stresses that must meet the limits provided in AISC *Specification* Section H3.3. These provisions include stress limits for yielding, along with buckling. The challenging portion of this analysis is the calculation of the required stresses. These stress calculations are summarized below. In addition, the angle of twist of the section should be considered. Guidance on this is also provided.

Note that bottom flange local bending should not control for the A-shapes provided in Table 1. Therefore, this calculation is not provided below. However, this should be checked for excessively large construction loading or sections with a reduced bottom flange thickness.

Normal Stress Yielding and Buckling

For the normal (or longitudinal) stress calculations, it is recommended to analyze the in-plane bending component separately from the torsional moment and then obtain the combined effect using superposition. Figure 4 illustrates the flange stress profiles due to flexure and torsion.

Calculation of the elastic in-plane bending stresses, σ_b , is relatively straightforward using Equation 3, which only includes the required bending moment, M_r , and the elastic section modulus, S_x .

$$\sigma_b = \frac{M_r}{S_x} \quad (3)$$

When analyzing a beam under torsion, it is essential to evaluate the two torsional moments resisted by the cross section. The first, given in Equation 4, is the uniform (or pure) torsion, also known as St. Venant torsion. The relationship between the first derivative of the angle of twist, θ' , and the torsional moment, T_t , is defined using the shear modulus of elasticity, G , and the torsional constant, J .

$$T_t = GJ\theta' \quad (4)$$

For noncircular cross sections such as AIBs, longitudinal deformation due to torsion (termed warping) is restrained from cross-flange bending. A torsional moment due to restrained warping of the section, T_w , is present. The extent of torsion caused by warping resistance is mathematically defined in Equation 5, which is the product of the modulus of elasticity, E_s ; warping constant, C_w ; and the third derivative of the angle of twist, θ''' . The warping constant for singly symmetric I-beams is provided in Equation 6, which is comprised of the distance between flange centroids, h_o , and the moment of inertia of the top flange about the y-axis, I_{yt} , and the moment of inertia of the bottom flange about the y-axis, I_{yb} .

$$T_w = -E_s C_w \theta''' \quad (5)$$

$$C_w = h_o^2 \frac{I_{yt} I_{yb}}{I_{yt} + I_{yb}} \quad (6)$$

The combined torsional resistance of the section, T_r , is provided in Equation 7. A detailed background of these expressions and section property definitions can be found in AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 2003).

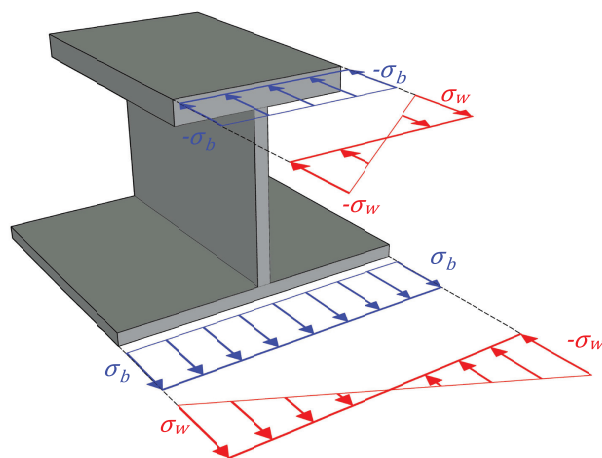


Fig. 4. Flange stresses due to bending and warping torsion.

$$T_r = GJ\theta' - E_s C_w \theta''' \quad (7)$$

Ultimately, the beam's angle of twist, θ , must be computed, along with the corresponding derivatives. The first step is to rearrange Equation 7 by dividing by the warping rigidity of the cross section, EC_w , resulting in Equation 8. Then, a parameter definition, a^2 , is introduced (shown in Equation 9). The resulting expression is provided in Equation 10, which is for a concentrated torsional moment, T_r .

$$\frac{T_r}{EC_w} = \frac{GJ\theta'}{E_s C_w} - \theta''' \quad (8)$$

$$a^2 = \frac{E_s C_w}{GJ} \quad (9)$$

$$\frac{T_r}{EC_w} = \frac{\theta'}{a^2} - \theta''' \quad (10)$$

Equation 10 can be revised to analyze a uniformly distributed torsional moment, t , as shown in Equation 11. The solution for this differential equation is conveyed in Equation 12 (Seaburg and Carter, 2003).

$$\frac{-t}{EC_w} = \frac{\theta''}{a^2} - \theta'''' \quad (11)$$

$$\theta = A + Bz + C \cosh \frac{z}{a} + D \sinh \frac{z}{a} - \frac{tz^2}{2GJ} \quad (12)$$

A simplification can be made by applying support conditions and solving for the constants, A , B , C , and D , where l is the beam length, and z is the location on the beam where θ is evaluated. For this analysis, θ and θ'' are needed for the calculation of the beam rotation and stresses, respectively. Equations 13 and 14 provide the resulting equations for torsionally pinned support conditions.

$$\theta = \frac{ta^2}{GJ} \left[\frac{l^2}{2a^2} \left(\frac{z}{l} - \frac{z^2}{l^2} \right) + \cosh \frac{z}{a} - \tanh \frac{l}{2a} \sinh \frac{z}{a} - 1.0 \right] \quad (13)$$

$$\theta'' = \frac{ta^2}{GJ} \left(\frac{1}{a^2} - \frac{\cosh \frac{z}{a}}{a^2} + \frac{\sinh \frac{z}{a} \tanh \frac{l}{2a}}{a^2} \right) \quad (14)$$

The longitudinal warping stresses, σ_w , are calculated using Equation 15, which requires the normalized warping constant, W_{no} . For an AIB, this value is different with respect to the top and bottom flanges. Equations 16 and 17 provide the normalized warping constant for the top flange, W_{not} , and bottom flange, W_{nob} (Heins, 1975). The required dimensional parameters shown in these equations include the location of the shear center from the centroid of the top flange, a (provided in Equation 18), the distance between the centroids of the flanges, h_o , the width of the top flange, b_{ft} , and the width of the bottom flange, b_{fb} .

$$\sigma_w = E_s W_{no} \theta'' \quad (15)$$

$$W_{not} = \frac{\alpha b_{ft}}{2} \quad (16)$$

$$W_{nob} = \frac{h_o - \alpha}{2} b_{fb} \quad (17)$$

$$\alpha = \frac{b_{fb}^3 t_{fb}}{b_{fb}^3 t_{fb} + b_{ft}^3 t_{ft}} h_o \quad (18)$$

As mentioned, stresses must be accurately combined to calculate the required normal stresses, f_r , at the critical points along the cross section (see Equation 19). These stresses must meet AISC *Specification* Section H3.3 limits.

$$f_r = \sigma_a \pm \sigma_b \pm \sigma_w \quad (19)$$

Shear Stress Yielding

Similar to the calculation of the normal stresses, shear stresses should be calculated using superposition. The shear stress due to in-plane loading, τ_b , can be calculated as shown in Equation 20, which is comprised of the applied shear demand, V , first moment of the area, Q , moment of inertia, I_x , and thickness of the element, t .

$$\tau_b = \frac{VQ}{I_x t} \quad (20)$$

The shear stresses due to pure torsion, τ_t , and warping torsion, τ_w , can be found with Equations 21 and 22, respectively. Another torsional property, S_w , is introduced in the warping equation, which is defined with respect to the top and bottom flanges in Equations 23 and 24, respectively. In these equations, the thickness of the top flange, t_{ft} , and the thickness of the bottom flange, t_{fb} , are needed. Also required for these equations are the first and third derivatives of the angle of twist (θ' and θ''' , respectively), which are provided in Equations 25 and 26. Note this is for pinned supports and uniformly distributed loading.

$$\tau_t = Gt\theta' \quad (21)$$

$$\tau_w = \frac{E_s S_w \theta'''}{t} \quad (22)$$

$$S_{wt} = \frac{W_{not} b_{ft} t_{ft}}{4} \quad (23)$$

$$S_{wb} = \frac{W_{nob} b_{fb} t_{fb}}{4} \quad (24)$$

$$\theta' = \frac{ta^2}{GJ} \left[\frac{l^2}{2a^2} \left(\frac{1}{l} - \frac{2z}{l^2} \right) + \frac{\sinh \frac{z}{a}}{a} - \frac{\cosh \frac{z}{a} \tanh \frac{l}{2a}}{a} \right] \quad (25)$$

$$\theta''' = \frac{ta^2}{GJ} \left(\frac{\sinh \frac{z}{a}}{a^3} - \frac{\cosh \frac{z}{a} \tanh \frac{l}{2a}}{a^3} \right) \quad (26)$$

To calculate the shear stresses, f_v , at different points along the cross section, stresses must be accurately combined (see Equation 27). These stresses must meet the AISC *Specification* Section H3.3 limits.

$$f_v = \tau_b \pm \tau_t \pm \tau_w \quad (27)$$

Angle of Twist

It is recommended to check the maximum angle of twist of the section, θ , calculated from Equation 13, which typically occurs at mid-span. This twist check is for constructability. Engineering judgment should be used for the allowable twist limit. This limit may be different for panelized systems versus deep decking. The development of the A-shape cross sections utilized a 5.0° limit. However, this is not a strict limit and is left to the discretion of the designer for their situation.

Composite Design—In-Service

The final composite system must meet strength and serviceability criteria. The required loading and load factors are provided in ASCE/SEI 7, *Minimum Design Load and Associated Criteria for Buildings and Other Structures* (2022). The required strength and serviceability structural checks are provided in the following sections. In addition, a discussion on fire resistance is included.

Strength Checks

The flexural strength of a partially concrete-encased AIB (compression flange and web), without mechanical connectors, is based on a partial composite design that includes two limit state checks. The first is the composite strength based on the bond shear strength between the steel-concrete interface. The second is the longitudinal shear strength of the concrete (only applicable for panel systems without

chamfers). These limit states and the corresponding calculations are provided later in this subsection.

The full composite moment strength must be determined since it is needed for the partial composite strength calculations. To perform this analysis, the effective width of the section, B_e , can be found using AISC *Specification* Section I3.1a. The composite section properties must be calculated for an effective section of the floor system (see Figure 5 for the parameter nomenclature). A plastic stress distribution is assumed for the full composite flexural strength. The steel AIB resists tensile demands below the plastic neutral axis (PNA). The compressive demands are resisted by the AIB above the PNA and the concrete topping slab, which has an effective width, B_e , and depth, D_c . The PNA of the composite section is determined by ensuring that the compressive forces in the concrete, C_c , and steel, C_s , are in equilibrium with the tensile force, T , from the steel cross section below the PNA.

The equations determining the compressive and tensile force resultants depend on the PNA location. There are five possible cases for the PNA. The first three cases are when the top of the precast panels or deep decking is below the bottom of the top flange. The PNA is located in (1) the steel top flange, (2) in the steel web above the top of panels/deep decking, or (3) in the steel web but below the top of panels/deep decking. The final two cases consider whether the top of the panels/deep decking is above the bottom of the top flange. These cases consist of the PNA located in the (4) steel top flange or (5) the web.

The PNA generally falls into the steel top flange, which means not all the topping slab is in compression (see Figure 5). For this case, the compressive force resultants C_c and C_s are determined with Equations 28 and 29, respectively. The compressive strength of the concrete, f'_c , is needed, along with the concrete cover above the AIB, D_c . The tensile force resultant is calculated with Equation 30, where A_s is the area of the steel in tension and F_y is the yield stress of the steel. The depth of the PNA, y_{pna} , is found by iteratively solving the equations below until equilibrium is satisfied.

$$C_c = 0.85f'_c [B_e D_c + (B_e - b_{fl})(y_{pna} - D_c)] \quad (28)$$

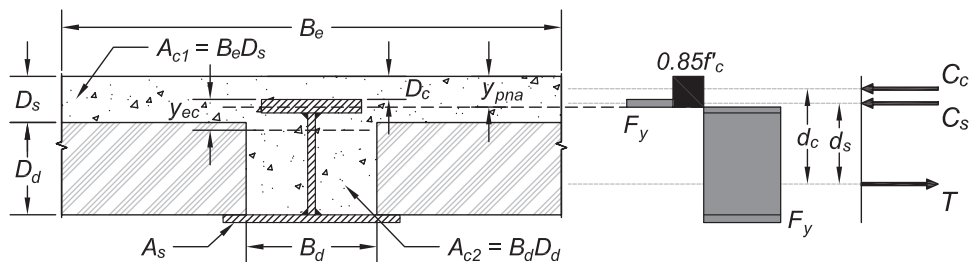


Fig. 5. Composite section nomenclature, plastic stress profiles, and force resultants.

$$C_s = F_y b_{ft} (y_{pna} - D_c) \quad (29)$$

$$T = A_s F_y - C_s \quad (30)$$

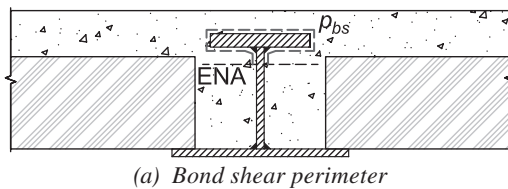
The force resultants are located at their respective centroidal locations, which are determined after the PNA is identified. These centroidal locations are calculated using standard geometric decomposition. Similar equations can be derived for the force components and their locations for the other PNA locations. The full composite plastic moment strength, M_{ncp} , is determined in Equation 31, which is the summation of moments due to the compressive forces in the concrete and steel about the tensile force centroid, with a moment arm of d_c and d_s , respectively.

$$M_{ncp} = C_c d_c + C_s d_s \quad (31)$$

A partial composite system will not achieve the full composite flexural strength as shown in Equation 31. However, a partial composite system can achieve significant flexural strength through the bond between the steel and the concrete. The steel-concrete bond is assumed to act around the top flange and web above the elastic neutral axis (ENA) with a uniform stress distribution (Lawson et al., 1997). The cross section will likely not reach the plastic state due to localized bond failure.

The ENA location, y_{ec} , is referenced from the top of the steel section as shown in Figure 5. The ENA can be determined by splitting the section into three areas: the AIB area, A_s , the topping slab above the panels or the top of the deep decking, A_{c1} , and the beam encasement area, A_{c2} . For the beam encasement area, the width of the encasement, B_d , and the depth of encasement, D_d , are needed (as shown in Figure 5). The ENA, y_{ec} , is determined with standard geometric decomposition, which results in Equation 32. This equation also includes the modular ratio, n , which is simply the ratio between the modulus of elasticity of the steel and concrete.

$$y_{ec} = \frac{A_s y_e + \frac{A_{c1}}{n} \left(\frac{D_s}{2} - D_c \right) + \frac{A_{c2}}{n} \left(d - \frac{D_d}{2} - t_{fb} \right)}{A_s + \frac{A_{c1}}{n} + \frac{A_{c2}}{n}} \quad (32)$$



(a) Bond shear perimeter

Bond Shear Strength

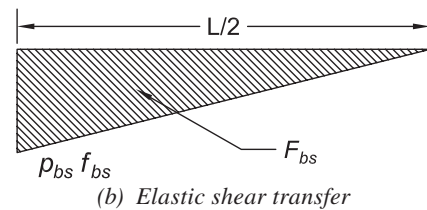
The first partial composite flexure limit state must determine the maximum compressive force that can be transferred between the concrete and steel from the bond shear, F_{bs} . This can be considered the longitudinal force to “push” the steel through the concrete when under flexure. Elastic shear flow is used for beams subjected to a uniformly distributed load. See Figure 6(b) for an illustration of the force transferred due to the formation of bond shear.

The bond shear strength is dependent on the bond shear perimeter, p_{bs} , which is illustrated in Figure 6(a) and calculated in Equation 33. The strength is also determined by the allowable bond shear stress, f_{bs} . The allowable bond shear stress recommended is 100 psi, which was determined through extensive experimental testing (further test details provided later). The research team adopted this allowable bond stress, which is the same value for filled composite members with rectangular cross sections (AISC, 2022). In literature, there is a wide range of estimates for the allowable bond shear, some of which are a function of f'_c (McCormac, 1981). However, experimental push-out tests show no distinct correlation between the allowable bond shear and f'_c (Roeder et al., 1999). The maximum compressive force resistance from the bond shear, F_{bs} , for simple span beams under uniformly distributed loading, is determined by calculating the resultant force that is transferred over half of the length of the beam [see Equation 34 and Figure 6(b)].

$$p_{bs} = 2(b_{ft} + y_{ec}) - t_w \quad (33)$$

$$F_{bs} = p_{bs} f_{bs} \frac{L}{4} \quad (34)$$

The partial composite moment strength can be calculated using several methods (further discussed later). The most conservative method linearly interpolates the strength between the steel-only yield moment, M_{sy} , and the full composite moment strength, M_{ncp} . Regardless of the determined flexural strength for the noncomposite (steel-only) section during the construction limit states, the steel-only yield moment is implemented as the minimum strength. This is because stability is not of concern since the concrete



(b) Elastic shear transfer

Fig. 6. Bond shear illustration.

has cured, providing continual section bracing. Therefore, the steel can reach the minimum yield moment strength of the compression or tension flange. For AIBs, the compression flange yield moment is determined with the steel yield strength, F_y , and the elastic section modulus of the top flange, S_{xc} , will control due to the asymmetry. Furthermore, the web plastification factor, R_{pc} , is also included per AISC *Specification* Section F4. This factor often increases the strength of the AIB to the full steel plastic moment strength. The minimum compression flange yield moment is defined in Equation 35.

$$M_{sy} = R_{pc} F_y S_{xc} \quad (35)$$

The percentage of composite action is determined by evaluating the maximum allowable compressive force resistance from the bond shear, F_{bs} , to the concrete compressive force, C_c , that forms in the slab if full composite action is achieved. See Equation 36 for the linear interpolation utilized to determine the partial composite moment strength, M_{nc} (Lawson et al., 1997).

$$M_{nc} = M_{sy} + \frac{F_{bs}}{C_c} (M_{ncp} - M_{sy}) \quad (36)$$

Aside from the linear interpolation method, other methods can be utilized to calculate the partial composite flexural strength. This research explored the partial plastic stress distribution method, which uses a plastic stress distribution approach. However, the compressive force resultant from the concrete is limited by the bond shear force. The other force resultants are found from equilibrium; then the resulting moment can be calculated. The research also considered the strain compatibility method. This method assumes a linear distribution of strains over the entire depth of the composite cross section. Unlike in the previous method, stress-strain models of concrete and steel are utilized in this procedure rather than a plastic stress distribution. The comparison of all three methods with composite beam testing (discussed later) found that the linear interpolation was accurate and slightly conservative (Ottmers et al., 2025a).

Longitudinal Concrete Shear Strength

The second limit state applies to HC or CLT panels that do not have a chamfered top edge, as shown in Figure 7. This limit state considers the failure mode from the panel to the edge of the top flange (line A-B; d_{A-B}), across the width of the top flange (line B-C), and down to the panel edge on the other side of the beam (line C-D; d_{C-D}), shown by the dashed line in Figure 7. The longitudinal shear force in the cross section is transferred by the allowable bond stress between the concrete and steel (100 psi recommended from full-scale testing, which is presented later) and the allowable concrete shear stress, f_{cs} , in Equation 37 (McCormac, 1981). The longitudinal shear available strength per unit length, v_{bs} , is quantified with Equation 38.

$$f_{cs} = 0.12 f'_c \quad (37)$$

$$v_{bs} = f_{cs} d_{A-B} + f_{bs} b_{ft} + f_{cs} d_{C-D} \quad (38)$$

The longitudinal shear limit state check loading demand depends on whether the floor system is shored during construction. Unshored construction is likely to be used for floor systems that utilize panels due to the panel span capabilities. Therefore, the load demand resisted by the longitudinal shear for unshored construction is only due to the in-service live load.

To evaluate the longitudinal shear required strength per unit length, v_d , the maximum shear flow due to the load demand is obtained. The shear flow is dependent on the maximum vertical shear force, V , due to live (and possibly dead) loads. The first moment of area of the concrete, Q_c , above line A-B-C-D in Figure 7, and the moment of inertia of the composite section, I_{xc} (shown later), are also utilized (see Equation 39).

$$v_d = \frac{V Q_c}{I_{xc}} \quad (39)$$

Serviceability Checks

The primary serviceability requirements for floor systems include deflections and vibration to ensure comfort for building residents. These criteria control in many cases for

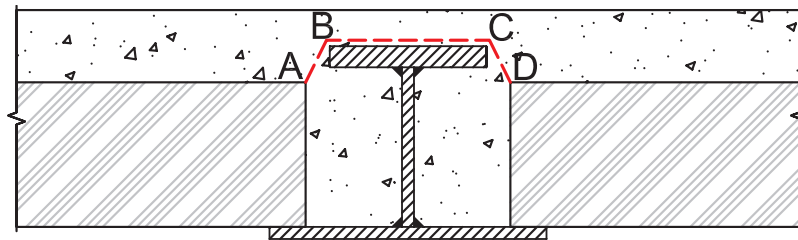


Fig. 7. Longitudinal shear transfer.

shallow-depth floor systems (even with a partial composite flexural strength). Designing for service deflections and vibrations is presented next.

Deflections

Vertical deflections of a floor system vary depending on whether shored or unshored construction is performed. Calculating deflections for unshored construction requires the AIBs to support the construction dead loads fully. The dead load deflection, δ_{DL} , due to uniformly distributed noncomposite dead load, w_{DL} , is evaluated with the AIB moment of inertia, I_x , as shown in Equation 40.

A common industry practice is to camber the beams, δ_{Camber} , to satisfy the deflection criteria. Cambering the beams also produces a relatively flat floor. Specifying camber is accomplished with approximately 80% of the dead load to ensure the camber is removed from the beam. However, a minimum camber of 0.75 in. is commonly required by fabricators. Further guidance on camber can be found in AISC Design Guide 36, *Design Considerations for Camber* (Kloiber and Burmeister, 2020).

The live load deflection, δ_{LL} , and superimposed dead load deflection, δ_{SDL} , are calculated according to Equations 41 and 42, respectively. The uniformly distributed live load, w_{LL} , and superimposed dead load, w_{SDL} , are evaluated with the uncracked transformed composite moment of inertia, I_{xc} . Note that if shored construction is used, the concrete will cure in the shored position. As a result, a portion of the dead deflection is determined with composite section properties.

$$\delta_{DL} = \frac{5w_{DL}L^4}{384E_sI_x} - \delta_{camber} \quad (40)$$

$$\delta_{LL} = \frac{5w_{LL}L^4}{384E_sI_{xc}} \quad (41)$$

$$\delta_{SDL} = \frac{5w_{SDL}L^4}{384E_sI_{xc}} \quad (42)$$

The moment of inertia for the uncracked composite transformed section, I_{xc} , can be quantified with Equation 43. Notice that the full composite section is utilized to capture the stiffness of the floor system. The full-scale testing (discussed later) found this to be the most accurate, yet conservative, assumption.

$$I_{xc} = I_x + A_s(y_e - y_{ec})^2 + \frac{A_{c1}D_s^2}{12n} + \frac{A_{c1}}{n} \left(\frac{D_s}{2} - D_c - y_{ec} \right)^2 + \frac{A_{c2}D_d^2}{12n} + \frac{A_{c2}}{n} \left(d - \frac{D_d}{2} - t_{fb} - y_{ec} \right)^2 \quad (43)$$

To ensure resident comfort, deflections from all dead and live loads (summation of Equations 40–42) must be less than the beam span length, L , divided by 240, and the live load deflection must be less than the span length divided by 360 (IBC, 2018). Note that the total deflection limit should also include concrete creep effects, which are not included here.

Vibration

Vibrations in steel floor systems can cause discomfort to residents, depending on their activity. To satisfy the vibration limit state, the ratio of the peak floor acceleration to gravity, a_p/g , of the floor system due to loading must be below the tolerated vibration levels for human comfort, a_p/g (Murray et al., 2016).

Similar to the deflection calculations, the composite uncracked transformed moment of inertia should be computed using the recommendations of AISC Design Guide 11, *Vibrations of Steel-Framed Structural Systems due to Human Activity* (Murray et al., 2016). However, the modulus of elasticity of concrete is increased by 35%, which modifies the modular ratio for vibration, n_v . The modular ratio for vibration replaces the modular ratio, n , in Equation 43 to quantify the increase in stiffness of the concrete deck during dynamic loading compared to static loading.

The natural frequency, f_n , is estimated based on the deflection of the floor system in Equation 44. Vibrations are usually the most severe when the floor is lightly loaded; therefore, expected day-to-day dead and live loads are utilized. Loadings consist of the structural dead load, an additional 4.0 psf for mechanical and ceiling installations when present, and live load for the applicable occupancy, as recommended by AISC Design Guide 11. The deflection for vibration, δ_v , is quantified similarly to the calculations in the prior section, but with the vibration-altered composite moment of inertia.

$$f_n = 0.18 \sqrt{\frac{g}{\delta_v}} \quad (44)$$

The type of loading producing the most severe dynamic response is dependent on the natural frequency of the floor system. For low-frequency floors, $f_n < 9$ Hz, the dynamic response is governed by the floor undergoing resonance from continuous footstep. For high-frequency floors, $9 \text{ Hz} < f_n < 15 \text{ Hz}$, the impulse response due to single footsteps governs the dynamic response. Floor systems with a steel deck and concrete slab having a natural frequency above 15 Hz are not known to be prone to excessive vibration (Murray et al., 2016).

The acceleration ratio, a_p/g , due to resonance is quantified in Equation 45. The amplitude of the driving force,

P_o , is a constant value of 65 lb. A damping ratio, β , ranges from 1 to 5% considering the structural system, ceiling and ductwork, furniture in the residence, and possible partitions (Murray et al., 2016). The effective panel weight, W , supported by the AIB is also needed.

$$\frac{a_p}{g} = \frac{P_o e^{-0.35 f_n}}{\beta W} \quad (45)$$

Predicted acceleration, a_p/g , is quantified in Equation 46 utilizing the damping ratio, β , floor natural frequency, f_n , and the effective panel weight, W . The footstep frequency, f_{step} , is determined based on the step frequency harmonic matching the natural frequency, h_s (Murray et al., 2016).

$$\frac{a_p}{g} = \frac{154}{W} \frac{f_{step}^{1.43}}{f_n^{0.3}} \sqrt{\frac{1 - e^{-4\pi h_s \beta}}{h_s \pi \beta}} \quad (46)$$

The peak floor acceleration response, either from resonance or impulse, is quantified and evaluated against the tolerant vibration levels for human comfort, a_p/g .

Fire

It is expected that some form of fire protection will be applied to the bottom flange of the AIBs (e.g., gypsum board, cementitious coatings, or intumescent coatings). The AIBs within shallow-depth floor systems have an inherent fire resistance due to concrete encasement. However, sustained high temperatures in the bottom flange can reduce the strength contribution of the bottom of the section. Therefore, fire protection is recommended, unless full-scale fire testing proves otherwise.

A lower-bound numerical fire study was performed where the AIB bottom flange was unprotected (Yarnold et al., 2025). The system performed adequately during fire events for an array of building geometries. As part of the numerical study, two strategies were evaluated for improved fire performance. The first was an increased AIB web thickness, and the second was fire reinforcement near the web of

the AIB (shown in Figure 8). Overall, it is recommended to add steel fire reinforcement as shown in Figure 8, instead of increasing the AIB web thickness. Fire reinforcement has a more significant impact on the performance while adding minimal cost. Note that this longitudinal reinforcement also increases the composite flexural capacity, which can be accounted for within the in-service design.

EXPERIMENTAL TESTING SUMMARY

An array of experimental testing was performed for AIBs within shallow-depth floor systems. The objective of the testing was to reduce the uncertainty of several design parameters and validate certain assumptions. Three main phases of testing were performed, which are summarized in this section.

Initial Concept Testing

The purpose of the initial large-scale testing was a preliminary constructability assessment of a shallow-depth steel-concrete composite floor system utilizing AIBs, along with a quantitative evaluation of the structural performance during construction and service loading. An initial proof-of-concept test was performed to analyze the behavior with precast HC panels. Figure 9 provides a typical cross-section detail. The AIBs utilized were a wide-flange shape with flanges cut down. These sections were available from earlier research (Stoddard and Yarnold, 2022).

Measurements were recorded throughout the construction process and during the simulated in-service loading. Figure 10 shows the setup during the final loading application. The primary takeaway from this work was that AIBs supporting precast panels and a concrete deck are a viable option for building floor systems. A detailed account of the initial concept testing and the results can be found in Davis et al. (2023). The critical findings included:

- Shallow-depth floor systems are easy and fast to construct by placing precast concrete panels on the bottom flange

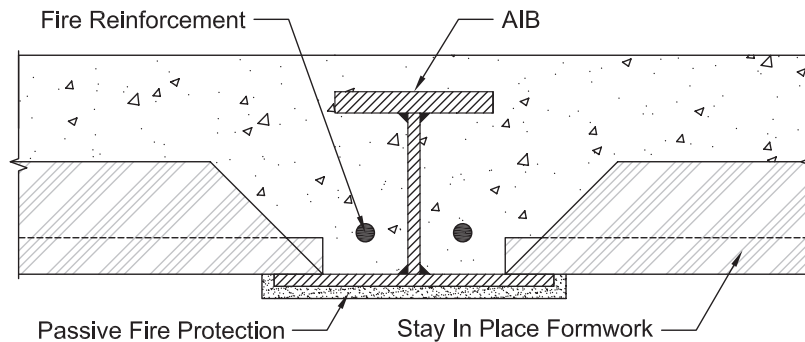


Fig. 8. Shallow-depth composite floor systems with fire reinforcement.

of an AIB and casting a deck slab that tops the panels and encases the steel beam.

- The noncomposite AIBs performed well in terms of lateral-torsional buckling and torsional response due to loading the beams on the bottom flange, creating a stabilizing load-height effect. The measured out-of-plane rotation was below the theoretical values.
- The floor system had a stiff response during service-level loading, indicating that the moment of inertia of the gross composite section can be utilized for serviceability analysis.
- Significant partial composite strength was achieved through bond shear forming between the steel beam and concrete. The composite floor system failed at an equivalent load of 500 psf applied using four-point bending over the center beam.

- Due to the beam spacing (limited by laboratory constraints) and the concrete not encasing the entire edge beam, the edge beams failed first due to load distribution from the center to the edge beams. Therefore, component-level beam tests and a full-scale system test with wider beam spacings were later utilized to analyze the composite behavior developed through bond stress for shallow-depth composite floor systems.

Composite Beam Testing

This testing aimed to evaluate the composite stiffness and strength of the shallow-depth steel-concrete composite floor system by conducting nine component-level beam tests (eight primary and one later validation). A detailed account of the initial eight tests can be found in Ottmers et al. (2025a), and the additional validation test can be found in Ottmers et al. (2025b).

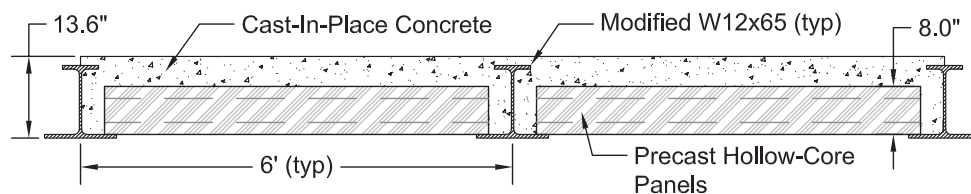


Fig. 9. Initial concept test cross section (slab reinforcement note shown).



Fig. 10. Initial concept testing.

Overall, the composite beam tests were designed to better understand the bond strength between the concrete and unpainted steel to address the uncertainty in composite action gained without mechanical connectors. Furthermore, the predicted partial composite flexural strength was analyzed using three methods: (1) linear interpolation between the steel and full composite moment strength, (2) partial plastic stress distribution, and (3) strain compatibility. These methods were compared to the experimental composite strength to establish the best methodology to analyze partially encased composite beams.

The composite beam tests were performed in three main stages: (1) lower load levels to analyze elastic behavior, (2) primary test cycle to define the partial composite strength, and (3) additional test cycles to analyze the reserve strength and ductility of the composite beams. Furthermore, the flexural stiffness of the composite section was analyzed at varying load levels. Figure 11 illustrates a component test and a sample loading time-history plot, with the three loading stages denoted. The steel beam yield moment, M_{sy} , and plastic moment, M_{sp} , are provided, along with the full composite strength, M_{ncp} , all using measured properties. The main conclusions drawn from the composite beam testing include:

- The flexural stiffness of the composite beams remained intact throughout the test, and the gross moment of inertia of the transformed section can be utilized for serviceability checks.
- The component-level beams reached 74 to 83% of the full composite strength in stage two loading before minimal bond slip occurred or a large flexural crack formed

(denoted as M_{fail}). The strength was well above the steel beam's plastic moment strength in all tests.

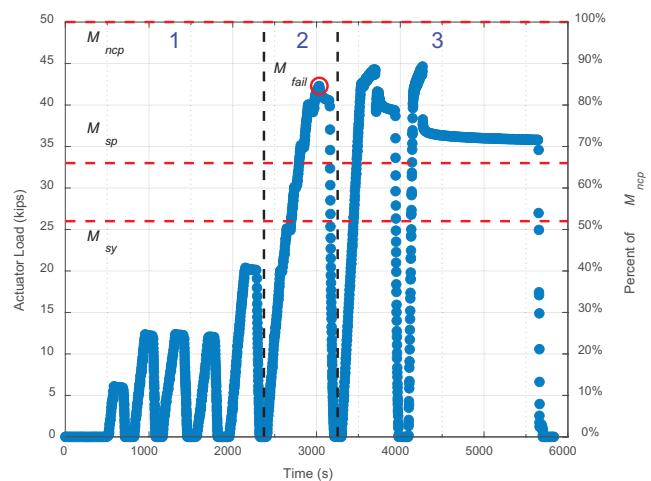
- The beams were reloaded in stage three and reached 77 to 91% of the full composite strength due to the bond reengaging between the steel and concrete, proving the ductile and robust behavior of the composite section.
- A bond stress of 100 psi undercuts most experimental results quantified with the maximum moment in stage two (111 psi average with 98 psi and 132 psi minimum and maximum, respectively). The constant bond stress was assumed to act over a bond perimeter above the composite section's elastic neutral axis. The maximum load transfer developed through bond shear is quantified as explained earlier in the Design Methodology section.
- The partial composite strength can be conservatively calculated using the linear interpolation method with a bond stress of 100 psi (this approach is recommended for future design with unpainted steel as outlined in the Design Methodology section). The partial plastic stress distribution and strain compatibility methods also provide reasonable results compared to the stage three experimental capacities.

System Validation Testing

A full-scale laboratory experiment was conducted to validate the use of AIBs within shallow-depth composite floor systems in multistory residential and commercial buildings. A prototype built-up plate girder equivalent to the A8×52 cross section (Ottmers et al., 2025c) was utilized in a system



Fig. 11. Example composite beam test under loading and load time-history plot.



test (dimensions shown in Table 1). Figure 12 shows the overall layout of the test specimen. The cross section was similar to that shown in Figure 1(a).

The system test setup was essentially two bays of a building. This consisted of three unpainted beams spanning between stub columns spaced 20 ft on center in both directions. New Millennium 4.5-in.-deep decking (18 gauge) was used to span over 19 ft between the A8×52 beams (Figure 13). A concrete slab was then cast monolithically to produce approximately a 10-in.-deep floor system that was 20 ft × 40 ft (center-to-center of the columns). A photo of the full-scale shallow-depth floor system is shown in Figure 14. The load frames shown were moved to various positions to conduct multiple loading scenarios. A detailed account of the full-scale validation testing design and execution can be found in Ottmers et al. (2025b).

The AIB behavior was evaluated throughout construction. This included the system constructability and the structural performance during the concrete slab pour. Next, vibration testing of the floor system was performed through heel drops, jumping, and walking scenarios. The floor system was then loaded in multiple service load locations to evaluate the stiffness. Finally, the center AIB was loaded until failure to determine the ultimate flexural strength (loading position shown in Figure 14). The main findings from the full-scale system testing included the following:

- The floor system was easy to construct, and the AIBs performed well structurally during construction. The measured deformations and rotations were below theoretical magnitudes with no signs of distress.

The deep decking was shored at midspan, per the manufacturer's requirements, which reduced the AIB demands. The maximum strain measured during the slab pour corresponded to a stress of 7.0 ksi.

- Vibration testing was performed in the constructed state to determine the fundamental frequencies and mode shapes. The first two natural frequencies of the floor system were 8.1 Hz and 9.3 Hz, and corresponded to the bending modes of the slab.
- Service-level testing was conducted in four locations around the slab to analyze the serviceability. This included loading at two locations near the midspan of the decking, close to the edge beams, and near the center beam. For midspan deck loading, equivalent to 80 psf, the maximum measured deflection was only $L/690$, indicating adequate deck stiffness.
- The AIB flexural stiffness was evaluated by loading directly over the center beam (shown in Figure 14). For a 100 psf equivalent area load over the center beam, the deflection of the floor system corresponds to $L/740$. The floor system had a relatively stiff response and further confirmed that the gross moment of inertia should be utilized for shallow-depth composite floor systems' stiffness calculations.
- The center AIB flexural performance from ultimate loading was excellent. The composite beam had a flexural stiffness greater than the gross moment of inertia of the composite section, quantified as two times the AISC *Specification* Section I3 specified effective width.

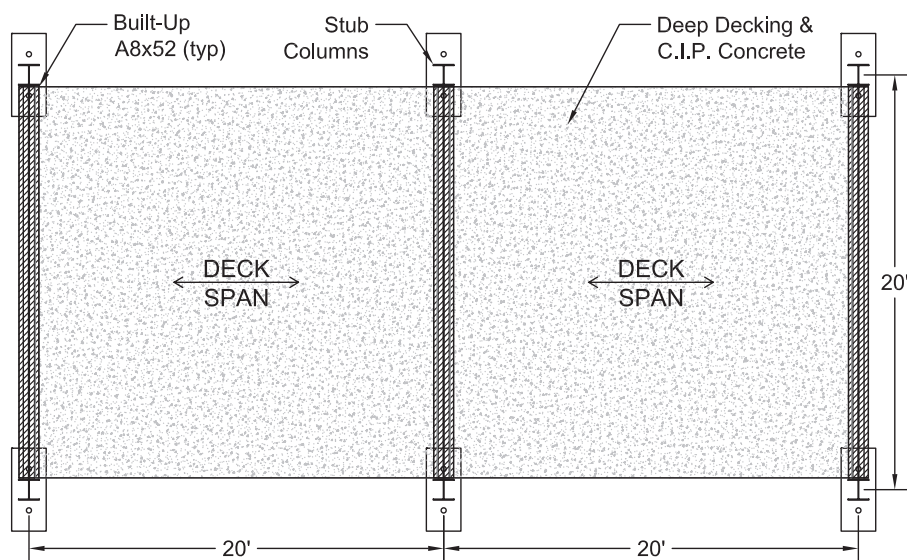
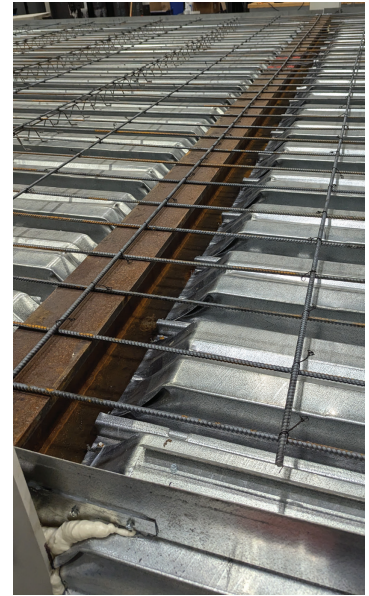


Fig. 12. System validation test plan view.



(a) AIBs supporting deep decking



(b) Close-up of center AIB

Fig. 13. System test specimen before deck casting.



Fig. 14. Full-scale system testing (final load position).

Furthermore, the system-level test reached 125% of the theoretical full composite strength. The overstrength and increased stiffness are due to more concrete than the specified effective width being engaged in the composite section. This was verified by measuring significant concrete strain on the slab surface beyond two times the specified effective width.

DESIGN AIDS

Design resources were developed to provide engineers with tools for the future design of steel-concrete composite floor systems utilizing AIBs. These resources are in the form of design tables and examples. The design tables provided are for hot-rolled A-shapes (dimensions shown in Table 1). However, the tables can be used for built-up AIBs with the same dimensions. The contribution of the fillets was not included in the calculations to produce the tables.

The first design table is for concentric noncomposite loading during construction (Table 2). The calculations for this table primarily follow AISC *Specification* Section F4. However, the stabilizing load height effect (discussed earlier) is incorporated. Note that the results are provided for allowable strength design (ASD) and load and resistance factor design (LRFD). Table 2 provides the available flexural strength for uniformly distributed loading. In addition, the available shear strength is provided. Note that the variables in the tables are consistent with the AISC *Steel Construction Manual* (2023).

The second design table is for eccentric noncomposite loading during construction (Table 3). The table provides the maximum distributed loading (ASD or LRFD) that can be applied to one bottom flange at a distance of 1.0 in. from the edge. This distance was selected because most deep decking and precast/prefabricated panel suppliers require a 2.0 in. minimum bearing length (placing the resultant force 1.0 in. from the edge). Overall, the intent of Table 3 is to allow a relatively easy check for anticipated construction loading. Table 3 follows the criteria provided earlier on a noncomposite design for eccentric loading.

The third design table is for composite loading in the completed state (Table 4). The available strength is provided for ASD and LRFD utilizing the earlier methodology. This includes the 100 psi bond stress (for unpainted steel) using the linear interpolation method for composite flexural strength (more conservative than the partial plastic stress distribution and strain compatibility methods). The effective width of the section is one-quarter of the span length. The concrete thickness above the top of the steel section is assumed to be only 1.5 in. However, the composite strength does not vary significantly with increasing the

topping thickness to 2.5 in. A welded wire fabric or single mat of steel reinforcement is recommended above the beam for crack control. This should be sized based on the grid geometry and decking type.

No mechanical connectors were assumed within the development of Table 4. If mechanical connectors (or other methods to increase the bond) are used, the flexural strength could be increased up to the full composite strength. The flexural strength could also be increased with supplemental longitudinal steel reinforcement (similar to that shown in Figure 8).

General capacity-to-demand tables were developed for each A-shape, but are not provided herein. Please refer to Yarnold et al. (2025), Appendix J. These tables include an array of beam lengths and spacings for different loading conditions.

Another valuable design resource that was developed is a comprehensive design example with all structural design checks. This can be found in Yarnold et al. (2025), Appendix I.

SUMMARY

The presented information aims to improve building floor system economy, speed, and efficiency. A design methodology was presented for shallow-depth composite floor systems utilizing AIBs (hot-rolled or built-up) with deck panels (HC or CLT) or deep metal decking. This methodology covers the limits states critical for construction in the noncomposite state and in-service as a composite system. The method presented is a lower-bound approach with no mechanical connectors for horizontal shear transfer (only bond and friction). Therefore, the calculations are for a partial composite design.

The design methodology was developed through an experimental testing program, which was summarized herein. This includes various experimental testing performed for AIBs within shallow-depth floor systems. The testing reduced the uncertainty of several design parameters and validated certain assumptions. Three main phases of testing were (1) an initial large-scale concept test, (2) nine composite beam tests, and (3) a full-scale system validation test. The critical findings of each phase were provided, which included the following recommendations.

- Shallow-depth floor systems are efficient to construct by placing panels or deep decking on the bottom flange of an AIB and casting a slab that tops the panels and encases the AIBs.
- A 100-psi steel-concrete bond stress should be used for concrete-encased (compression flange and web)

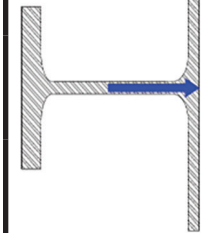
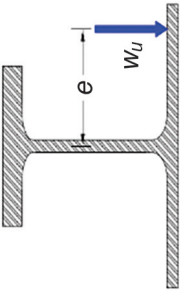
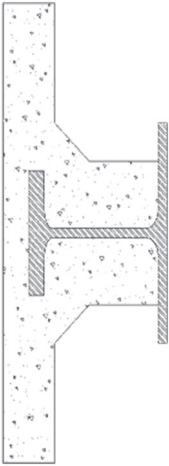


Table 2. Noncomposite Concentric Uniformly Distributed Loading Design Table
Available Flexural Strength
kip-ft
A-Shapes
 $F_y = 50$ ksi

Shape		A12x				A10x				A8x				A6x		
		108		85		88		65		72		52		43		
Design	Unbraced Length, L_b , ft	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
		20	384	577	294	442	256	385	188	282	160	240	118	177	58	87
		21	384	577	294	442	256	385	188	282	160	240	118	177	58	87
		22	384	577	294	442	256	385	186	280	160	240	116	174	58	87
		23	384	577	294	442	256	385	181	272	160	240	112	169	58	87
		24	384	577	294	442	256	385	176	265	160	240	110	165	58	87
		25	384	577	287	431	256	385	171	257	160	240	106	160	58	87
		26	384	577	279	420	256	385	166	249	156	235	104	156		
		27	384	577	272	409	256	385	161	242	154	231	100	151		
		28	384	577	265	398	256	385	156	235	152	228	98	147		
		29	384	577	257	387	256	385	152	228	149	224	95	143		
		30	384	577	250	376	255	383	147	221	146	220	92	139		
		31	384	577	244	366	250	376	142	213						
		32	384	577	236	355	246	370	136	205						
		33	384	577	229	344	242	363	132	198						
34	383	575	221	332	237	356	127	191								
35	376	565	214	321	233	350	123	185								
ASD	LRFD	Available Shear Strength, kips														
$\Omega_b = 1.67$	$\phi_b = 0.90$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	
$\Omega_v = 1.50$	$\phi_v = 1.00$	135	203	135	203	113	169	100	150	100	150	80	120	60	90	

 Table 3. Noncomposite Eccentric Uniformly Distributed Loading Design Table Maximum Torsional Load kip/ft A-Shapes $F_y = 50 \text{ ksi}$ $e = 0.5b_{fb} - 1 \text{ in.}$													
Shape		A12x				A10x				A8x			
		108		85		88		65		72		52	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
15								2.40	3.60	3.03	4.55	1.43	2.15
16								2.03	3.05	2.79	4.20	1.20	1.80
17								1.76	2.65	2.59	3.90	1.03	1.55
18								1.53	2.30	2.43	3.65	0.90	1.35
19								1.33	2.00	2.16	3.25	0.80	1.20
20				1.96	2.95	1.90	2.85	1.20	1.80	1.93	2.90	0.70	1.05
21				1.73	2.60	1.66	2.50	1.06	1.60	1.73	2.60	0.63	0.95
22				1.56	2.35	1.50	2.25	0.93	1.40	1.56	2.35	0.57	0.85
23				1.40	2.10	1.33	2.00	0.86	1.30	1.43	2.15	0.53	0.80
24				1.26	1.90	1.23	1.85	0.77	1.15	1.30	1.95	0.47	0.70
25		1.40	2.10	1.16	1.75	1.10	1.65	0.70	1.05	1.20	1.80	0.43	0.65
26		1.26	1.90	1.06	1.60	1.00	1.50	0.63	0.95	1.10	1.65	0.40	0.60
27		1.16	1.75	0.96	1.45	0.93	1.40	0.60	0.90	1.03	1.55	0.37	0.55
28		1.06	1.60	0.90	1.35	0.86	1.30	0.53	0.80	0.93	1.40	0.33	0.50
29		0.96	1.45	0.80	1.20	0.80	1.20	0.50	0.75	0.86	1.30	0.30	0.45
30		0.90	1.35	0.77	1.15	0.73	1.10	0.47	0.70	0.83	1.25	0.30	0.45
31		0.83	1.25	0.70	1.05	0.67	1.00						
32		0.77	1.15	0.67	1.00	0.63	0.95						
33		0.70	1.05	0.60	0.90	0.60	0.90						
34		0.67	1.00	0.57	0.85	0.57	0.85						
35		0.63	0.95	0.53	0.80	0.53	0.80						
ASD	LRFD												
$\Omega_b = 1.67$													
		The available strength is based on AISC Specification Section H3.3.											
		$\phi_b = 0.90$											

 Table 4. Composite Design Table Available Composite Flexural Strength kip-ft Composite A-Shapes $F_y = 50 \text{ ksi}$ $f'_c = 4 \text{ ksi}$														
Shape	A12x				A10x				A8x				A6x	
	108		85		88		65		72		52		43	
Design	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Unbraced Length, L_b , ft	20	413	621	317	477	413	203	305	172	258	128	192	67	100
	21	414	622	318	478	413	203	305	172	258	128	193	67	101
	22	415	623	319	479	414	204	306	172	259	128	193	67	101
	23	415	624	319	479	415	204	307	172	259	129	194	67	101
	24	416	625	319	480	416	204	307	173	260	129	194	68	102
	25	417	626	320	481	416	205	308	173	260	130	195	68	102
	26	417	627	321	482	417	205	308	174	261	130	195		
	27	418	628	321	483	418	206	309	174	261	130	195		
	28	418	629	322	484	419	206	310	174	262	130	196		
	29	419	630	322	484	419	206	310	174	262	130	196		
	30	420	631	323	485	420	207	311	175	263	131	197		
	31	420	632	323	486	421	208	312						
	32	421	633	324	487	421	208	312						
	33	422	634	325	488	422	208	313						
	34	422	635	325	488	423	208	313						
	35	422	635	325	489	424	209	314						
ASD	Composite strength is quantified using linear interpolation between the steel and full composite moment capacity. The degree of composite action is determined with a bond stress of 100 psi that acts above the composite elastic neutral axis and develops over a quarter of the span length.													
$\Omega_b = 1.67$	$\phi_b = 0.90$													

unpainted AIB partial composite flexural strength calculations.

- The gross moment of inertia is conservative for the in-service stiffness assumption when checking composite deflections and vibrations.
- The linear interpolation method is accurate, but conservative for partial composite flexural strength calculations. The partial plastic stress distribution and strain compatibility methods are also acceptable methods.
- The effective width per AISC *Specification* Section I3 (one-quarter the span length) is conservative for shallow-depth steel-concrete composite systems.
- Shallow-depth floor systems with AIBs are ductile and robust, withstanding substantial loading under repetitive load cycles.
- Finally, design aids are provided to equip future engineers with resources to design shallow-depth floor systems efficiently. These aids are in the form of design tables and a comprehensive design example.

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Evaluation of Seismic Design Parameters for Metal Buildings with Heavy Wall and Heavy Roof in High Seismic Zones

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ABSTRACT

The objective of this paper is to explore the seismic performance of metal building systems when they are designed as steel ordinary moment frames (OMFs) but have wall and roof dead loads that exceed prescriptive limits in the United States' loads standard (ASCE 7). A set of archetype frames—spanning both clear span and modular metal building system geometries—was developed in collaboration with industry and subjected to a FEMA P-695-based seismic assessment. At the core of the assessment is an advanced shell finite element model of the metal building systems, exercised through a nonlinear hysteretic protocol to accurately capture the response of the system. The shell finite element models inform the calibration of nonlinear single-degree-of-freedom models used in incremental dynamic analyses under the FEMA P-695 ground motion suite. The results show that heavier roof and wall loads can reduce system ductility and collapse margins in certain configurations. However, all archetypes that satisfy a set of simple frame member proportioning criteria, achieve acceptable performance under the evaluated seismic demands. These findings support the extension of OMF-based designs to heavier wall and roof loads if the additional proportioning requirements are included.

Keywords: metal buildings, seismic performance, nonlinear, incremental dynamic analysis, fragility.

INTRODUCTION

Steel ordinary moment frames (OMFs) are generally considered less ductile than special moment frames, and their use is typically restricted in high seismic design categories (SDCs) D, E, and F per ASCE 7-16 (2016), which was used for design in this study. However, historical seismic performance of single-story OMFs—particularly in metal building systems with limited height, mass, and number of stories—has shown that these systems can perform adequately in strong ground motions. Following the 1994 Northridge earthquake, observations of metal building performance contributed to the development of ASCE 7 provisions that permitted OMFs in certain single-story structures. For instance, ASCE 7-98 (1998) and ASCE 7-02 (2002) allowed OMFs up to 60 ft (18.3 m) in height with

roof dead loads below 15 psf (718 Pa), specifically acknowledging bolted end-plate moment connections common in metal buildings. These provisions have since evolved and are currently found in ASCE 7-16 and ASCE 7-22 (2022) Sections 12.2.5.6.1 and 12.2.5.6.2.

With the introduction of the FEMA P-695 methodology (ATC, 2009), the Metal Building Manufacturers Association (MBMA) initiated a series of studies to evaluate whether OMFs used in metal buildings meet seismic performance expectations under current and extended design conditions. An initial peer-reviewed effort applied FEMA P-695 to index archetypes representing conventional clear-span buildings, including some with heavier wall systems such as archetype A1 and A2 (Moen et al., 2019). This study also used full-scale shake table testing of a specimen with precast concrete panels to validate modeling protocols confirming that such systems satisfy collapse performance objectives when designed with a response modification factor $R = 3.5$ (Meimand et al., 2018; Moen et al., 2019). Although these heavy wall configurations were shown to meet FEMA P-695 collapse safety criteria, their performance was assessed within a narrow design space (clear span and light roof), without establishing general geometric or structural limits for broader code application. That work laid the foundation for a validated high-fidelity modeling framework—supported by experimental research, including shake table and component testing (Smith et al., 2013; Uang et al., 2011)—that has since been extended to modular building configurations in a follow-up study by Nikoukalam et al. (2026).

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Table 1. Metal Building Archetypes Dimensions and Design Loads

Archetype	Roof Dead Load		Wall Panel Self-Weight		H^c	$N^c \times \text{Span}$	Bay	V_r^c
		psf (Pa)		psf (Pa)	ft (m)	N $\times$ ft (m)	ft (m)	kips (kN)
M-HW-1	Light	20 (958)	Heavy ^b	116 (5554)	45 (13.7)	2 $\times$ 50 (15.2)	25 (7.6)	127.8 (568)
M-HW-2	Light	20 (958)	Heavy	116 (5554)	45 (13.7)	2 $\times$ 100 (30.4)	25 (7.6)	142.7 (635)
M-HW-3	Heavy ^a	30 (1436)	Heavy	116 (5554)	45 (13.7)	2 $\times$ 50 (15.2)	25 (7.6)	123.4 (549)
M-HW-4	Heavy	30 (1436)	Heavy	116 (5554)	45 (13.7)	2 $\times$ 100 (30.4)	25 (7.6)	171.8 (764)
C-HR-1	Heavy	30 (1436)	Light	~2 (96)	25 (7.6)	1 $\times$ 50 (15.2)	25 (7.6)	23.9 (106)
C-HR-2	Heavy	30 (1436)	Light	~2 (96)	45 (13.7)	1 $\times$ 50 (15.2)	25 (7.6)	23.6 (105)
C-HR-3	Heavy	30 (1436)	Light	~2 (96)	25 (7.6)	1 $\times$ 100 (30.4)	25 (7.6)	43.2 (192)
C-HR-4	Heavy	30 (1436)	Light	~2 (96)	45 (13.7)	1 $\times$ 100 (30.4)	25 (7.6)	43.6 (194)

^a Heavy roof dead load is defined as 30 psf (1440 Pa), which is in excess of the ASCE 7-16 limit.

^b Heavy wall self-weight is defined as 116 psf (5550 Pa) and is based on a typical precast concrete wall panel.

^c H = eave height of the archetype building; N = number of spans (modules) in the direction of the main frame; V_r = design base shear used for the equivalent lateral force procedure at the Design Earthquake level.

Building on those prior evaluations, the present study uses the framework of Moen et al. (2019) to investigate the seismic performance of both clear span and modular metal buildings with increased roof and wall dead loads (e.g., precast concrete walls) and heights beyond ASCE 7 limits, per the FEMA P-695 methodology. A suite of archetype buildings representing both clear span and modular configurations was developed in collaboration with industry partners to reflect practical design scenarios beyond the limits evaluated in previous studies and beyond those specified by ASCE 7. Modular metal buildings are moment-resisting frames having intermediate gravity-only columns that create multiple adjacent roof modules within a single-story structure. Each archetype was analyzed using modal analysis, nonlinear static and cyclic analysis, and dynamic procedures to assess key performance metrics, including system overstrength, ductility, and collapse margin. The study quantifies the impact of increased seismic mass on system behavior and establishes additional proportioning criteria that ensure acceptable performance under the FEMA P-695 framework.

DESIGN OF METAL BUILDING ARCHETYPES

A critical step in the seismic performance evaluation under FEMA P-695 is the development of representative building archetypes. In prior work, archetypes were established for clear span and modular metal buildings under typical wall and roof load conditions (Moen et al., 2019; Nikoukalam et al., 2026). In the current study, a total of eight archetypes were developed, consisting of four modular buildings evaluated with increased wall loads (designated as M-HW: modular-heavy wall) and four clear span buildings evaluated with increased roof loads (designated as C-HR: clear

span-heavy roof). Table 1 summarizes the building geometries and design loads. Detailed frame design packages and member schedules can be provided upon request.

All M-HW archetypes have a building height of 45 ft (13.7 m) [which is more than the 35 ft (10.7 m) limit in ASCE 7] and consist of two spans of either 50 ft or 100 ft (15.2 m or 30.4 m), with a longitudinal bay spacing of 25 ft (7.6 m). Two roof dead load levels are considered: 20 psf and 30 psf (958 Pa and 1440 Pa), while the wall panel self-weight is consistently taken as 116 psf (5550 Pa), representing a 9.25-in.- (235-mm-) thick precast concrete wall panel.

The C-HR archetypes have single span frames with heights of either 25 ft or 45 ft (7.6 m or 13.7 m) and spans of 50 ft or 100 ft (15.2 m or 30.4 m). A roof dead load of 30 psf (1440 Pa) is applied to all clear span buildings, while wall panel self-weight is approximately 2 psf (96 Pa), consistent with typical light cladding systems. All clear span buildings share the same 25 ft (7.6 m) longitudinal bay width. These archetypes isolate the influence of increased roof load on seismic behavior while keeping other parameters in line with previous studies.

The archetypes were designed and detailed by engineers from Metal Building Manufacturers Association (MBMA) member companies using their own in-house design, optimization, and detailing software, based on the geometric and loading parameters defined in Tables 1 and 2. All buildings were designed as Risk Category II structures subjected to Design Earthquake (DE) ground motions per ASCE 7-16. Note, the bracing provisions of AISC 360, Appendix 6 (2016b), were enforced in all designs, as these provisions are not always satisfied in conventional strength-only bracing design practice for metal buildings, as were previously found to be critical in ensuring acceptable ductility (Nikoukalam et al., 2026).

Table 2. Archetype Design Assumptions

Summary of Demand Side (ASCE 7) Design Assumptions	
Building risk category	II
Roof slope	1:12
Roof dead load [psf (Pa)]	2.2 (105.3)
Self-weight [psf (Pa)]	2.5 (119.7)
Collateral dead load [psf (Pa)]	15 (718) for light roof designs 23 (1101) for heavy roof designs
Roof live (reducible) [psf (Pa)]	20 (957.6)
Snow load [psf (Pa)] ^b	0 (0)
Rain load [psf (Pa)]	0 (0)
Wind speed [mph (kph)]	110 (177)
Wind exposure category	B
Seismic importance factor	1.0
Seismic design category	D
Site class	D
F_a, F_v	1, 1.7
S_s (g), S_1 (g)	1.5, 0.6
S_{DS}, S_{D1}	1, 0.68
R, Ω_o, C_d	3.5, 2.5, 3
Redundancy factor (ρ)	1.3
Design method	Allowable Stress Design
Seismic load	Equivalent lateral force procedure
Drift limit	$\Delta_a = H/40$
Summary of Capacity Side (AISC 360) Design Assumptions	
Metal building frame	OMF ordinary moment frame
Seismic detailing specifications	Three-plate members, pinned base, no restriction on member compactness, no restriction on member splice locations, no restrictions on taper, pinch point, belly points, no special design of the panel zone
AISC 341 (2016a): overstrength conditions	AISC Design Guide 39 (Eatherton and Murray, 2023) connection type, design knee connection with overstrength factor of $\Omega_o = 3.0$ (OMF overstrength-level seismic demands) ^a
	Use Ω_o forces only for column and base plate axial load, including anchor rods and welding.
Material	Specified $F_y = 55$ ksi (379 MPa), $F_u = 70$ ksi (482 MPa)
Welding	AWS D1.8/D1.8M (2025) provisions, no protected zones
Anchor rods	ASTM F1554 Grade 36 (2020)
^a Earlier phases of the research used $R = 1.0$ as shorthand for elastic-level connection demand; this phase uses $\Omega_o = 3.0$ consistent with current overstrength-based OMF connection design intent.	
^b To ensure that frame designs were governed by seismic loads, snow load was not considered.	

To ensure that frame designs were governed by seismic loads, snow load combinations were not considered. Secondary structural elements such as purlins, girts, roof decks, diaphragms, and lateral braces were designed for wind loads representative of the average component and cladding pressures in the United States, corresponding to a wind speed of 110 mph (177 kph). However, seismic loads governed the design of the primary frames. Seismic parameters such as spectral response accelerations and site class follow FEMA P-695 recommendations for a typical high seismic region; see Tables 1 and 2 for the full design narrative details.

NUMERICAL MODELING

The numerical modeling framework used in this study follows the high-fidelity finite element approach previously developed for metal building seismic performance evaluation (Meimand et al., 2018; Moen et al., 2019) and extended in later modular frame studies by Nikoukalam et al. (2026). The modeling approach was validated against subassembly tests conducted per AISC 341-10 (AISC, 2010; Smith et al., 2013) and full-scale shake table tests at University of California–San Diego (UCSD) (Smith et al., 2013; Uang et al., 2011).

All finite element simulations were conducted in ABAQUS (Simulia, 2014), with automated input file generation via MATLAB scripts (MathWorks, 2018). Primary frame members—including built-up, tapered columns and rafters—were modeled using S4R shell elements (Figure 1), following mesh density guidelines from Schafer et al. (2010), with a minimum of four nodes per local buckling

half-wavelength. This discretization has been shown to provide mesh-converged prediction of local buckling mode shapes and associated post-buckling strength and stiffness degradation, which are critical to accurately capturing the global nonlinear force-deformation response of metal building frames. Therefore, the model is capable of capturing local and global buckling of the rafters and columns; local, distortional, and global buckling (including stiffness loss) of the purlins and girts; as well as buckling and yielding of the panel zone. The model does not include explicit simulation of fracture in flange braces, their connections, or other components. Local and global imperfections were included: a web distortion pattern with $\delta = h/250$ and a global sweep of $L/1000$ (Smith, 2013). Residual stresses were modeled using thermal self-equilibrating distributions based on Prawel et al. (1974) and Kim (2010).

An isotropic-kinematic plasticity model where the von Mises yield surface both expands and shifts to include residual stresses that remain after elastic unloading (i.e., the Bauschinger effect) is considered in both static and cyclic models. All secondary structural elements, including purlins, girts, and lateral braces, are of a yield stress of 50 ksi (345 MPa) and an engineering strain hardening slope of 0.25%E with isotropic hardening, where E is the elastic modulus of the steel material. Secondary elements with open cold-formed steel sections such as purlins, girts, and roof and wall panels were modeled using shell elements, rod braces were modeled using beam elements, and fasteners represented using multipoint constraints.

In modular buildings (M-HW archetypes), the wall system consisted of concrete panels rather than steel wall panels. These panels were modeled using shell elements

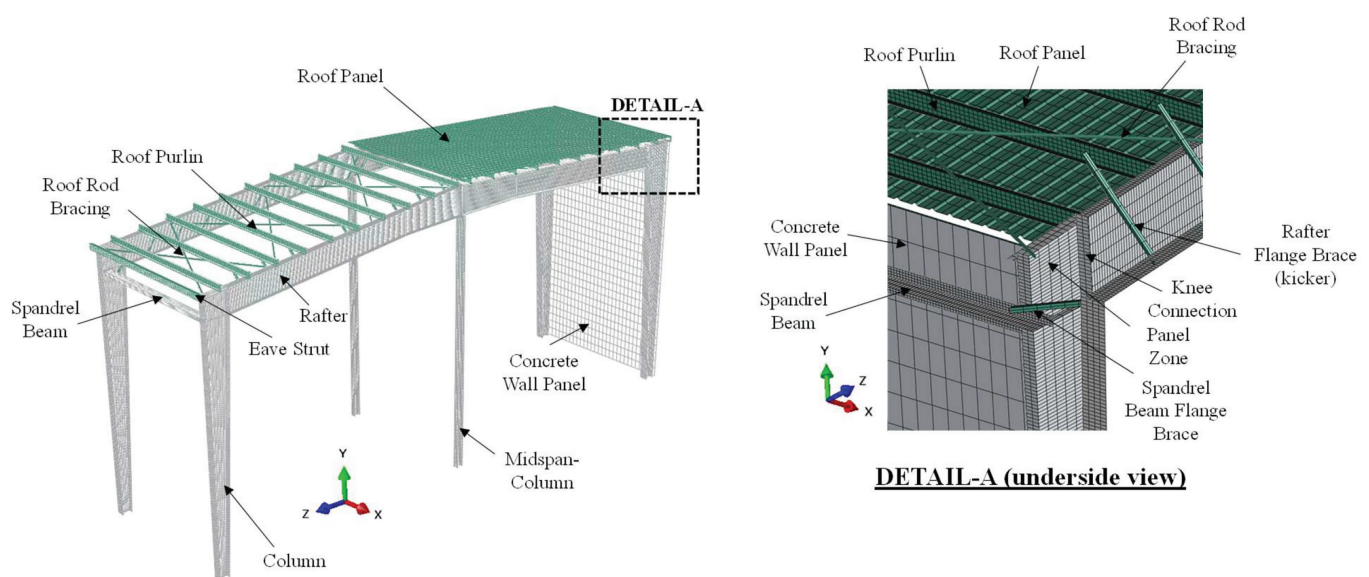


Fig. 1. Annotated example metal building system high-fidelity finite element model.

and connected to the structural frame via multipoint constraints. Additionally, spandrel beams were used in place of girts to support the wall panels. These beams were modeled using shell elements, consistent with the modeling of primary frame components. In contrast, C-HR buildings used conventional steel wall panels, as in the previous study, and girts were included as part of the secondary framing system. Further modeling assumptions, boundary conditions, and connection modeling details are provided in Nikoukalam et al. (2026).

ANALYSIS AND EVALUATION OF ARCHETYPE BUILDINGS

Pushover Analysis

Prior to conducting nonlinear cyclic or dynamic analyses, nonlinear static pushover analyses were performed for all archetypes. These analyses were used to evaluate the system overstrength, Ω_o , ductility, μ , and period-based ductility, μ_T , parameters in accordance with FEMA P-695. The pushover results are used to confirm that the numerical model produces reasonable global stiffness, strength, and expected inelastic response characteristics prior to cyclic and dynamic analyses, identify the governing failure mechanisms, and assist in defining drift-based collapse limits (discussed later). The high-fidelity models are fully nonlinear with respect to both material and geometry, including large deformation effects ($P-\Delta$ and $P-\delta$).

Figure 2 presents the base shear–roof displacement response for all modular buildings (M-HW archetypes), and Figure 4 shows their corresponding deformed shapes and stress contours. All M-HW buildings demonstrated adequate ductility before exceeding 6% story drift. The initial limit state in each case involved a combination of rafter yielding, local buckling, and lateral-torsional buckling (LTB), without exceeding 20% strength loss.

Figure 3 provides the static pushover response for the C-HR buildings. the pushover responses for the clear span (C-HR) archetypes. The short buildings, C-HR-1 and C-HR-3, showed stable post-peak behavior and adequate ductility before 20% strength degradation. Their first limit states involved rafter yielding, local buckling, and LTB (Figure 5).

In contrast, the tall clear span archetypes, C-HR-2 and C-HR-4, experienced more than 20% strength degradation at peak base shear, indicating insufficient ductility. The observed post-peak response for C-HR-2 and C-HR-4 are provided in Figures 6(a) and (c), respectively. Although both had peak strengths well above their design base shear, their post-peak response was dominated by rafter lateral-distortional buckling (LDB)—a mode characterized by top flange translation coupled with web bending, as

shown in Figure 7(b). This failure mechanism is not explicitly addressed in current AISC design specifications but becomes significant for certain cross-section geometries and unbraced lengths.

Geometric data analysis of the modeled frames (Figure 8), focusing on the dimensions at the first brace away from the knee in the rafter, revealed common features among the underperforming clear span frames:

- h/b_f ratios greater than 6
- A_w/A_f ratios greater than 2.5

where h is the clear distance of the web, b_f is the total flange width, A_w is the area of the web, and A_f is the area of the flange. That is, in the poor performing frames, the rafter is overly tall and narrow with the web carrying a large amount of the demand. These proportioning characteristics are the same required to develop stable shear post-buckling through tension field action (TFA) as specified in AISC 360-16, Section G2.2 (2016b). Based on this observation, C-HR-2 and C-HR-4 were redesigned by MBMA engineers to meet the AISC 360 TFA proportioning limits. The updated archetypes, labeled C-HR-2R and C-HR-4R, were reassessed using the same pushover protocol. Their responses are shown in Figure 3 and their deformed shapes in Figures 6(b) and (d), respectively. The redesigns led to improved ductility and shifted the governing limit state from rafter LDB to panel zone yielding and local buckling.

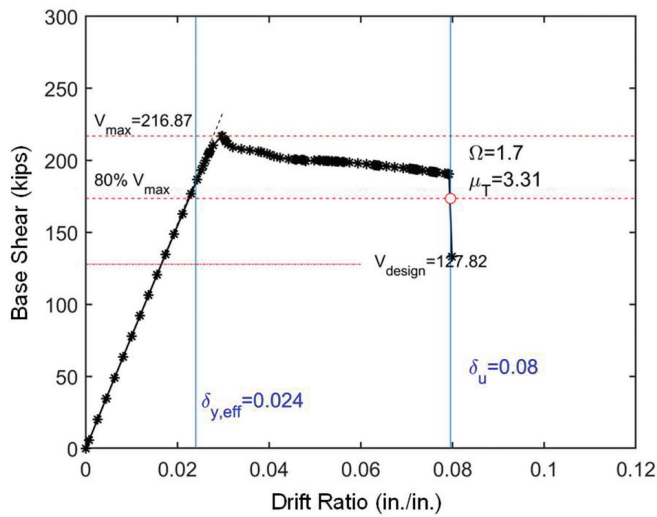
The nonlinear static pushover analysis provides a clear assessment of the governing limit states in each archetype. The sequence of failure mechanisms observed across the full drift range is summarized in Table 3.

Major observations from the nonlinear static response include these:

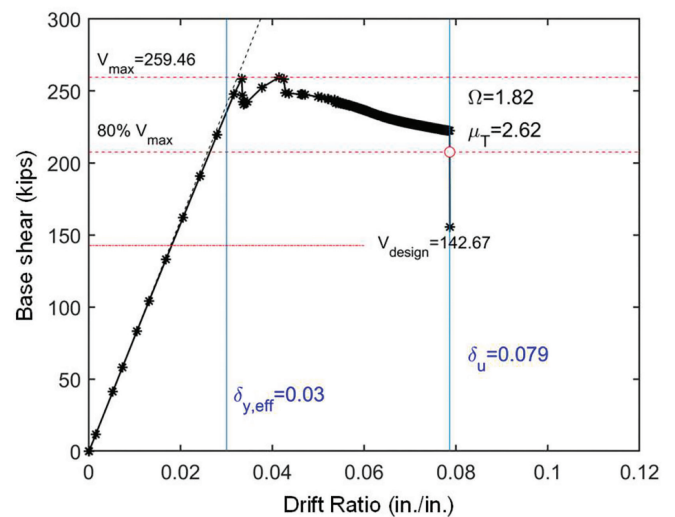
- In the modular heavy wall (M-HW) buildings, the first significant behavior consistently involved rafter yielding and local buckling near the haunch region. For M-HW-1 and M-HW-2, no significant strength degradation occurred beyond initial limit states, and panel zone yielding appeared only at higher drift levels. For M-HW-3 and M-HW-4, lateral-torsional buckling (LTB) in the rafter and purlin LTB appeared in the post-peak and large-drift stages, but the overall behavior remained stable.
- In the clear span heavy roof (C-HR) archetypes, short buildings (C-HR-1 and C-HR-3) exhibited limit states involving rafter yielding, local buckling, and LTB. At higher drift levels, C-HR-1 showed minor column local buckling, and C-HR-3 developed purlin LTB, but neither affected overall system performance. These frames demonstrated sufficient strength and ductility for seismic design compliance.

- The original tall clear span buildings, C-HR-2 and C-HR-4, exhibited lateral-distortional buckling (LDB) in the rafters at low drift levels, followed by purlin LTB. These failure modes resulted in a brittle post-peak response and inadequate ductility. Their behavior reinforced the importance of section proportioning for tall, clear span frames and led to the identification of two key geometric limits related to h/b_f and A_w/A_f ratios (see earlier discussion).

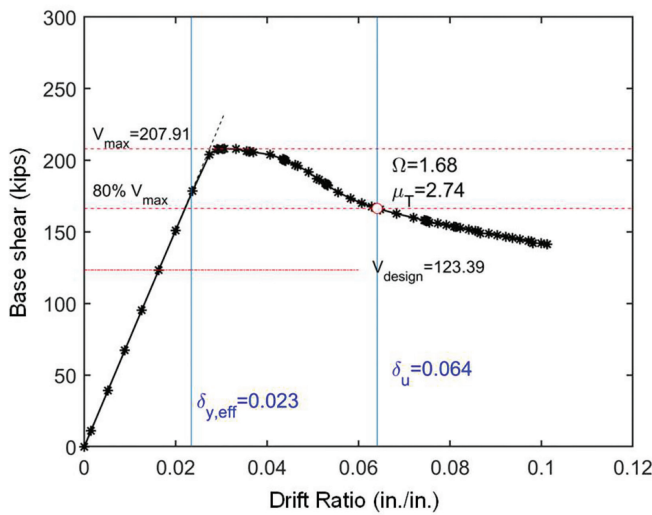
- The redesigned tall clear span archetypes, C-HR-2R and C-HR-4R, implemented the AISC 360, Section G2.2-based proportioning limits. As a result, the failure modes transitioned from rafter LDB to more ductile mechanisms such as panel zone yielding, local buckling, and, in some cases, minor rafter LTB. These redesigned configurations achieved stable post-peak behavior, validating the proposed proportioning criteria.



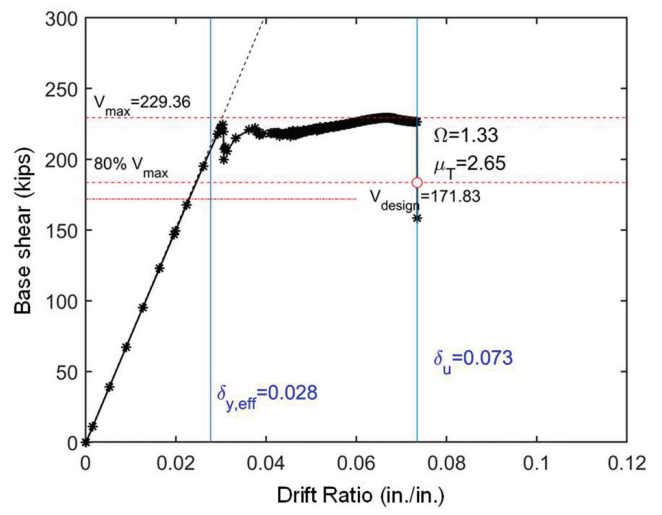
(a) M-HW-1



(b) M-HW-2

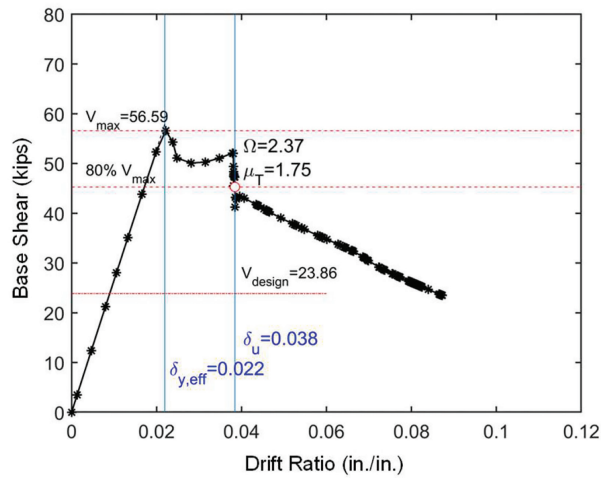


(c) M-HW-3

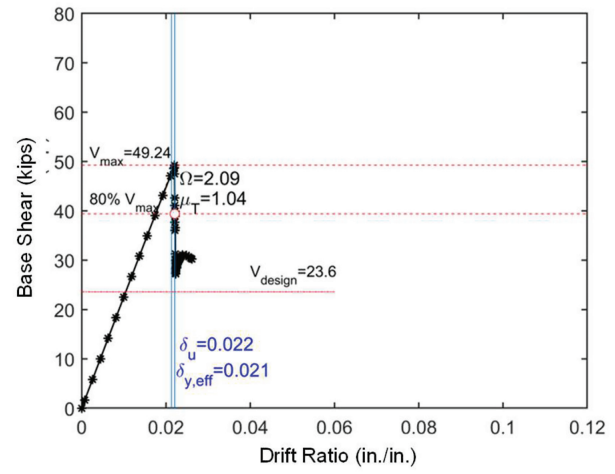


(d) M-HW-4

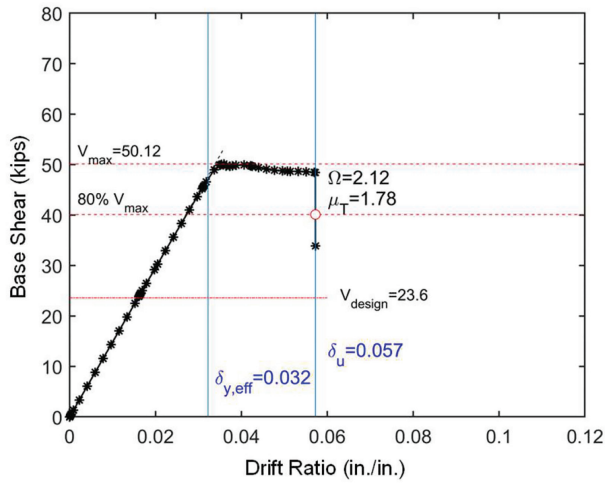
Fig. 2. Static pushover curve of modular heavy wall archetypes (1 kip = 4.448 kN).



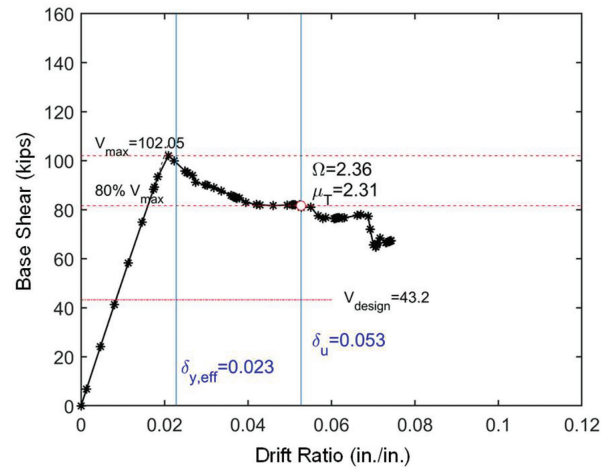
(a) C-HR-1



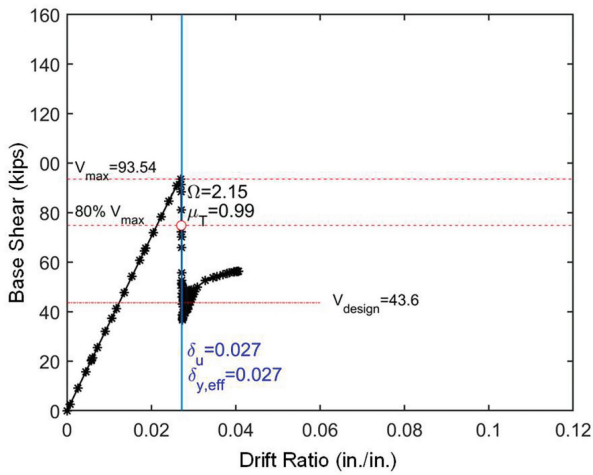
(b) C-HR-2



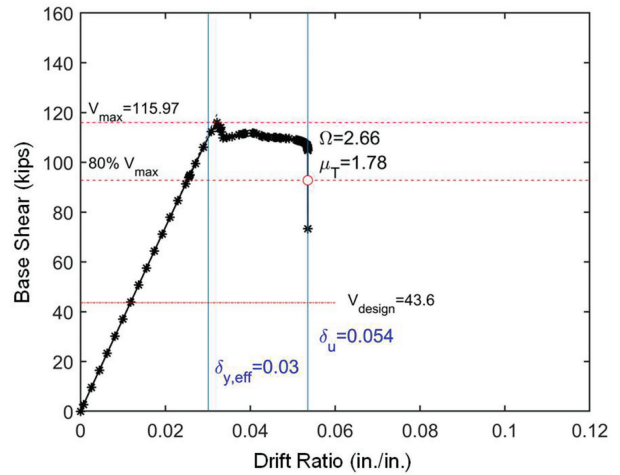
(c) C-HR-2R



(d) C-HR-3



(e) C-HR-4



(f) C-HR-4R

Fig. 3. Static pushover curve of clear-span heavy roof archetypes (1 kip = 4.448 kN).

Table 3. Sequence of Observed Behavior in Each Archetype						
Archetype	Observed Behaviour					
	Drift (%)	First Significant Behavior	Drift (%)	Post-Peak Behavior	Drift (%)	Large-Drift Behavior
M-HW-1	3.8	Rafter Y and LB	5.0	Rafter Y and LB PZ Y	—	—
M-HW-2	3.4	Rafter Y and LB	4.3	Rafter Y, LB, and LTB PZ Y	—	—
M-HW-3	3.7	Rafter Y, LB, and LTB	4.4	Rafter Y, LB, and LTB Purlin LTB	5.3	Rafter Y, LB, and LTB Purlin LTB, PZ Y
M-HW-4	3.8	Rafter Y, LB, and LTB	4.3	Rafter Y, LB, and LTB Purlin LTB	5.7	Rafter Y, LB, and LTB Purlin LTB, PZ Y
C-HR-1	2.8	Rafter Y, LB, and LTB	3.9	Rafter Y, LB, and LTB Column LB		
C-HR-2R	3.6	PZ Y	4.2	PZ Y and LB Column Y and LB		
C-HR-3	2.4	Rafter Y, LB, and LTB	5.4	Rafter Y, LB, and LTB Purlin LTB		
C-HR-4R	3.3	PZ Y	4.3	PZ Y and LB Rafter Y, LB, and LTB		
Designs Prior to Redesign per AISC 360, Section G2.2						
C-HR-2	2.2	Rafter LDB and LTB Purlin LTB				
C-HR-4	2.7	Rafter LDB and LTB				
Note: Y: yielding, LB: local buckling, LTB: lateral-torsional buckling, LDB: lateral distortional buckling, PZ: panel zone						

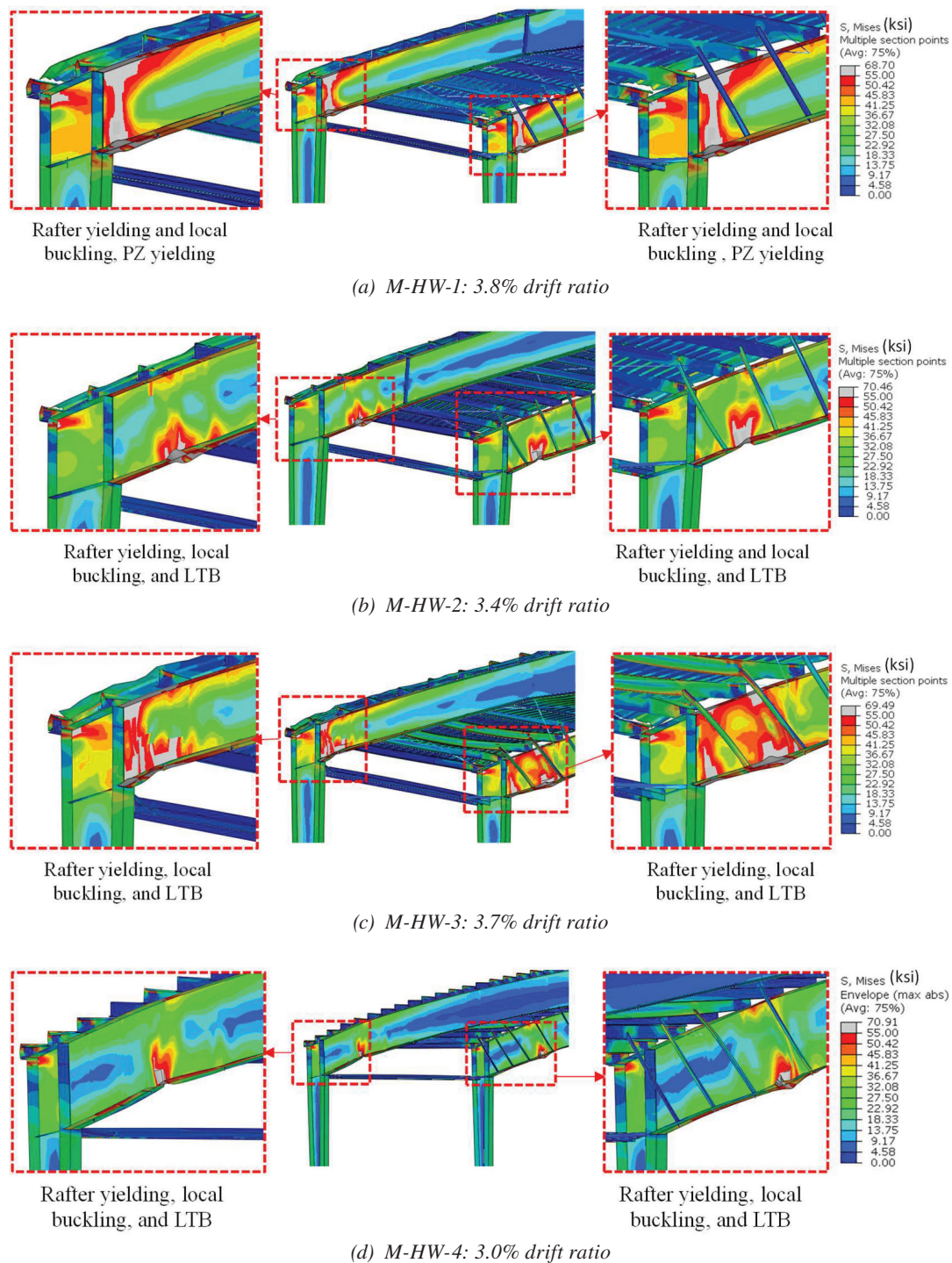


Fig. 4. First failure mode in modular heavy wall (1 ksi = 6.89 MPa).

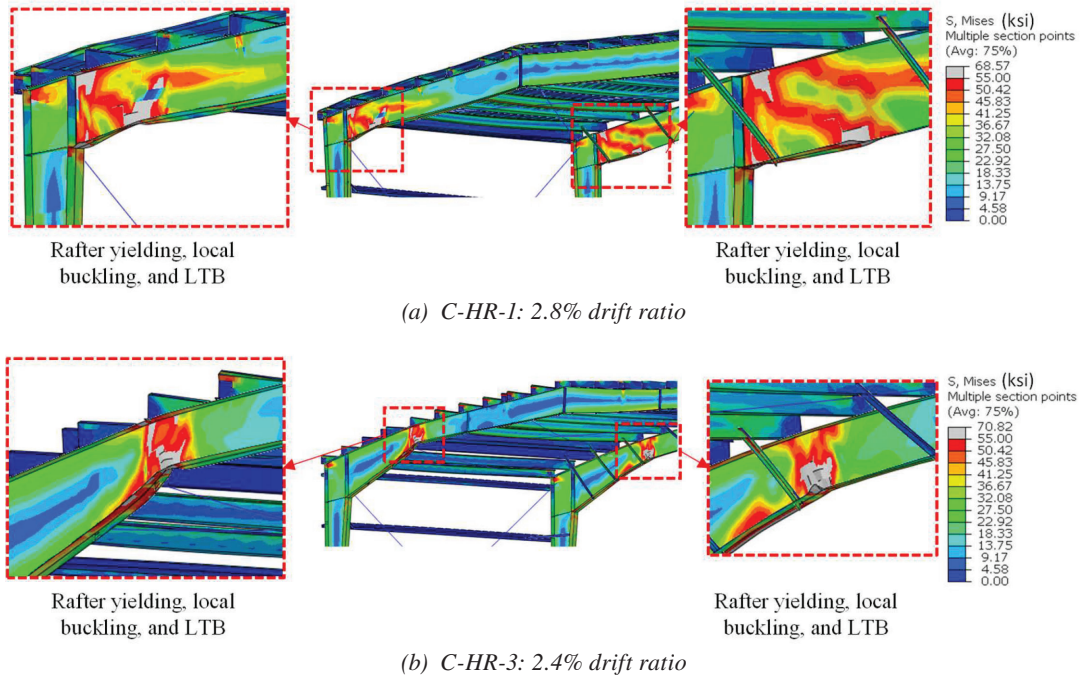


Fig. 5. First failure mode in short clear-span heavy roof buildings [$h = 25$ ft (7.6 m)] (1 ksi = 6.89 MPa).

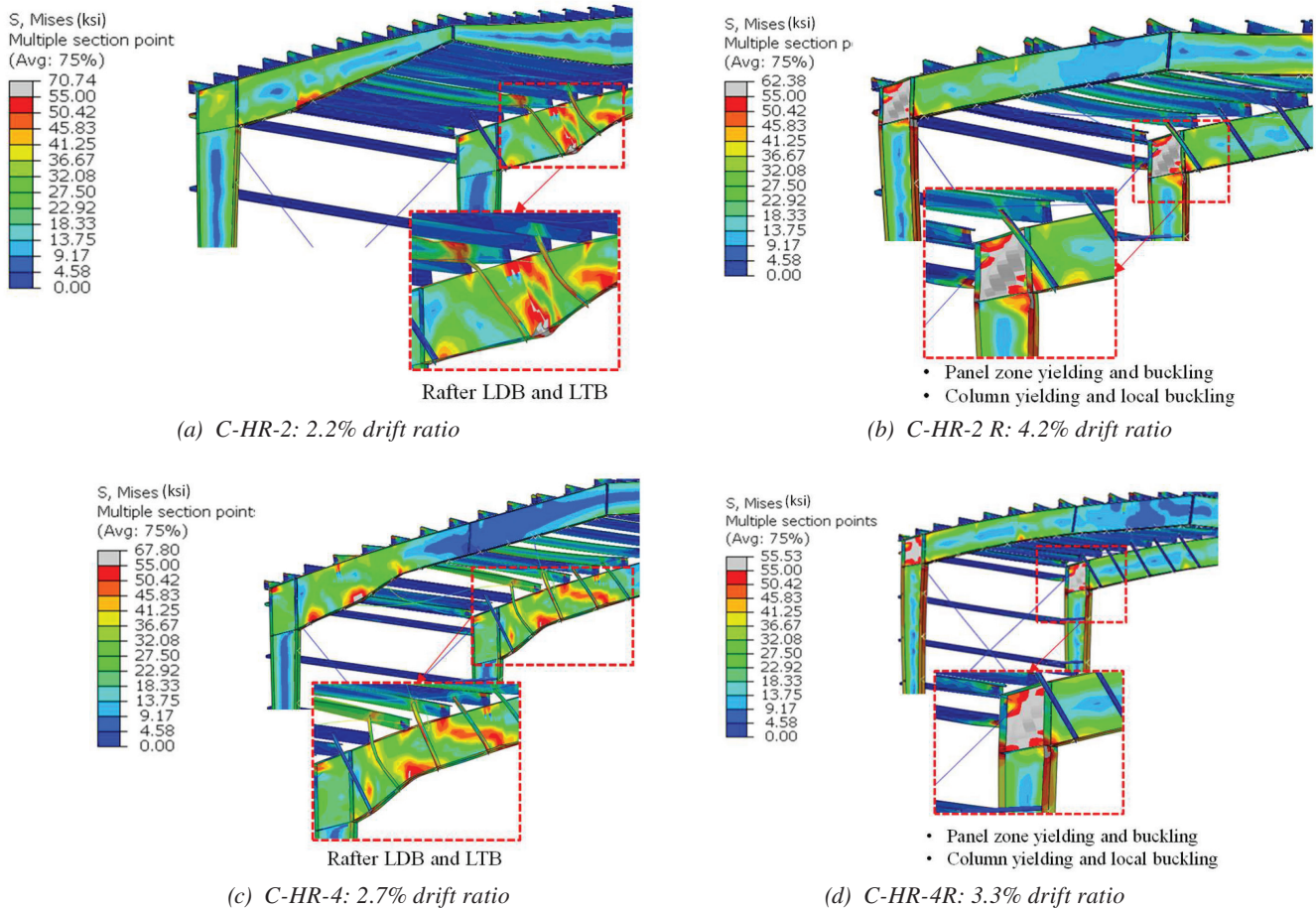


Fig. 6. First failure mode in tall clear-span heavy roof buildings [$h = 45$ ft (13.7 m)] (1 ksi = 6.89 MPa).



Fig. 7. Lateral torsional buckling (LTB) vs lateral distortional buckling (LDB) of I-shape sections.

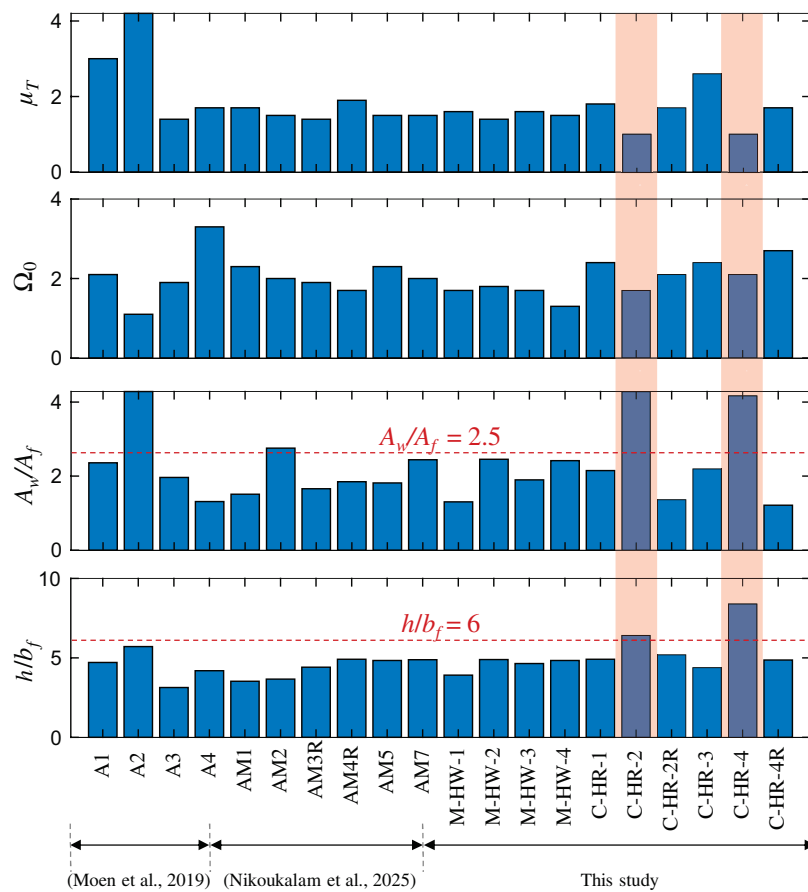


Fig. 8. Geometric data analysis of the modeled archetype buildings studied during 2018–2025, focusing on the dimensions at the first brace away from the knee in the rafter.

Table 4. Summary of System Overstrength (Ω_o) and Period-Based Ductility (M_I) Factors for Each Archetype				
		Overstrength	Ductility	Period-Based Ductility
	Archetype	Ω_o	μ^b	μ_T^b
Modular Heavy Wall	M-HW-1	1.7	>1.61	>1.87
	M-HW-2	1.82	>1.39	>1.50
	M-HW-3	1.68	1.61	1.88
	M-HW-4	1.31	>1.52	>1.66
	Average	1.63	>1.53	>1.73
Clear-Span Heavy Roof	C-HR-1	2.4	1.8	1.75
	C-HR-2R	2.1	>1.70	>1.78
	C-HR-3	2.4	2.60	2.31
	C-HR-4R	2.7	>1.70	>1.78
	Average	2.4	>1.95	>1.91
	Designs prior to redesign per AISC 360, Section G2.2			
	C-HR-2	1.7	1.00 ^a	1.00 ^a
	C-HR-4	2.1	1.00 ^a	1.00 ^a
^a premature failure ^b at 4.5% drift ratio				

The resulting system overstrength, Ω_o , ductility, μ , and period-based ductility, μ_T , for each archetype are summarized in Table 4. These parameters were computed from the nonlinear static pushover responses in accordance with FEMA P-695. The definitions and calculation methodology follow the procedures outlined in FEMA P-695 and previously documented by Nikoukalam et al. (2026). Effective yield displacement for μ_T calculations incorporates the larger of the ASCE 7-16 design period and the fundamental period obtained from eigenvalue analysis (discussed later).

According to Table 4, the calculated overstrength factors are in the range of 1.3 to 2.7. According to the FEMA P-695 procedures, the system overstrength, Ω_o , is calculated as the maximum of the average from the different performance groups, but not to exceed a maximum value of $\Omega_o = 3.0$. Accordingly, from the archetype analysis summarized in Table 4, the design value would be set to $\Omega_o = 2.0$, governed by the average value of the performance group.

Frequency (Modal) Analysis

The natural periods and mass participation ratios for all archetypes are summarized in Table 5. The results are based on eigenvalue analysis of the high-fidelity shell models, with first-mode mass participation ranging from 0.87 to 0.99. As expected, modular buildings (M-HW archetypes) exhibit longer periods than clear span buildings due to their increased flexibility and gravity-only interior columns.

The modular heavy wall (M-HW) archetypes have fundamental periods between 1.55 and 2.00 seconds. These relatively long periods reflect the increased height [45 ft (13.7 m)], heavier wall systems, and longer spans supported by gravity-only interior columns. In contrast, the clear span heavy roof (C-HR) buildings exhibit shorter periods, ranging from 0.98 to 1.72 s, depending on height and span. As expected, taller clear span frames show longer periods than their shorter counterparts.

Across all archetypes, the empirical period estimates from ASCE 7-16, Equation 12.8-7, significantly underpredict the actual periods observed in the high-fidelity models, with ratios of $T_1/T_{analytical}$ ranging from approximately 2.4 to 3.2. In contrast, the periods used in industry design—based on beam element approximations—exhibit closer agreement with the shell model results, generally falling within 10 to 15%.

Additional details regarding the modal analysis procedures and underlying modeling assumptions can be found in Nikoukalam et al. (2026).

Cyclic Analysis

Quasi-static cyclic simulations were conducted on the high-fidelity models of each archetype to evaluate strength and stiffness degradation under repeated lateral loading. The simulations follow the cyclic displacement protocol specified in AISC 341, applied horizontally at the

Table 5. Summary of the Modal Analysis Results of the High-Fidelity Models and Period Evaluation of the Archetype Buildings per ASCE 7-16 Method

Archetype	Mass Participation Ratio	T_1 (s)	T_a (s)	$T_{analytical}$ (s)	T_1/T_a	$T_1/T_{analytical}$
M-HW-1	0.91	1.55	0.61	NA	2.54	NA
M-HW-2	0.88	1.79	0.63	1.80	2.84	0.99
M-HW-3	0.91	1.64	0.61	1.69	2.69	0.97
M-HW-4	0.87	2.00	0.63	1.81	3.17	1.11
C-HR-1	0.99	0.99	0.38	NA	2.61	NA
C-HR-2	0.97	1.42	0.60	1.46	2.37	0.97
C-HR-2R	0.97	1.72	0.60	1.96	2.87	0.88
C-HR-3	0.88	0.98	0.39	0.91	2.51	1.07
C-HR-4	0.97	1.62	0.61	1.49	2.66	1.09
C-HR-4R	0.97	1.54	0.61	1.45	2.52	1.06

T_1 : first period of high-fidelity shell finite element model

T_a : estimate period based on ASCE 7-16, Eq. 12.8-7; the period used for the design base shear was taken as the lesser of $T_{analytical}$ and the code-limited period $C_u T_a (C_u = 1.4)$.

$T_{analytical}$: period estimate by metal building designer based on beam element approximation

rafter-column knee level. The resulting hysteretic behavior is expressed in terms of base shear versus story drift ratio and is presented in Figure 9 for the modular heavy wall (M-HW) buildings and Figure 10 for the clear span heavy roof (C-HR) buildings.

The cyclic responses generally trace the same envelope as the monotonic backbone curves with some additional degradation in strength and stiffness across cycles. This degradation is modest in most cases, and the post-peak behavior remains stable in all archetypes.

The cyclic response results are used to calibrate nonlinear single-degree-of-freedom (SDOF) models for each archetype. These calibrated SDOF representations form the basis for the incremental dynamic analyses (IDA) discussed in the following section.

Incremental Dynamic Analysis (IDA)

Incremental dynamic analyses (IDA) were conducted for each archetype using the FEMA P-695 methodology to assess collapse performance and determine the acceptability of the seismic response modification factors used in design. Each archetype was represented by a calibrated nonlinear single-degree-of-freedom (SDOF) model (Figure 11), which was subjected to the far-field ground motion set recommended in FEMA P-695 Appendix A, comprising 22 pairs (44 total) of horizontal records.

As in previous work (Nikoukalam et al., 2026), the SDOF system was calibrated to replicate both the cyclic backbone and dynamic properties of the corresponding high-fidelity finite element model. The simplified model is shown

schematically in Figure 11, where the structural system is represented by a lumped mass, a nonlinear spring, and linear viscous damping. The hysteretic behavior of the spring is modeled using the Pinching 04 material in OpenSees (McKenna et al., 2004), which captures strength and stiffness degradation as well as pinching under reversed cyclic loading. Four key response points define the monotonic envelope of the hysteretic model, and additional parameters control unloading behavior and degradation (see Figure 12). These parameters were tuned to minimize the cycle-by-cycle error between the SDOF and high-fidelity cyclic responses and are provided in the Appendix, Table A [see Figure 13 for quasi-static cyclic response of C-HR-1—high fidelity simulation and fitted response using Pinching 04 hysteretic model in OpenSees (McKenna et al., 2004)].

The equivalent seismic mass was taken as the design seismic weight of the building, and the initial stiffness of the SDOF model was computed to match the fundamental period obtained from modal analysis. This approach follows the procedure detailed in FEMA P-695 Chapter 6. A constant 2% viscous damping ratio was applied in the SDOF analyses to represent low-amplitude elastic energy dissipation, while all inelastic mechanisms are accounted for within the hysteretic model. While the nonlinear dynamic analyses are performed using calibrated single-degree-of-freedom models in accordance with FEMA P695, it is acknowledged that this idealization neglects higher modes of vibration and their nonlinear effects; however, these effects are expected to be limited for the single-story archetypes considered herein, which exhibit dominant first-mode response.

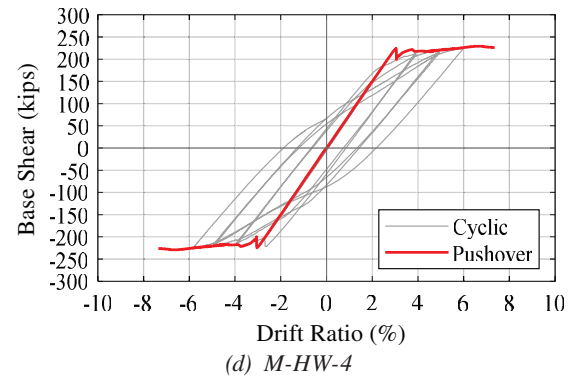
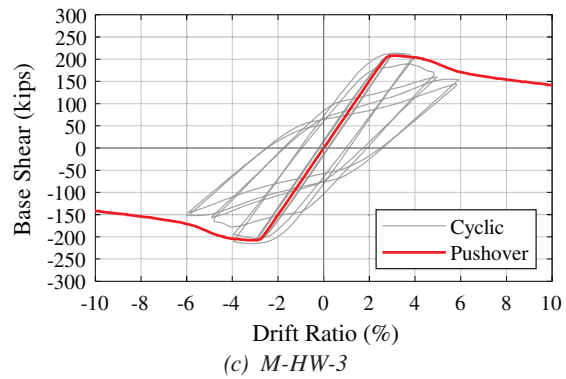
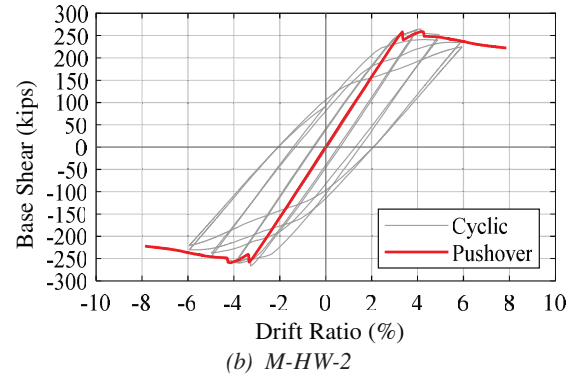
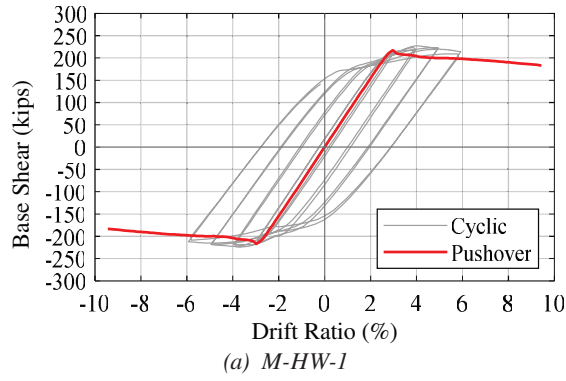


Fig. 9. Quasi-cyclic and pushover response of modular heavy wall buildings (1 kip = 4.448 kN).

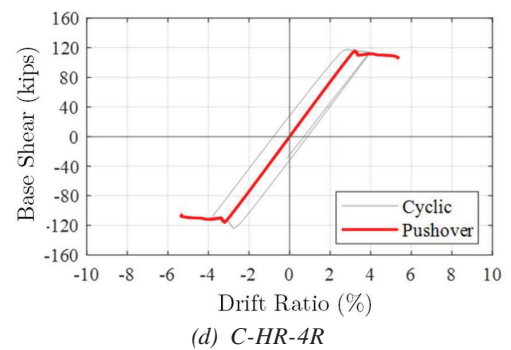
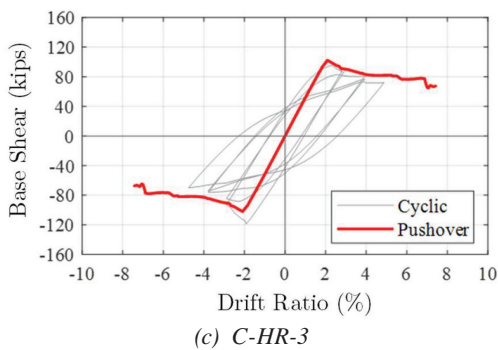
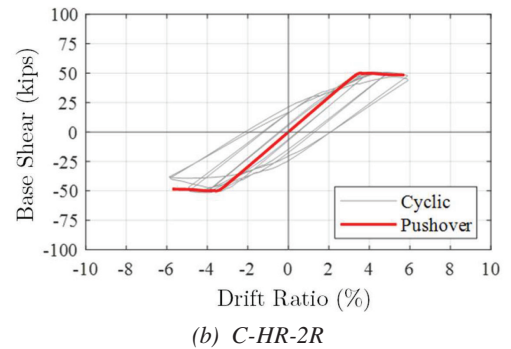
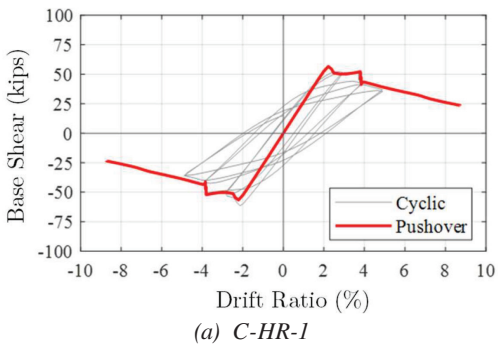


Fig. 10. Quasi-cyclic and pushover response of clear-span heavy roof buildings (1 kip = 4.448 kN).

In this study, a collapse drift limit of 4.5% was adopted for all archetypes. This limit is consistent with values used in earlier studies on metal buildings (Moen et al., 2019) and reflects the drift range at which significant local damage—such as panel zone yielding, flange local buckling, or cyclic strain accumulation—can compromise the load-carrying capacity of the system. Unlike the more flexible modular archetypes from the previous phase (Nikoukalam et al., 2026), several buildings studied here begin exhibiting non-linear response prior to 4.5% drift, making this limit both conservative and appropriate for defining collapse in the

IDA framework. Further details on the modeling approach and calibration process can be found in Nikoukalam et al. (2026).

The IDA results for the modular heavy wall (M-HW) and clear-span heavy roof (C-HR) archetypes are presented in Figures 14 and 15, respectively. Each figure shows the base shear versus drift IDA curves and the corresponding collapse fragility curves. For each archetype, the median collapse intensity, $\hat{S}_{CT}$, is determined from the IDA results, and the collapse margin ratio (CMR) is computed as $\hat{S}_{CT}/S_{MT}$,

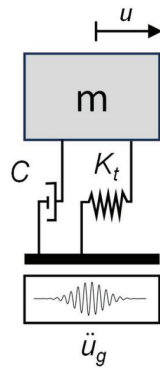


Fig. 11. Metal building nonlinear SDOF model definitions (Nikoukalam et al., 2026).

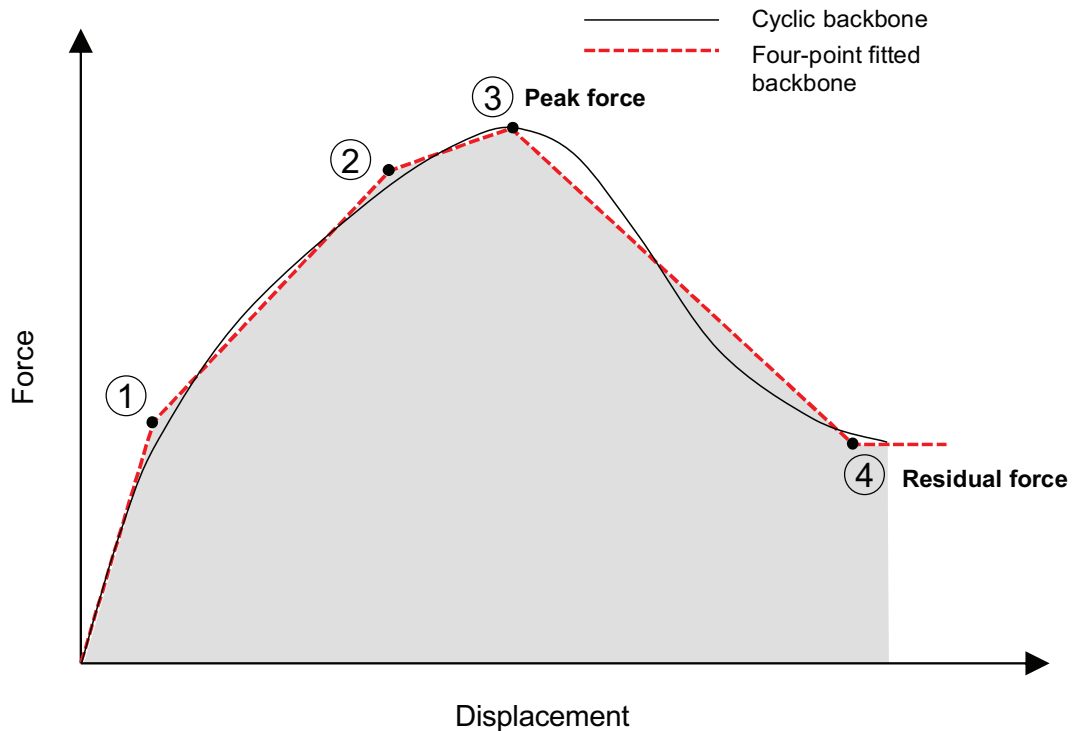


Fig. 12. A four-point backbone fitted to the cyclic backbone is shown in the first quadrant (Nikoukalam et al., 2026).

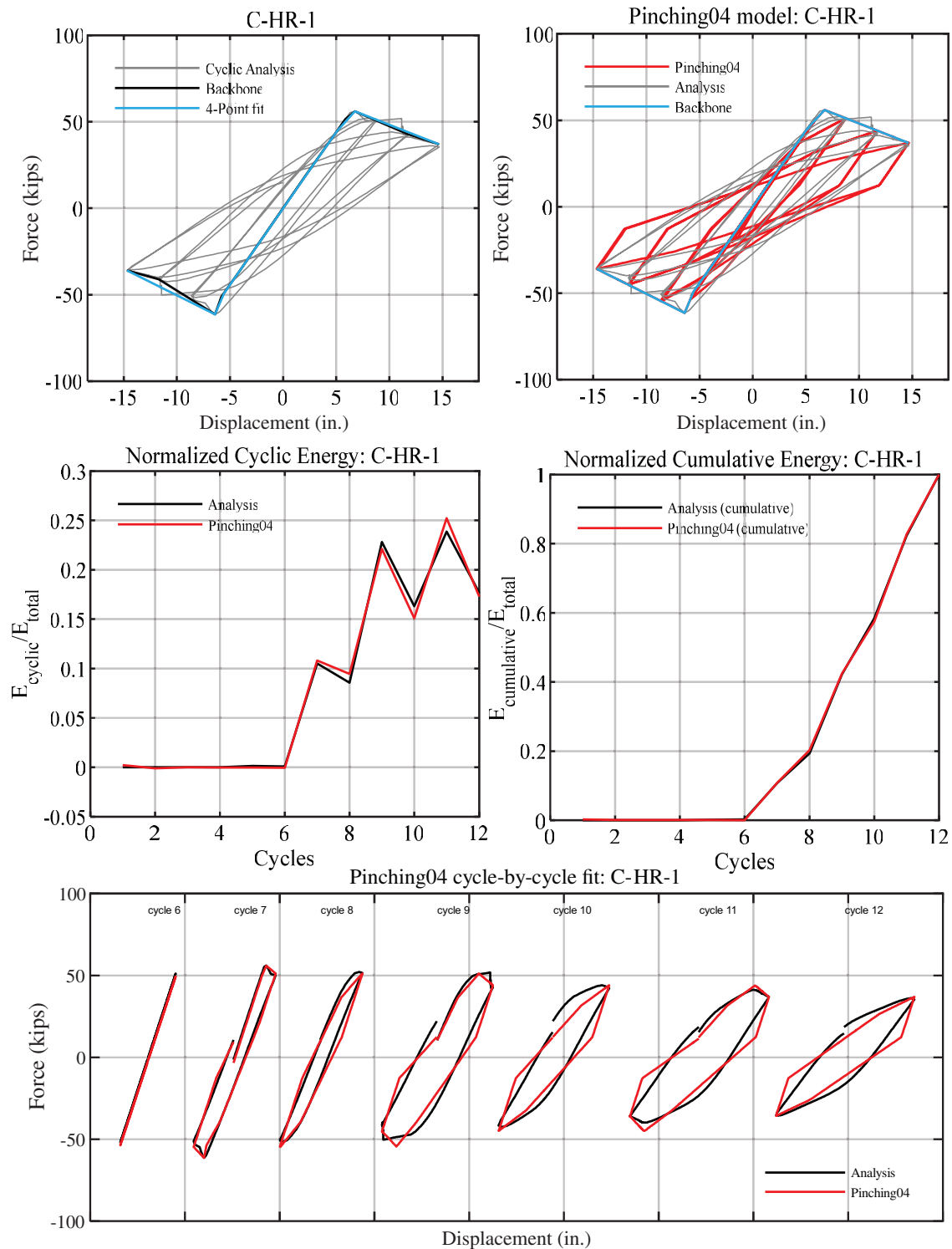
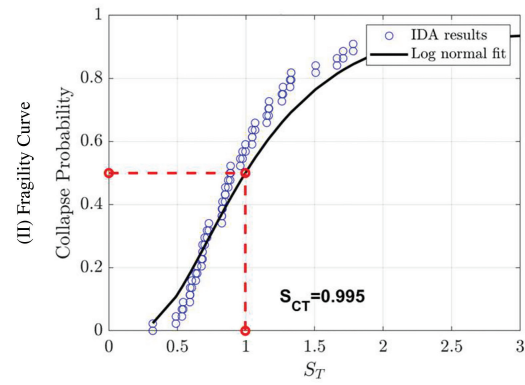
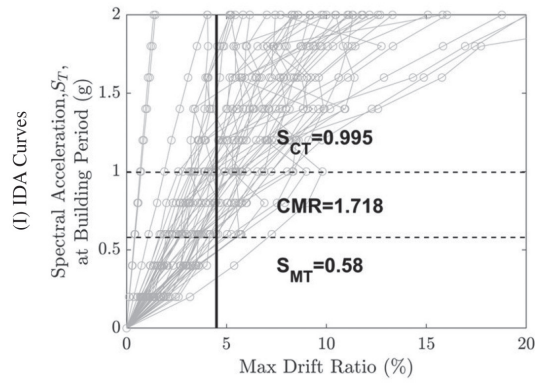
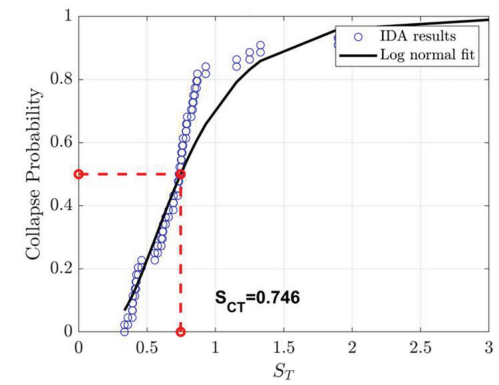
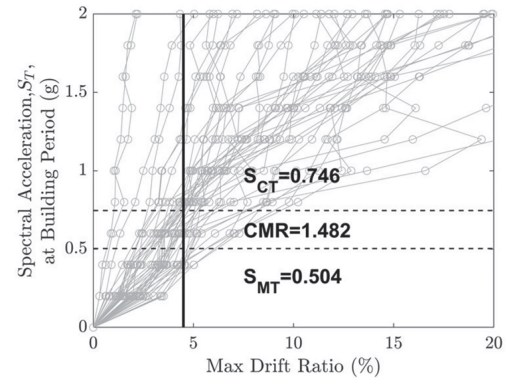


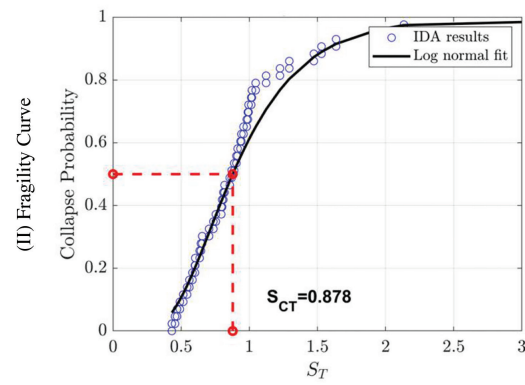
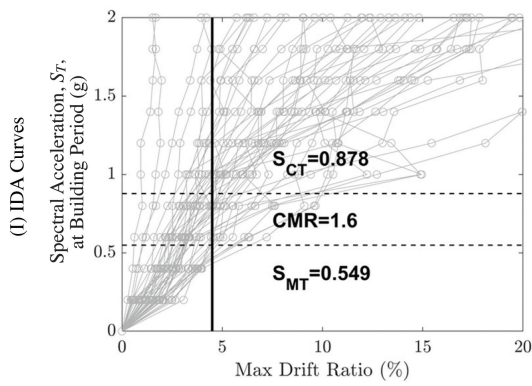
Fig. 13. Quasi-static cyclic response of C-HR-1—high fidelity simulation and fitted response using Pinching 04 hysteretic model in OpenSees (McKenna et al., 2004) (1 kip = 4.448 kN; 1 in. = 25.4 mm).



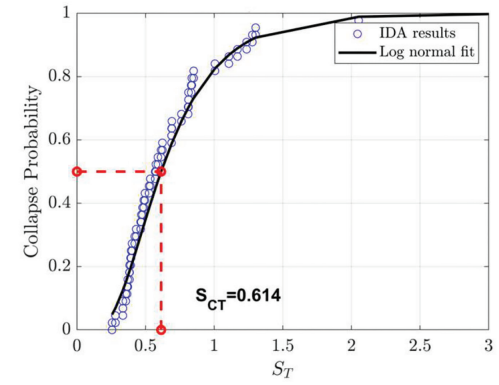
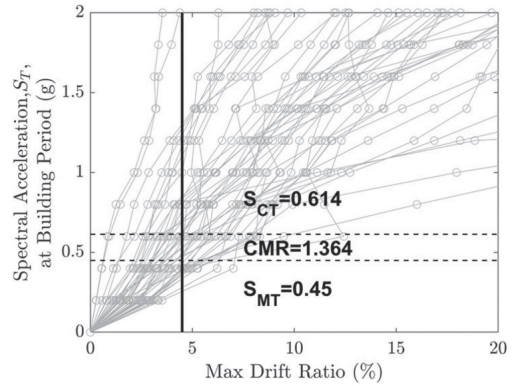
(a) M-HW-1



(b) M-HW-2

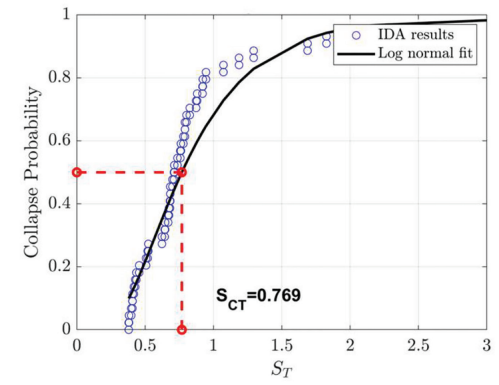
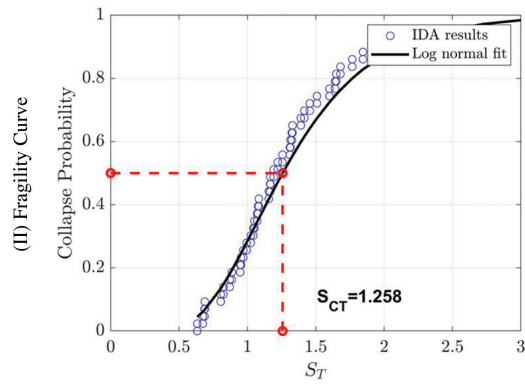
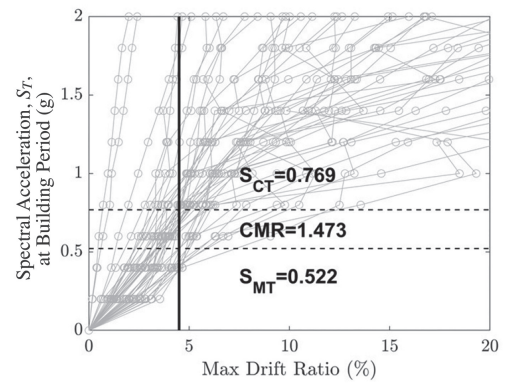
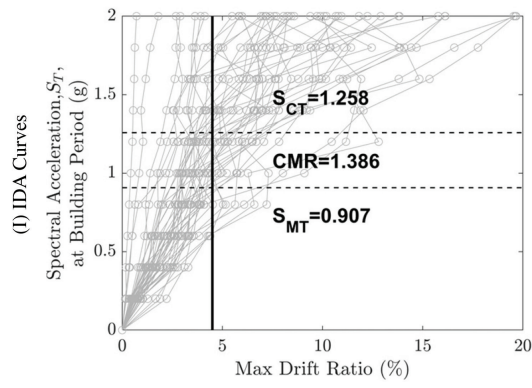


(c) M-HW-3



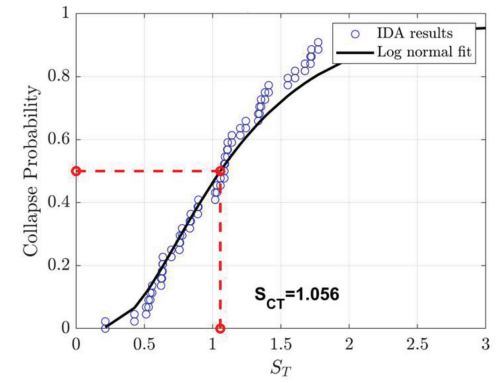
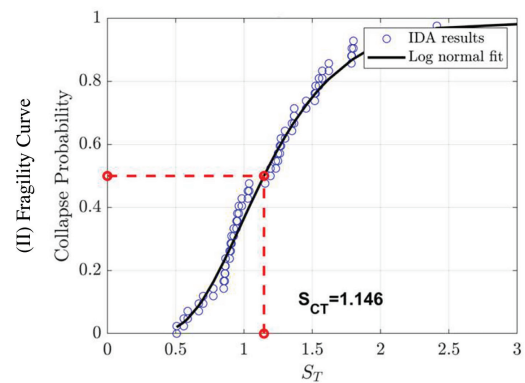
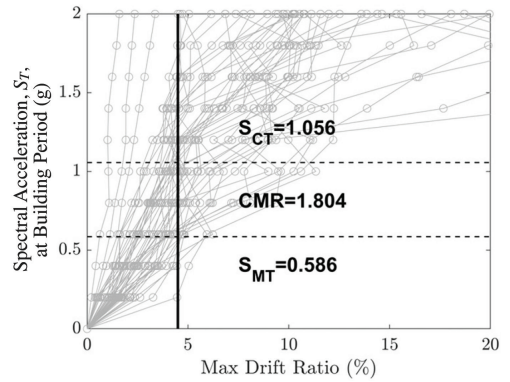
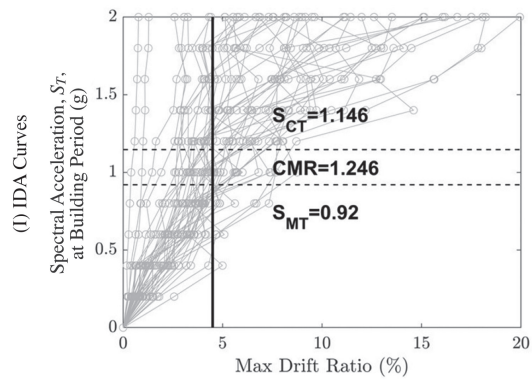
(d) M-HW-4

Fig. 14. IDA and fragility curves for heavy wall archetype buildings.



(a) C-HR-1

(b) C-HR-2R



(c) C-HR-3

(d) C-HR-4R

Fig. 15. IDA and fragility curves for clear span heavy roof archetype buildings.

Table 6. Summary of Archetypes Analyses Results: Collapse Drift Limit of 4.5% ($R = 3.5$, $\Omega_o = 3$, $C_d = 3$)

Archetype	Ω_o	μ_T	S_{CT}	S_{MT}	CMR	SSF	ACMR	$ACMR_{10\%}$	$ACMR_{20\%}$	Check (Pass/Fail)
	(–)	(–)	(g)	(g)	(–)	(–)	(–)	(–)	(–)	
M-HW-1	1.7	1.87	1.00	0.58	1.72	1.21	2.09	1.70	1.42	Pass
M-HW-2	1.8	1.50	0.75	0.50	1.48	1.17	1.73	1.65	1.39	Pass
M-HW-3	1.7	1.88	0.88	0.55	1.60	1.22	1.94	1.70	1.42	Pass
M-HW-4	1.3	1.66	0.61	0.45	1.36	1.19	1.62	1.67	1.40	Pass
Average							1.85	1.68	—	Pass
C-HR-1	2.4	1.75	1.26	0.91	1.39	1.15	1.60	1.68	1.41	Pass
C-HR-2R	2.1	>1.39	0.77	0.52	1.47	1.15	1.69	1.64	1.38	Pass
C-HR-3	2.4	2.31	1.15	0.92	1.25	1.17	1.46	1.71	1.42	Pass
C-HR-4R	2.7	>1.49	1.06	0.59	1.80	1.17	2.11	1.65	1.39	Pass
Average							1.76	1.71	—	Pass

where S_{MT} is the maximum considered earthquake (MCE) spectral acceleration at the fundamental period.

To account for the influence of spectral shape, a spectral shape factor (SSF) is applied to adjust the CMR, resulting in the adjusted collapse margin ratio ($ACMR = SSF \times CMR$). The SSF is a function of the fundamental period and the period-based ductility μ_T , and is computed using FEMA P-695 guidelines. Final ACMR values for all archetypes are summarized in Table 6.

Seismic Performance Assessment per FEMA P-695 Approach

In accordance with FEMA P-695, the seismic performance of each archetype is evaluated based on its adjusted collapse margin ratio (ACMR). Each archetype must exceed the individual acceptable collapse margin ratio ($ACMR_{20\%}$), and the performance group average must exceed the group threshold ($ACMR_{10\%}$). These limits correspond to collapse probabilities of 20% and 10%, respectively, under MCE-level ground motions.

The acceptable ACMR values are calculated assuming a lognormal distribution of collapse intensities, with a median equal to $\hat{S}_{CT}$, and a standard deviation of β_{TOT} , which represents the total uncertainty in the collapse behavior of the system calculated as:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \quad (1)$$

Following FEMA P-695 guidance, Appendix F Tables 3-1 to 3-2, and prior work (Nikoukalam et al., 2026), the component uncertainties were taken as:

- $\beta_{DR} = 0.20$, reflecting professionally engineered and peer-reviewed designs, (B) Good confidence in Table 3-1.

- $\beta_{TD} = 0.20$, based on availability of high-quality shake table and subassembly test data, (B) Good confidence in Table 3-2.
- $\beta_{MDL} = 0.10$, due to the validated high-fidelity and SDOF models, (A) Superior confidence in Table 3-3.
- $\beta_{RTR} = 0.1 + 0.1\mu_T \leq 0.4$, representing record-to-record variability.

Table 6 summarizes the key parameters for all archetypes, including overstrength (Ω_o), period-based ductility (μ_T), median collapse capacity ($\hat{S}_{CT}$), MCE spectral intensity (S_{MT}), collapse margin ratio (CMR), spectral shape factor (SSF), adjusted collapse margin ratio (ACMR), and the acceptance thresholds $ACMR_{10\%}$ and $ACMR_{20\%}$.

All archetypes in both performance groups—modular heavy wall (M-HW) and clear-span heavy roof (C-HR)—exceed the required collapse safety limits individually and on average. As a result, the use of seismic design parameters $R = 3.5$, $\Omega_o = 3$, and $C_d = 3$ is validated for these building types under the applied configurations.

DISCUSSION

The introduction of rafter proportioning limits, $h/b_f \leq 6$ and $A_w/A_f \leq 2.5$, was successful in developing stable post-peak response in the C-HR-2 and C-HR-4 frames. The path to this finding included additional supplementary work that sheds useful insight on the performance of these systems. Previously, it was found that purlin (bracing) stiffness was important to stable frame response; however, it was found here that once the purlin met the AISC 360, Appendix 6, bracing requirements, additional bracing stiffness did little to improve the post-peak response. Even an infinitely

stiff purlin brace in a model of C-HR-4, fully limiting LTB response to between the brace points, did not improve the post-peak ductility appreciably because web bending and the LDB mode occurred in this condition. Thus, bracing stiffness alone cannot improve the post-peak response of frames with rafters not meeting the proportioning limits.

The rafters in C-HR-2 and C-HR-4 meet strength requirements, but the post-peak failure mechanisms do not provide sufficient ductility (nor is there enough overstrength present to account for a nonductile response). The C-HR-2 and C-HR-4 rafter geometries are not typical of rolled steel sections—rolled shapes generally do not have the h/b_f or A_w/A_f ratios observed here; these proportions are more consistent with built-up plate members, which are common in metal building primary frames. In the extensive study of moment-shear interaction in plate girders, White et al. (2008) studied 148 tests, and only 14 had $h/b_f > 6$ and $A_w/A_f > 2.5$. It was concluded that “the strength behavior of these types of members is dramatically different than that of the member types (that meet the proportioning limits).” This would ultimately be realized in the AISC 360 shear provisions where the limits are utilized to determine if the girder is stable enough to develop shear post-buckling and significant tension-field action; it is also the reason moment-shear interaction need not currently be checked in AISC 360.

Across the following reports and papers (Meimand et al., 2018; Moen et al., 2019; Nikoukalam et al., 2026), and including this study, a significant number of FEMA P-695 analyses have been conducted on metal building system frames. The results of all evaluations are provided in Table 7 and cover (1) clear span and modular metal building systems with, (2) eave heights between 25 ft (7.6 m) and 45 ft (13.7 m), (3) span lengths between 50 ft (15.2 m) and 200 ft (61.0 m), (4) walls with tributary weights up to 116 psf (5550 Pa), and (5) roofs loaded up to 30 psf (1440 Pa). ASCE 7-16 OMF designs use a seismic response modification coefficient, R , to account for overstrength and ductility of 3.0. It is observed that this R is supported by a typical metal building frame overstrength of 2.0 and ductility of 2.0. All criteria of AISC 360 must be adhered to in an OMF design, and previous work shows the particular importance of meeting the Appendix 6 bracing provisions. New from this paper is the additional limitation that rafter geometric proportioning limits, consistent with the AISC 360 shear provisions for TFA, are also necessary. Taken together, the simulations provide a strong underpinning for the codified seismic design of metal building system frames in the United States.

SUMMARY AND CONCLUSION

This study evaluated the seismic performance of modular and clear span metal building systems with heavier wall

and roof loads in high seismic zones, using the FEMA P-695 methodology. The motivation for this investigation stems from the ongoing effort to assess whether the current seismic design parameters assigned to ordinary moment frames (OMFs) remain valid when applied to metal building system frames beyond the post-Northridge limits on height and mass. While a previous study (Nikoukalam et al., 2026) confirmed the adequacy of OMF seismic parameters for typical modular systems with light wall [2 psf (96 Pa)] and roof loads [<20 psf (958 Pa)], the present study specifically targeted configurations with heavier wall [116 psf (5550 Pa)] and roof [30 psf (1440 Pa)] to determine whether additional limits or design considerations are needed.

Eight archetypes were developed in collaboration with industry, covering two performance groups: modular buildings with precast concrete wall panels (M-HW) and clear span buildings with heavier-than-typical roof loading (C-HR). High-fidelity shell finite element models were used to perform nonlinear static pushover and quasi-static cyclic analyses. Each archetype was also represented by a calibrated nonlinear SDOF model for efficient collapse assessment per FEMA P-695. A collapse drift limit of 4.5% was adopted for all archetypes, based on observed nonlinear behavior and consistency with prior studies (Moen et al., 2019).

All archetypes included in the FEMA P-695 performance assessment (including redesigned versions of the two tall clear-span heavy-roof archetypes) demonstrated adequate seismic performance and met the individual and group acceptance criteria of FEMA P-695. The modular heavy wall buildings generally exhibited stable post-peak behavior governed by rafter yielding, local buckling, and panel zone yielding. In the heavy roof clear span buildings, lateral-distortional buckling of rafter was identified as a critical limit state in taller buildings [45 ft (13.7 m)] in their original designs, leading to insufficient post-peak ductility. However, through section redesign incorporating proportioning limits inspired by AISC tension field action criteria, the redesigned systems exhibited improved stability and ductility.

The results support the use of OMF seismic design parameters ($R = 3.5$, $\Omega_o = 3$, $C_d = 3$) for single-story metal building systems with roof dead loads up to 30 psf (1440 Pa) and building heights up to 45 ft (13.7 m), when supported by appropriate member proportioning and bracing practices. The findings also indicate that heavier exterior wall systems can be accommodated without compromising seismic performance. Overall, this study provides a basis for refining height and load-related design limits for OMFs in metal buildings and demonstrates that with proper detailing and proportioning, systems with increased mass demands can reliably meet seismic performance expectations.

Table 7. Summary Table of All Archetype Buildings Studied during 2018–2025

Reference	Archetype	Frame	Wall Weight ⁽²⁾	Roof Load ⁽³⁾	N × Span	Eave Height	Over-strength	Period-Based Ductility	$h/b_f^{(1)}$	$A_w/A_f^{(1)}$	Drift Limit	ACMR	ACMR _{10%}	ACMR _{20%}	Check (Pass/Fail)
Moen et al. (2019)	A1	Clear span	Heavy	Light	1 × 70 (21.3)	25 (7.6)	2.1	3.0	4.7	2.4	4.5	1.72	1.89	1.52	Pass
	A2	Clear span	Heavy	Light	1 × 70 (21.3)	45 (13.7)	1.1	4.2	5.7	4.3	4.5	2.50	2.15	1.52	Pass
	A3	Clear span	Light	Light	1 × 70 (21.3)	25 (7.6)	1.9	1.4	3.1	1.9	4.5	1.64	1.67	1.4	Pass
	A4	Clear span	Light	Light	1 × 200 (61.0)	35 (10.7)	3.3	1.7	4.2	1.3	4.5	1.72	1.64	1.38	Pass
Nikoukalam et al. (2026)	AM1	Modular	Light	Light	2 × 50 (15.2)	25 (7.6)	2.3	1.9	3.5	1.5	6.0	1.54	1.7	1.42	Pass
	AM2	Modular	Light	Light	3 × 50 (15.2)	25 (7.6)	2.0	1.6	3.7	2.8	6.0	1.52	1.66	1.39	Pass
	AM3R	Modular	Light	Light	2 × 50 (15.2)	45 (13.7)	1.9	1.5	4.4	1.7	6.0	2.07	1.65	1.39	Pass
	AM4R	Modular	Light	Light	3 × 50 (15.2)	45 (13.7)	1.7	2.0	4.9	1.8	6.0	2.20	1.72	1.43	Pass
	AM5	Modular	Light	Light	2 × 100 (30.5)	25 (7.6)	2.3	1.5	4.8	1.8	6.0	1.48	1.65	1.39	Pass
	AM7	Modular	Light	Light	2 × 100 (30.5)	45 (13.7)	2.0	1.6	4.9	2.4	6.0	1.69	1.67	1.4	Pass
	M-HW-1	Modular	Heavy	Light	2 × 50 (15.2)	45 (13.7)	1.7	1.9	3.9	1.3	4.5	2.09	1.7	1.42	Pass
This study	M-HW-2	Modular	Heavy	Light	2 × 100 (30.5)	45 (13.7)	1.8	1.5	4.9	2.5	4.5	1.73	1.65	1.39	Pass
	M-HW-3	Modular	Heavy	Heavy	2 × 50 (15.2)	45 (13.7)	1.7	1.9	4.7	1.9	4.5	1.94	1.7	1.42	Pass
	M-HW-4	Modular	Heavy	Heavy	2 × 100 (30.5)	45 (13.7)	1.3	1.7	4.8	2.4	4.5	1.62	1.67	1.4	Pass
	C-HR-1	Clear span	Light	Heavy	1 × 50 (15.2)	25 (7.6)	2.4	1.8	4.9	2.2	4.5	1.60	1.68	1.41	Pass
	C-HR-2R	Clear span	Light	Heavy	1 × 50 (15.2)	45 (13.7)	2.1	1.8	5.2	1.4	4.5	1.69	1.64	1.38	Pass
	C-HR-3	Clear span	Light	Heavy	1 × 100 (30.5)	25 (7.6)	2.4	2.3	4.4	2.2	4.5	1.46	1.71	1.42	Pass
	C-HR-4R	Clear span	Light	Heavy	1 × 100 (30.5)	45 (13.7)	2.7	1.8	4.9	1.2	4.5	2.11	1.65	1.39	Pass
						Average	2.0	1.9	—	—	—	1.80	1.71	—	Pass

⁽¹⁾ Rafter proportions are provided in the second braced span from the knee.

⁽²⁾ Heavy wall and light wall self-weight are defined as 75–116 psf (3590–5550 Pa) and ~2 psf (96 Pa), respectively.

⁽³⁾ Heavy roof and light roof dead load are defined as 30 psf (1440 Pa) and 15–20 psf (718–958 Pa), respectively.

h : rafter web depth

b_f : rafter flange width

A_w : area of the rafter web

A_f : area of the rafter flange

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APPENDIX

Table A. Pinching 04 Model Parameters for All Archetypes

		Archetype							
Parameter	Unit	M-HW-1	M-HW-2	M-HW-3	M-HW-4	C-HR-1	C-HR-2R	C-HR-3	C-HR-4R
ePf1	(kips)	89.3	104.7	82.8	88.8	22.4	20.2	41.4	46.8
ePd1	(in.)	6.2	7.1	5.9	6.4	2.5	7.4	2.4	6.8
ePf2	(kips)	178.7	209.4	165.6	177.6	44.8	40.4	82.8	93.6
ePd2	(in.)	13.2	14.4	12.1	12.8	5.1	14.8	4.9	13.7
ePf3	(kips)	223.4	261.8	207.0	222.0	56.0	50.5	103.5	117.0
ePd3	(in.)	20.8	21.3	15.8	16.1	6.8	24.6	6.1	17.4
ePf4	(kips)	202.0	224.5	141.0	200.0	37.0	30.0	72.2	70.0
ePd4	(in.)	31.5	32.0	31.5	32.0	14.7	42.0	14.6	32.4
eNf1	(kips)	-88.8	-103.6	-83.8	-88.8	-24.6	-20.1	-47.5	-45.2
eNd1	(in.)	-6.1	-7.0	-5.9	-6.2	-2.8	-7.3	-2.8	-6.6
eNf2	(kips)	-177.6	-207.2	-167.6	-177.6	-49.1	-40.1	-95.0	-90.4
eNd2	(in.)	-12.8	-14.1	-12.2	-12.7	-5.6	-14.7	-5.0	-13.2
eNf3	(kips)	-222.1	-259.0	-209.5	-222.0	-61.4	-50.1	-118.8	-113.0
eNd3	(in.)	-21.0	-20.7	-15.8	-15.9	-6.4	-25.0	-5.7	-17.4
eNf4	(kips)	-202.5	-220.6	-146.0	-200.0	-35.9	-28.0	-69.9	-70.0
eNd4	(in.)	-31.6	-32.1	-32.0	-32.0	-14.7	-42.0	-14.3	-32.4
rDispP	(-)	0.463	0.492	0.407	0.538	0.498	0.735	0.477	0.617
rForceP	(-)	0.988	0.797	0.715	0.700	0.718	0.888	0.747	0.825
uForceP	(-)	0.082	-0.330	-0.333	-0.332	-0.345	-0.097	-0.342	-0.293
rDispN	(-)	0.463	0.492	0.407	0.538	0.498	0.735	0.477	0.617
rForceN	(-)	0.988	0.797	0.715	0.700	0.718	0.888	0.747	0.825
uForceN	(-)	0.082	-0.330	-0.333	-0.332	-0.345	-0.097	-0.342	-0.293
gK1	(-)	-0.000124	0.0004701	0.0003455	2.41E-05	-0.000695	7.75E-05	-5.90E-04	7.73E-05
gK2	(-)	-0.000124	0.0004701	0.0003455	2.41E-05	-0.000695	7.75E-05	-5.90E-04	7.73E-05
gK3	(-)	-0.000124	0.0004701	0.0003455	2.41E-05	-0.000695	7.75E-05	-5.90E-04	7.73E-05
gK4	(-)	-0.000124	0.0004701	0.0003455	2.41E-05	-0.000695	7.75E-05	-5.90E-04	7.73E-05
gKLim	(-)	-0.000124	0.0004701	0.0003455	2.41E-05	-0.000695	7.75E-05	-5.90E-04	7.73E-05
gD1	(-)	-8.86E-05	0.0005727	1.86E-05	-0.0004713	-0.0001199	-1.84E-05	-4.84E-05	7.24E-05
gD2	(-)	-8.86E-05	0.0005727	1.86E-05	-0.0004713	-0.0001199	-1.84E-05	-4.84E-05	7.24E-05
gD3	(-)	-8.86E-05	0.0005727	1.86E-05	-0.0004713	-0.0001199	-1.84E-05	-4.84E-05	7.24E-05
gD4	(-)	-8.86E-05	0.0005727	1.86E-05	-0.0004713	-0.0001199	-1.84E-05	-4.84E-05	7.24E-05
gDLim	(-)	-8.86E-05	0.0005727	1.86E-05	-0.0004713	-0.0001199	-1.84E-05	-4.84E-05	7.24E-05
gF1	(-)	4.51E-05	-0.0005971	0.0002777	0.000522	0.0002581	2.17E-05	2.01E-04	6.78E-05
gF2	(-)	4.51E-05	-0.0005971	0.0002777	0.000522	0.0002581	2.17E-05	2.01E-04	6.78E-05
gF3	(-)	4.51E-05	-0.0005971	0.0002777	0.000522	0.0002581	2.17E-05	2.01E-04	6.78E-05
gF4	(-)	4.51E-05	-0.0005971	0.0002777	0.000522	0.0002581	2.17E-05	2.01E-04	6.78E-05
gFLim	(-)	4.51E-05	-0.0005971	0.0002777	0.000522	0.0002581	2.17E-05	2.01E-04	6.78E-05
gE	(-)	0.50	0.48	0.48	0.52	0.50	0.49	0.48	0.50
dmgType	(-)	Energy	Energy	Energy	Energy	Energy	Energy	Energy	Energy

Design Recommendation for Two-Sided Bolted Rectangular HSS End-Plate Connections in Axial Tension

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ABSTRACT

Recent studies conducted on the design of bolted hollow structural section end-plate connections have demonstrated that the use of the plate ultimate stress in place of the plate yield stress provides an improved correlation to experimental data. This research applies this flexural strength model to two-sided bolted rectangular hollow section end-plate connections to achieve a heightened degree of consistency across North American end-plate connection design. An updated design approach is compared to historical databases, and a suitable resistance factor is computed using the numerical Monte Carlo simulation (MCS) technique. The MCS is compared against the reliability assessment method of AISI S100, as well as a recent reformulation of the separation factor approach, providing an opportunity for the evaluation of these approaches. A design example is then given to illustrate the recommended design procedure, in LRFD.

KEYWORDS: hollow structural section, bolted connection, end-plate splice, prying action, design recommendation.

INTRODUCTION

Bolted end-plate connections are prevalent in structures that incorporate hollow structural sections (HSS), as they allow for larger spans, changes in cross section, and the ease of assembly. In round HSS connections, the bolts are arranged in a ring around the tube, whereas for square and rectangular HSS, the bolts are arranged in rows along either all four sides of the HSS or along two sides only. While four-sided bolted connections are more common in practice, two-sided connections, seen in Figure 1, still enjoy considerable usage.

Due to the bolts being positioned outside of the HSS cross section, end-plate connections will experience prying forces on the bolts when subjected to member tensile loading. Thus, connection nominal strength equations have historically used the prying effect model of the AISC *Steel Construction Manual*, hereafter referred to as the AISC *Manual* (2023), that is derived from the tee-stub model of Struik and de Back (1969). These procedures originally used the plate yield stress (F_{yp}) in their formulation; however, work by

Thornton (1992, 1996) and Swanson (2002) showed that use of the plate ultimate stress (F_{up}), with the Struik-de Back model, provided an improved correlation between experimental and predicted results. Accordingly, the 13th Edition AISC *Manual* (2005) replaced F_{yp} with F_{up} when considering prying effects.

A similar change has been made to nominal strength models for bolted HSS end-plate connections. Works by Fidalgo and Packer (2022) with round HSS and Thornton (2017) with four-sided rectangular HSS connections (which utilized test data from Willibald et al., 2003) have both demonstrated an improved correlation using F_{up} . These changes are reflected in the recent AISC Design Guide 24, *Hollow Structural Section Connections* (DG 24) (Packer and Olson, 2024), which presents methods for the design of round HSS and both four- and two-sided rectangular HSS bolted end-plate connections. However, no study has been conducted to investigate the viability of the use of F_{up} for two-sided bolted rectangular HSS end-plate connections. Thus, DG 24 used F_{yp} for these connections only.

This study aims to address this inconsistency through the development of an updated two-sided bolted rectangular HSS end-plate design model that demonstrates an improved correlation between experimental and predicted capacities. This is achieved through the incorporation of F_{up} into the prying effect model, as well as changes based on observations from previous physical experiments. Following this, a reliability study is conducted, using a Monte Carlo simulation (MCS), to determine a resistance factor that provides a satisfactory degree of structural reliability. The use of this numerical approach produces the “true” resistance factor, which presents the opportunity for an evaluation of the

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relative accuracies of several closed-form reliability techniques. These techniques include the reliability assessment method presented in AISI S100, *North American Specification for the Design of Cold-Formed Steel Structural Members* (2020), recalibrated to apply to structural steel, as well as a recent reformulation of the separation factor approach. The resistance factors computed using these approaches are compared against the MCS resistance factor to assess the viability of the closed-form approaches and provide recommendations on the conduction of a proper reliability study for structural steel.

NOMINAL STRENGTH MODEL

Axial Tension Expression Improvement

When a two-sided bolted HSS end-plate connection is loaded in axial tension, the lateral offset between the tube and the bolts creates flexure within the end plates, which generates a prying force along the end-plate edges. Since the prying force is proportional to end-plate rotation, increasing the connection stiffness by selecting thicker end plates will reduce the prying force. Thus, two potential failure modes govern these connections: When the end plates are thin, plate plastic hinging will occur, whereas bolt rupture will govern when the end plates are sufficiently thick. This forms the basis of the prying effect model advanced by Thornton (1992) and Swanson (2002), upon which the DG 24 strength equations for four- and two-sided bolted HSS end-plate connections are based.

An assumption of the traditional prying effect model is that during plate plastification, two yield lines can form across the end plates. These yield lines are located at the

inside face of the bolts and at the element face (or the weld toe). While this behavior has been observed for tee-stubs and four-sided bolted rectangular HSS end-plate connections, the inner or hogging yield lines of the two-sided connections have been seen to occur within the tube wall dimension, rather than at the HSS weld toes. This observation is based on the work of Packer et al. (1989), whose physical experiments displayed a clear flaking of the white-wash, on the underside of the plate, within the tube wall region. This behavior is attributed to a lack of restraint from the unbolted sides, which allows for an inward progression of yielding as the member load increases, and is therefore uncharacteristic of the four-sided bolted connections.

DG 24 therefore includes the thickness of the HSS wall (t) in Equation 1 to compute b' , which represents the distance between the hogging and sagging yield lines. This parameter is then used within Equation 2 to obtain the plate thickness required to avoid prying (t_c), which, notably, uses F_{yp} . The remainder of the nominal strength model is identical to that of a four-sided connection, with computations performed on a per-bolt basis, making the connection capacity highly dependent on the ratio of the applied load per bolt without prying (T_r) to the available tensile strength per bolt (T_c). This ratio (T_r/T_c) is used to compute the prying ratio (β) and flexibility factor (α^*) in Equations 3 and 4, respectively, which are used in Equation 5 to compute the required plate thickness (t_p).

$$b' = b - \frac{d_b}{2} + t \quad (1)$$

$$t_c = \sqrt{\frac{4T_c b'}{pF_{yp}}} \quad (2)$$



Fig. 1. Examples of two-sided bolted rectangular HSS end-plate connections.

$$\beta = \frac{1}{\rho} \left(\frac{T_c}{T_r} - 1 \right) \quad (3)$$

$$\alpha^* = \min \left[\frac{1}{\delta} \left(\frac{\beta}{1-\beta} \right), 1 \right] \quad (4)$$

$$t_p = t_c \sqrt{\frac{T_r}{T_c} \left(\frac{1}{1+\delta\alpha^*} \right)} \quad (5)$$

where p is the bolt tributary width, $\rho = b'/a'$, $\delta = 1 - d_h/p$, d_h is the bolt diameter, d_h is the bolt hole diameter, and a' is the distance from the plate edge to the sagging yield line. Other geometric variables are illustrated in Figure 2. An adjusted available tensile capacity per bolt ($T_{c,adj}$) can then be computed, including the effects of prying using Equations 6 to 8, which employ the prying adjustment factor (Q). The overall connection capacity (T_a) is then simply the product of the adjusted bolt tensile strengths and the number of bolts (n_b).

$$\alpha' = \frac{1}{\delta(1+\rho)} \left[\left(\frac{t_c}{t_p} \right)^2 - 1 \right] \quad (6)$$

$$Q = \begin{cases} 1 & \alpha' \leq 0 \\ \left(\frac{t_p}{t_c} \right)^2 (1+\delta\alpha') & 0 < \alpha' \leq 1 \\ \left(\frac{t_p}{t_c} \right)^2 (1+\delta) & 1 < \alpha' < 1.5 \\ 0 & \alpha' \geq 1.5 \end{cases} \quad (7)$$

$$T_{c,adj} = T_c Q \quad (8)$$

$$T_a = T_{c,adj} n_b \quad (9)$$

In the 1980s, 23 physical experiments were performed (Figure 3) on two-sided bolted HSS end-plate connections

under axial tension (Birkemoe and Packer, 1986; Packer et al., 1989). These researchers then used the experimental results to validate a capacity prediction method. Although the method was highly accurate, practitioners did not adopt it because it required computing multiple failure mechanisms to determine the governing case. One conclusion of this work was that under axial tension loading, the bolt force eccentricity—one-half a bolt diameter, as reported by Struik and de Back (1969)—was negligible. Incorporating this into the DG 24 method changes the computation of b' to Equation 10, as the sagging yield line passes through the bolt centerlines. Additionally, the parameter a' can then be taken as a .

$$b' = b + t \quad (10)$$

The bolt tributary width (p) is defined as the plate width (w_p) divided by the number of bolts per side of the connection. This approach reduces the connection to a 2D strip, whereby the tee-stub prying effect model is applied to a pair of bolts. However, Dowswell (2011) recognized that if p is excessively large, independent yield line patterns will develop around the bolts; therefore, Equation 11 imposes a maximum bolt tributary width.

$$p_{max} = 4\sqrt{b'(a+b)} \quad (11)$$

DG 24 states that, for two-sided bolted HSS end-plate connections, the bolt centerlines should not be positioned beyond the HSS width as this may create an uneven load distribution on the bolts. For some connection geometries, equating p to the upper limit given in Equation 11 will result in excessively wide end plates that artificially boost the connection nominal strength, despite there being no actual increase in capacity. It is thus recommended that a more practical upper limit of $w_p \leq H + 4$ in. be imposed, where H is the HSS dimension parallel to the bolt line. This limit is based on the experimental connection geometries

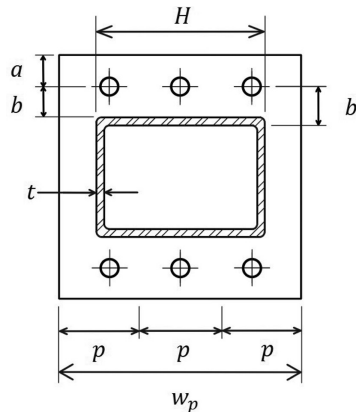


Fig. 2. Two-sided bolted end-plate connection cross section, with b' per Equation 10.

of Packer et al. (1989) and typical design practice, with the 4 in. allowing for the placement of welds. Such connection proportions will prevent unconservative strength prediction and the development of independent yield line patterns. Using this limit, an alternative formulation for p_{max} is shown in Equation 12.

$$p_{max} = \frac{H + 4 \text{ in.}}{n_b/2} \quad (12)$$

Use of F_{up} in lieu of F_{yp} in Equation 2 is also recommended for consistency with other bolted end-plate nominal strength models in DG24.

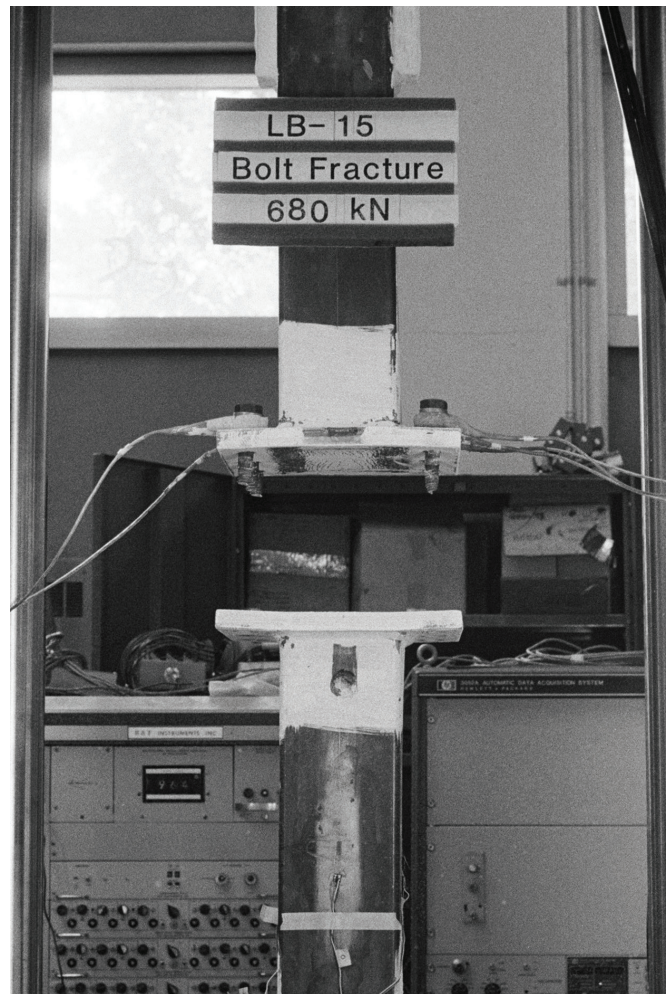
In order to evaluate the proposed capacity prediction method, 23 laboratory experiments by Birkemoe and Packer (1986) and Packer et al. (1989) are considered, with the measured properties listed in Table 1. The experimental results (T_{nx}) and corresponding failure modes are listed

in Table 2, along with the predicted capacities based on the nominal strength method presented herein, using both F_{yp} and F_{up} (denoted in Table 2 as T_{ny} and T_{nu}). In some cases, the value of α' exceeded 1.5, indicating excessive prying, which, using Equation 7, will give a connection capacity equal to zero in Table 2. The test LB-16 was not considered in the capacity prediction method evaluation due to its governing limit state—tube yielding—not being considered as a connection failure mode.

A comparison between the experimental results and predicted capacities, using the recommendations made herein, is seen in Figure 4. The discrepancy in the number of tests (n) assessed for F_{yp} and F_{up} is due to the exclusion of data when α' exceeded 1.5 (shown by N/A in Table 2). Figure 4 (in which both data sets use Equation 10 for b') shows that the use of F_{up} significantly reduces the mean value relative to F_{yp} [while the two coefficients of variation (CoVs) are very similar], indicating an improved correlation.



(a) Experimental setup



(b) Ultimate failure

Fig. 3. Experiments on two-sided bolted end-plate connections, using bolt load-indicating washers.

Table 1. Measured Geometric and Material Properties of Experimental Specimens

Test Name	h (in.)	b (in.)	t (in.)	n_b	d_b (in.)	F_{ub} (ksi)	w_p (in.)	a (in.)	b (in.)	t_p (in.)	F_{yp} (ksi)	F_{up} (ksi)
LB-1	4	4	0.225	4	0.625	98.1	6	1.63	1.75	0.794	43.5	71.1
LB-2	4	4	0.225	4	0.625	104	6	1.63	1.75	0.500	41.6	65.0
LB-3	4	4	0.225	4	0.75	106	6	1.63	1.75	1.01	39.9	69.3
LB-4	5	7	0.188	6	0.75	106	9	1.18	2.17	1.01	39.9	69.3
LB-5	5	7	0.188	6	0.75	106	9	1.18	1.77	1.01	39.9	69.3
LB-6	5	7	0.188	6	0.75	106	9	1.18	1.38	1.01	39.9	69.3
LB-7	5	7	0.188	6	0.75	106	9	1.57	1.38	1.01	39.9	69.3
LB-8	5	7	0.188	6	0.75	106	9	2.17	1.38	1.01	39.9	69.3
LB-9	5	7	0.188	4	0.75	106	9	1.18	1.38	1.01	39.9	69.3
LB-10	5	7	0.188	6	0.625	103	9	1.18	1.38	1.01	39.9	69.3
LB-11	5	7	0.188	6	0.875	108	9	1.18	1.38	1.01	39.9	69.3
LB-12	5	7	0.188	6	0.875	106	9	1.18	1.38	0.895	48.6	81.1
LB-13	5	7	0.188	6	0.875	104	9	1.18	1.38	0.639	42.5	67.6
LB-14	5	7	0.188	6	0.625	104	9	1.18	1.38	0.500	41.6	65.0
LB-15	5	7	0.188	6	0.625	98.1	9	1.18	1.38	0.794	43.5	71.1
LB-16	5	7	0.188	6	0.875	108	9	1.97	1.38	1.01	39.9	69.3
A	4	4	0.225	4	0.625	111	5.12	1.18	1.46	0.794	43.5	71.1
B	4	4	0.225	4	0.625	111	5.12	1.16	1.85	0.794	43.5	71.1
C	4	4	0.225	4	0.625	111	5.12	1.18	1.65	0.794	43.5	71.1
D	4	4	0.225	4	0.625	111	5.12	1.97	1.61	0.794	43.5	71.1
E	4	4	0.225	4	0.75	112	5.12	1.75	1.63	0.794	43.5	71.1
F	4	4	0.225	4	0.75	112	5.12	1.16	1.85	1.01	39.9	69.3
G	4	4	0.225	4	0.75	112	5.12	1.18	1.85	0.895	48.6	81.1

RELIABILITY STUDY

Structural reliability is achieved through load and resistance factor design (LRFD), where a design is considered safe if the product of the nominal capacity (R_n) and a corresponding resistance factor (ϕ) exceeds the product of the nominal load effects ($S_{n,i}$) and constituent load factors (γ_i).

$$\phi R_n \geq \sum \gamma_i S_{n,i} \quad (13)$$

Load and resistance factors are intended to account for the uncertainty within the nominal terms and are based on probabilistic analyses. Whereas load factors are presented as load combinations in so-called loading codes [e.g., ASCE 7 (2022), NBC (NRC, 2020)], resistance factors are specific to nominal strength equations and must be computed on a case-by-case basis. This establishes the notion of target structural reliability as resistance factors must be selected such that their use within LRFD produces an acceptable probability of failure. When probabilistic distributions are used to represent the resistance (R) and the load effect (S),

the failure probability (p_F) can be determined using their difference, termed the safety margin (G):

$$p_F = p(G < 0) \quad (14)$$

Ellingwood et al. (1980) more usefully represent the failure probability using the reliability index (β^+), which is equal to the number of standard deviations (σ_G) from the mean value of the safety margin ($\bar{G}$) to the failure condition.

$$\beta^+ = \frac{\bar{G}}{\sigma_G} \quad (15)$$

In the United States, benchmark values of the reliability index and their associated failure probabilities are defined in ASCE 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (2022), for a 50 yr design life. These target reliability indices are based on both risk category and failure mode, though it is typically assumed that design equations apply to Risk Category II structures. Thus, ductile failure has a target reliability

Table 2. Connection Experimental and Predicted Capacities						
Test Name	T_{nx} (kips)	Experimental Failure Mode	T_{ny} (kips)	T_{nu} (kips)	T_{nx}/T_{ny}	T_{nx}/T_{nu}
LB-1	99.6	Bolt fracture	63.3	77.6	1.57	1.28
LB-2	78.7	Plate plastification	0	0	N/A	N/A
LB-3	140	Bolt fracture	97.1	122	1.44	1.15
LB-4	178	Bolt fracture	123	161	1.45	1.11
LB-5	193	Bolt fracture	138	181	1.40	1.07
LB-6	215	Bolt fracture	158	207	1.36	1.04
LB-7	218	Bolt fracture	164	208	1.33	1.05
LB-8	219	Bolt fracture	172	208	1.27	1.05
LB-9	179	Bolt fracture	128	141	1.40	1.27
LB-10	179	Bolt fracture	128	142	1.40	1.26
LB-11	252	Bolt fracture	192	242	1.31	1.04
LB-12	243	Bolt fracture	187	230	1.30	1.06
LB-13	209	Plate plastification	0	0	N/A	N/A
LB-14	110	Plate plastification	0	82.6	N/A	1.33
LB-15	153	Bolt fracture	103	132	1.48	1.16
LB-16	262	HSS yielding	199	253	1.32	1.03
A	103	Bolt fracture	66.8	82.4	1.54	1.25
B	87.5	Bolt fracture	58.5	72.2	1.50	1.21
C	92.2	Bolt fracture	62.5	77.1	1.47	1.20
D	98.9	Bolt fracture	66.1	83.1	1.50	1.19
E	122	Bolt fracture	0	104	N/A	1.17
F	120	Bolt fracture	84.9	110	1.42	1.10
G	121	Bolt fracture	80.7	105	1.50	1.15

index of 3.0, and brittle failure has a recommended index of 4.0. Herein, the computation of a resistance factor for the two-sided bolted HSS end-plate connections is performed using a target reliability index of 4.0, due to the potential for sudden bolt fracture to govern the connection design.

Several practical methods exist for the computation of resistance factors, which are said to achieve an acceptable probability of failure by using the target reliability index as an input. Herein, a resistance factor is computed using a Monte Carlo simulation (MCS), a numerical technique that converges to the true solution by performing a large number of iterations. A single MCS iteration simulates a design scenario: Values of all participating material and geometric variables are randomly sampled from probabilistic distributions that reflect actual parametric variation and are used to compute the corresponding nominal capacity. This process is repeated for the load effect and then compared against the capacity to determine the degree of structural reliability under these conditions. When this process is repeated a large number of times across a range of representative

connections, the MCS converges on the overall reliability of the nominal strength model. A suitable resistance factor is selected based on the reliability index distributions produced by trial resistance factor values, relative to the target reliability index.

Rudman et al. (2022) present detailed steps for the completion of a MCS; however, slight adaptations were required herein due to the nature of the nominal strength model. Unlike typical models, where a capacity is computed and compared against the load effect, the method presented herein calculates an initial plate thickness using an applied load per bolt (T_r), which is then rounded upward to a discrete value. This is incorporated in the MCS by randomly generating an applied load per bolt, which informs the selection of the requisite bolt diameter and allows for the initial plate thickness to be computed. Randomly sampled values are only employed after this discretization because designers would use nominal values to arrive at this step. MATLAB (MathWorks, 2023) was used to perform the simulations, using a range of live-to-dead load (L/D) ratios

of $0 \leq L/D \leq 3$ and consisted of 1 million iterations per L/D and representative connection. To ensure a conservative resistance factor, representative reliability index values for each trial resistance factor were taken as the minimum across the range of L/D . The calibration of the resistance factor using the minimum reliability index ensures a standardized approach across all limit states in LRFD, and follows similar studies by Rudman et al. (2022) and Rahbarimanesh and Tousignant (2024).

Representative Connections

The representative connections used in the MCS vary based on randomly generated applied loads, though finite ranges of input parameters are possible. For this study, the connection geometrical parameters were defined based on the following ranges: $\frac{3}{16}$ in. $\leq t \leq \frac{1}{2}$ in., 1.5 in. $\leq a, b \leq 2$ in., $\frac{1}{2}$ in. $\leq d_b \leq 1$ in., and 2 in. $\leq p \leq 4$ in. A value of 65 ksi was selected as the tensile strength of the end plates, based on the minimum specified tensile strength of ASTM A572 Grade 50 (ASTM, 2021). Bolt tensile strength was assigned a value of 120 ksi, based on the minimum specified tensile strength of ASTM F3125 Grade A325 bolts (ASTM, 2025). Due to the nominal strength model performing computations on a per-bolt basis, an input number of bolts was not required for the MCS, though typical bolting arrangements were considered to derive the ranges of geometric

parameters. A total of 30 representative connections were thus generated for each resistance factor.

Statistical Parameters

Computation of the nominal capacity of two-sided bolted end-plate connections requires the following material and geometric parameters: plate ultimate stress, HSS thickness, plate thickness, bolt diameter, distance to bolt holes, bolt tributary width, number of bolts, and bolt tensile strength (which is a function of the bolt ultimate stress and the bolt hole diameter). Excluding the number of bolts, log-normal probabilistic distributions for the variation in all parameters were created using the corresponding mean ratio and coefficient of variation (CoV). Cumulative distribution functions (CDF) required for the MCS were then created from these distributions.

Statistical parameters describing the variation of ASTM A572 Grade 50 plate ultimate stress were taken from data collated by Lui et al. (2007), specifically pertaining to plate thicknesses greater than $\frac{1}{2}$ in. The variation in the tensile strength of ASTM F3125 Grade A325 bolts was taken from Moore et al. (2008), who also produced statistical parameters for Grade A325 bolt diameters. The variation in plate width and thickness for ASTM A572 Grade 50 were obtained from a study by Schmidt and Bartlett (2002a). Plate width statistics are assumed to be representative of all

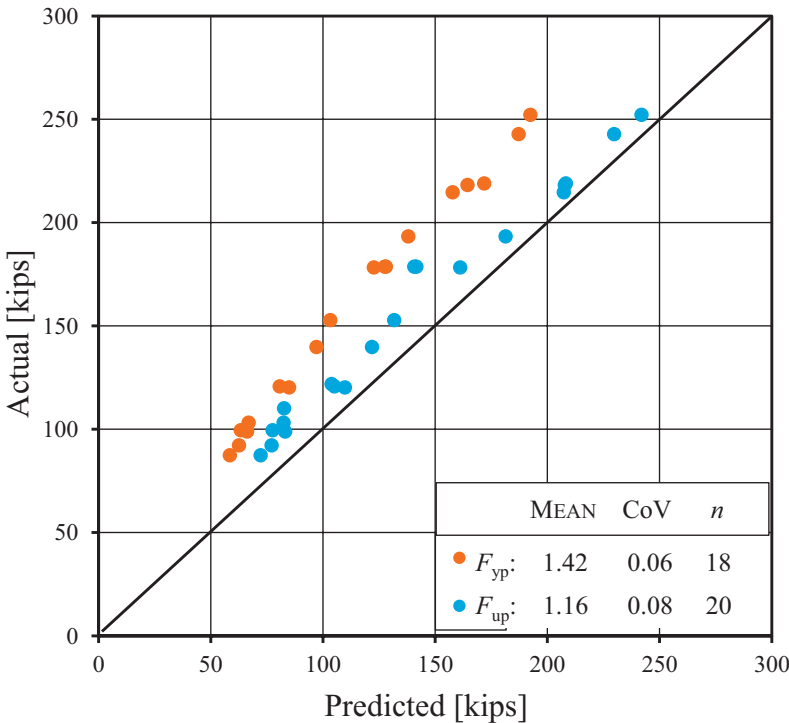


Fig. 4. Comparison of actual to nominal capacities using both the plate yield and ultimate stress.

Table 3. Statistical Parameters Describing Each Participating Variable			
Variable	Source	Mean (Actual/Nominal) Ratio	CoV
F_{up}	Liu et al. (2007)	1.26	0.070
F_{ub}	Moore et al. (2008)	1.22	0.047
d_b		0.994	0.003
t	Schmidt and Bartlett (2002a)	0.975	0.025
t_p		1.04	0.025
b, p		0.998	0.004
D	Schmidt and Bartlett (2002b)	1.05	0.100
L		0.78	0.320

in-plane (fabricator-dependent) plate geometric parameters, including the bolt tributary width (p) and the distance from the bolt hole centers to the tube face (b). This study also surveyed the thickness of ASTM A500 (2023) Grade B/C HSS. Statistical parameters that describe the variation in the load effect terms are also required to perform the MCS. Probabilistic distributions of the dead and live load using normal and Gumbel distributions, respectively, were produced by Ellingwood et al. (1980) and Ellingwood and Culver (1977). These were both converted to equivalent log-normal distributions by Schmidt and Bartlett (2002b), from which statistical parameters were taken. A summary of these values is seen in Table 3.

A probabilistic distribution is required for the professional factor, which measures the variation in accuracy of the nominal strength method. This distribution was created using the actual-to-predicted data from Figure 4 and was fitted with a log-normal curve using a maximum likelihood

estimation (MLE) technique, resulting in Figure 5(a). From this, a CDF was produced, as seen in Figure 5(b).

Results

Three trial resistance factors were assessed using the MCS, with values of $\phi = 0.90, 0.95$, and 1.00 . Figure 6 presents the reliability index distributions associated with each trial resistance factor over the range of $0 \leq L/D \leq 3$. The reliability index distributions exhibit a dip in reliability at L/D of 0.1 due to the change from the principal dead to the principal live load combination. However, Figure 6 shows that the minimum reliability is created at $L/D = 3$. All three curves are observed to exceed the target reliability index of 4.0 over the full range of L/D , and thus all trial resistance factors achieve satisfactory structural reliability. However, a (very conservative) resistance factor value of $\phi = 0.90$ is selected to align with the current recommendations in

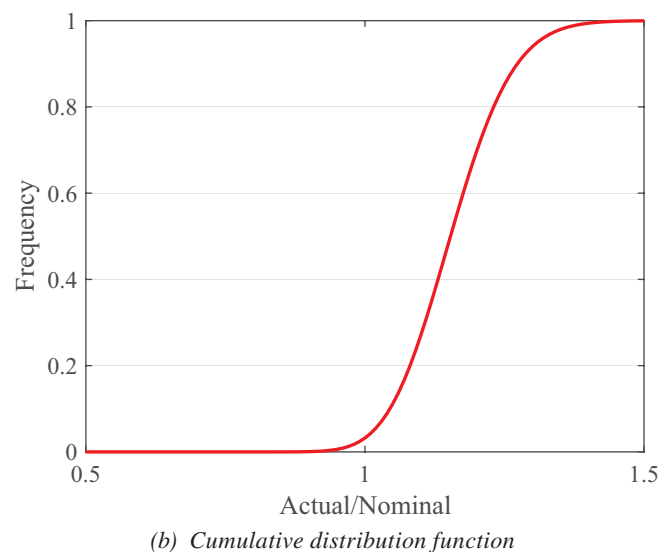
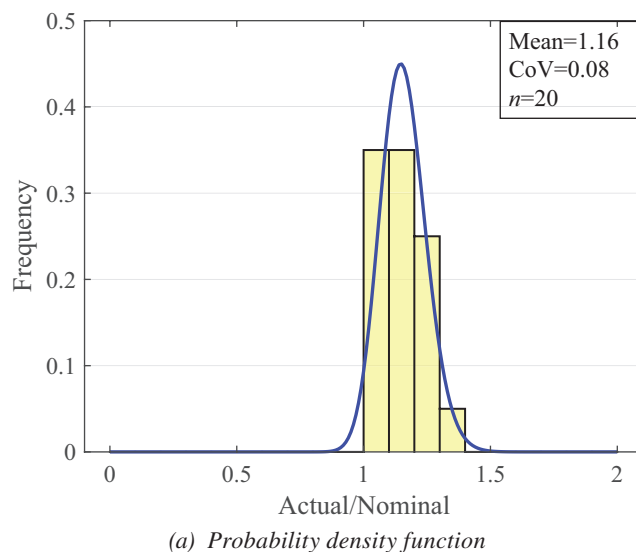


Fig. 5. Professional factor probabilistic distributions for the nominal strength model recommended by the authors.

DG 24 for other HSS end-plate connections, as well as the prying effect model given in Part 9 of the AISC *Manual* (2023).

The MCS was conducted using statistical parameters for ASTM A572 Grade 50 plate and ASTM F3125 Grade A325 bolts, thus the computed resistance factor may not be applicable if alternate material grades are used in design. However, the resistance factor of $\phi = 0.90$ is shown in Figure 6 to give reliability well in excess of the target reliability index of 4.0, which expands the validity of the design model. Studies by Xi and Packer (2021) and Rudman et al. (2022) have shown that the MCS results may be sensitive to the mean ratios of key material parameters. A sensitivity analysis was therefore undertaken to determine the effects of variation in the mean values of F_{up} and F_{ub} , with the results shown in Figure 7. This figure shows that the resistance factor of $\phi = 0.90$ still produces adequate structural reliability when the mean ratios of these parameters are as low as 1.10. Applicability for mean ratios of 1.10, for both F_{up} and F_{ub} simultaneously, allow for this model to be applicable to most structural steel and structural bolt grades. This value is less than or equal to the mean reported for 34 of 35 structural steel grades covered in AISC 341, *Seismic Provisions for Structural Steel Buildings* (2022a), and all ASTM F3125 bolt grades reviewed by Moore et al. (2008). Thus, the recommended resistance factor can be confidently extended to cover the vast majority of grades of structural steel and structural bolts. The CoVs of these parameters were not varied within the sensitivity analysis because V_R is dominated by the CoV of the professional factor, V_P .

Because the nominal strength model is most commonly used to compute a plate thickness, the resistance factor must be used within the determination of this parameter instead

of for the determination of connection capacity. However, DG 24 also allows designers to select the plate thickness that avoids prying. Therefore, it is recommended that the plate thickness needed to avoid prying be computed using Equation 16, with $\phi = 0.90$. This allows the resistance factor to be inversely proportional to the design plate thickness and proportional to the connection capacity. A comparison of actual and factored nominal (available) capacities using the recommended design method (where Equation 16 is used in lieu of Equation 2, and Equation 10 is used in lieu of Equation 1), is presented in Figure 8. In this figure, the mean ratio is seen to be adequately conservative, and all data points lie above the 45° line.

$$t_c = \sqrt{\frac{4T_c b'}{\phi p F_{up}}} \quad (16)$$

AISI S100

The current study presents an opportunity for the evaluation of the reliability assessment method referred to as Approach 1 in AISI S100, Chapter K (2020), which is intended for the determination of the design strength by experimentation of replicate laboratory specimens. This approach is evaluated via a comparison to the MCS solution using the experimental results of two-sided bolted HSS end-plate connections.

AISI S100 states that the resistance factor shall be computed using Equation 17:

$$\phi = C_\phi M_m F_m P_m e^{-\beta^+ \sqrt{V_M^2 + V_F^2 + C_p V_P^2 + V_Q^2}} \quad (17)$$

where M_m and V_M , F_m and V_F , and P_m and V_P are the mean values and CoVs of the material, fabrication, and professional factor, respectively. The sample size correction factor, C_p , is computed using Equation 18.

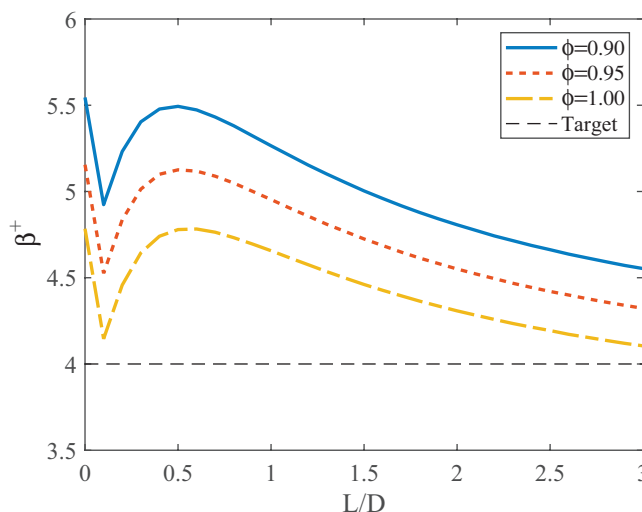


Fig. 6. Reliability index distributions for each trial resistance factor value.

$$C_p = \frac{(1+1/n)m}{(m-2)} \text{ for } n \geq 4 \quad (18)$$

$$= 5.7 \text{ for } n = 3$$

where $m = n - 1$, and n is the number of specimens. Work by Meimand and Schafer (2014) has shown that this reliability

assessment method can be extended to apply for various materials and associated statistical parameters. The AISI S100 method is intended to apply to light-gauge steel structural elements; thus the code is calibrated for $L/D = 5$. The parameter C_ϕ is computed using Equation 19; using load factors from ASCE 7 (2022), the statistical parameters for the dead and live load effect from Ellingwood et al. (1980)

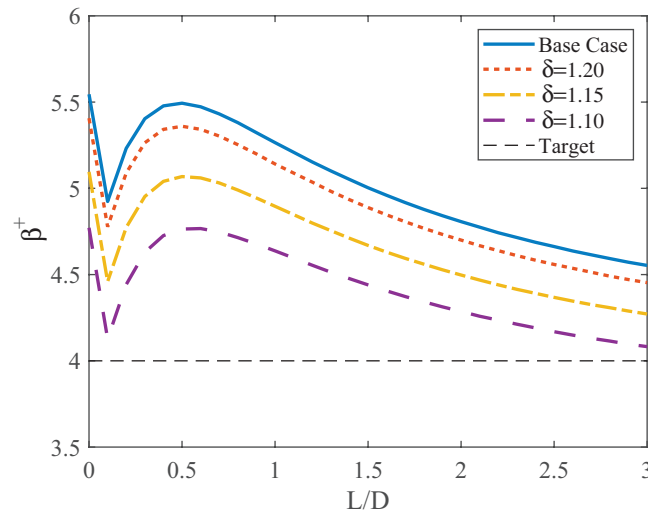


Fig. 7. Sensitivity analysis reliability index distributions for variations in the mean value of the plate and bolt ultimate stresses.

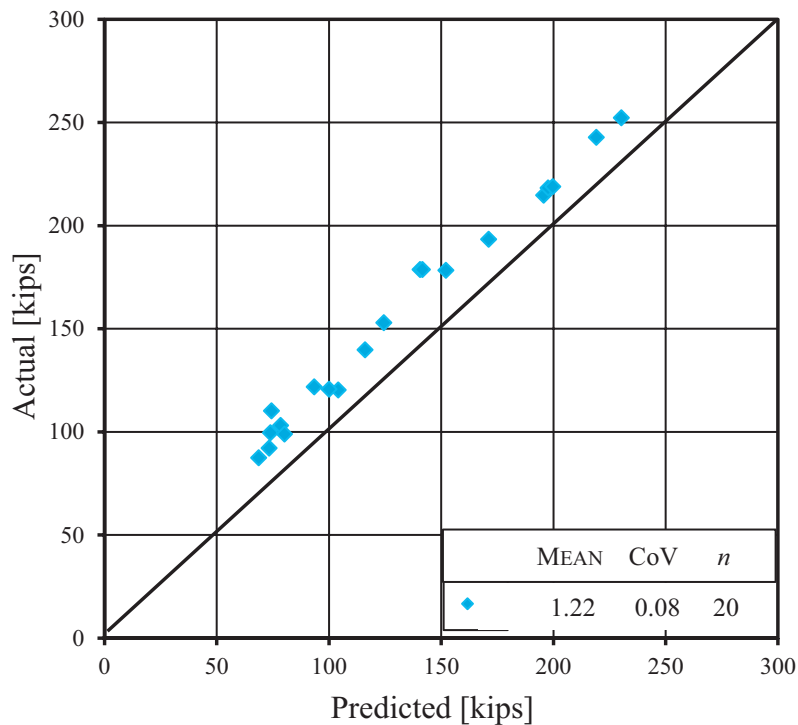


Fig. 8. Comparison of actual-to-factored nominal capacities using the recommended design model.

and $L/D = 5$ produces a value of 1.52, as given in AISI S100 (2020). Recalculating this parameter with $L/D = 3$, the ratio used to calibrate AISC 360, *Specification for Structural Steel Buildings* (2022b), yields a magnitude of 1.48.

$$C_\phi = \frac{\gamma_D + \gamma_L L/D}{B_D + B_L L/D} \quad (19)$$

$$C_\phi = \frac{1.2 + (1.6)(3)}{1.05 + (1)(3)} = 1.48$$

where γ_D and γ_L are the dead and live load factors and B_D and B_L are the dead and live load bias coefficients, respectively. Similarly, the CoV of the load effect can be recomputed, using Equation 20, yielding a value of 0.19 instead of the value of 0.21 given in AISI S100:

$$V_Q = \frac{\sqrt{(B_D V_D)^2 + (B_L V_L L/D)^2}}{B_D + B_L L/D} \quad (20)$$

$$V_Q = \frac{\sqrt{[(1.05)(1)]^2 + [(1)(0.25)(3)]^2}}{1.05 + (1)(3)} = 0.19$$

The mean values and CoVs for the material and fabrication factors are given in AISI S100, Table K2.1.1-1 (2020), but are intended for light-gauge steel elements and connections.

Thus, these parameters can be supplanted by credible documented statistical data. In the case of two-sided bolted HSS end-plate connections, several material and fabrication variables contribute unequally to the nominal strength. However, the partial derivative approach detailed in Kennedy and Gad Aly (1980), which is typically used in such cases, cannot provide a reasonable approximation of material and fabrication factors due to the discretization of the

end-plate thickness. This approach is therefore assessed twice: once using values from Table K2.1.1-1 for bolted connections in AISI S100 (2020) and again using the minimum mean values and maximum CoVs seen in Table 3 for both the material and fabrication factors ($M_m = 1.22$, $V_M = 0.070$, $F_m = 0.975$ and $V_F = 0.025$), which are taken from the literature. AISI recommends the use of target reliability indices of 2.5 and 3.5 for structural members and connections in LRFD, respectively, which is not consistent with ASCE 7 (2022) nor Section B3.1 of the AISC 360 Commentary (2022), where the latter states that the resistance factors presented therein achieve an implied reliability index of 4.0 for connections. Thus, target reliability indices of 3.5, per AISI S100 (2020), and 4.0, per AISC 360 Commentary Section B3.1 (AISC, 2022b) and ASCE 7 (2022), are also used to assess this approach. The results of these analyses are shown in Figure 9.

Figure 9 shows that, as expected, use of a reduced target reliability index ($\beta^+ = 3.5$ vs. $\beta^+ = 4.0$) produces less conservative results for ϕ . Using $\beta^+ = 4.0$, both predictions for ϕ are conservative, though the use of statistical parameters from the literature provides a better approximation to the true (MCS) solution. This is due to the mean values and CoVs given in AISI S100 being very conservative, particularly for the material mean overstrength. The use of generally larger mean values and smaller CoVs, as presented in Table 3, produces higher resistance factors.

Separation Factor Approach

The disassociation of the load and resistance terms of Equation 13 using a separation factor (α_R) forms the basis of separation factor-based approaches (SFA), a group of reliability assessment methods widely used to compute resistance

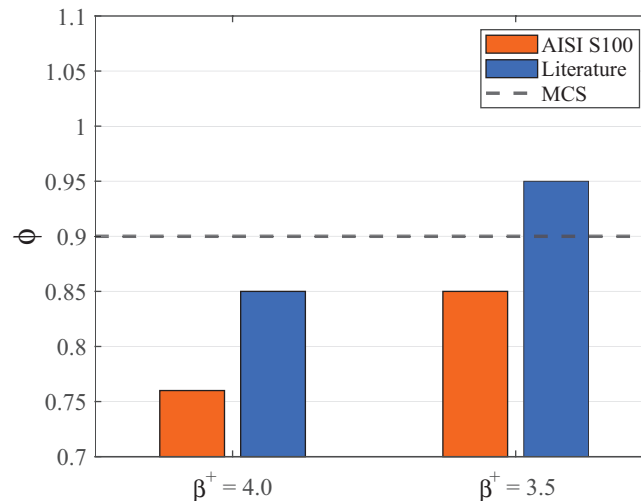


Fig. 9. Comparison of AISI S100 (2020) reliability assessment method against MCS.

factors in North America over the past half century (Lind, 1971; Galambos and Ravindra, 1973). The general form of the method is shown as Equation 21, where the load ratio correction factor (C_R), used to correct the solution when $\beta^+ \neq 3.0$, is computed using Equation 22 (Franchuk et al., 2002).

$$\phi = C_R \rho_R \exp(-\beta^+ \alpha_R V_R) \quad (21)$$

$$C_R = 0.0062\beta^{+2} - 0.131\beta^+ + 1.338 \quad (22)$$

where ρ_R and V_R refer to the bias coefficient and the CoV of the resistance, respectively, originally taken as the mean ratio and CoV of the professional factor. However, recognition that material and fabrication factors can influence the reliability of a nominal expression led to researchers taking the bias coefficient as the product of the mean values of the material, fabrication, and professional factors. Similarly, the CoV of the resistance is typically taken as the square root of the sum of the squares of the three individual CoVs. This principle is reflected in many North American reliability assessment methods (e.g., Equation 17) and is termed the expanded separation factor approach (ESF).

A value of 0.55 for α_R , suggested by Galambos and Ravindra (1973), is most common, though other definitions exist [e.g., ASCE 7 (2022) suggests 0.7, Lind (1971) suggests 0.75]. Jollymore et al. (2025) recently performed a review of the SFA, finding that so long as V_R was reasonably low, different values of α_R best approximate the true solution for unique target reliability indices. Recommendations by Jollymore et al. are summarized in Equation 23, where C_R is not required.

$$\phi = \begin{cases} \rho_R \exp(-0.55\beta^+ V_R; 0.05) \leq V_R \leq 0.30, \beta^+ \approx 3.0 \\ \rho_R \exp(-0.70\beta^+ V_R; 0.10) \leq V_R \leq 0.40, \beta^+ \approx 3.5 \\ \rho_R \exp(-0.80\beta^+ V_R; 0.10) \leq V_R \leq 0.40, \beta^+ \approx 4.0 \end{cases} \quad (23)$$

The experimental results of two-sided bolted HSS end-plate connections are also used to evaluate the Jollymore-corrected SFA via comparison to the true (MCS) resistance factor. Use of a target reliability index of 4.0 corresponds to $\alpha_R = 0.8$, and material and fabrication factors are taken as the minimum mean values and maximum CoVs found in Table 3.

Use of the traditional $\alpha_R = 0.55$ and Equations 21 and 22 leads to a resistance factor of $\phi = 1.00$, whereas the use of $\alpha_R = 0.80$ per Equation 23 with Equation 21 gives $\phi = 0.90$. The use of the larger Jollymore-corrected α_R values in Equation 23 will always lead to more conservative results when $\beta^+ > 3.0$. Figure 6 shows that both of these resistance factor values exceeded the target reliability index across the full range of L/D .

CONCLUSIONS

This paper presents a reformulation of the AISC DG 24 (Packer and Olson, 2024) design method for two-sided bolted rectangular HSS end-plate connections under axial tension, including the use of F_{up} in lieu of F_{yp} . These changes create uniformity across the rectangular HSS bolted end-plate connection design models in DG 24 and also with the prying effect model of the AISC *Manual* (2023). The revised design method is verified through comparison between actual and predicted connection capacities. A reliability study was conducted using a Monte Carlo simulation (MCS) to determine a suitable resistance factor for the design method that met the ASCE 7 (2022) recommended target reliability index of 4.0. It is thus recommended that the computation of the tensile resistance of two-sided bolted HSS end-plate connections follow the current DG 24 design method for four-sided bolted RHS end-plate connections under axial tension loading, with the following changes:

- Parameter b' be computed using Equation 10.
- Parameter t_c be computed using Equation 16 (which employs F_{up}), with the resistance factor of $\phi = 0.90$.
- Parameter p be taken as equal to, or less than, the limit given by Equation 12.

Reliability analyses were also performed using the reliability assessment method given in AISI S100 (2020), as well as a recent reformulation of the separation factor approach. This paper illustrates how the MCS method can be used to evaluate simple closed-form methods for calculating resistance factors. For the particular connection database studied herein, it was found that:

- The AISI S100 reliability assessment method (Equations 17 to 20) can be used to compute resistance factors for non-lightweight materials when adjusted to consider typical structural steel loading ratios. The approach was able to compute an accurate yet conservative resistance factor when statistical parameters are taken from the literature for the material and fabrication factors and the target reliability indices given by Section B of the AISC 360 Commentary and ASCE 7 are used.
- Use of the default statistical parameters and target reliability indices in AISI S100 are poorly suited for the computation of resistance factors for structural steel members and connections.
- The traditional separation factor approach (SFA) for calculating the resistance factor (Equations 21 and 22 with $\alpha_R = 0.55$) produced a result close to the true (MCS) value. On the other hand, the modified SFA approach by Jollymore et al. (2025), using Equation 23 with $\alpha_R = 0.80$ in conjunction with Equation 21, was conservative.

EXAMPLE PROBLEM—DESIGN

Given:

Determine the design plate thickness required for a two-sided bolted HSS end-plate connection to resist a dead load $P_D = 20$ kips and a live load $P_L = 65$ kips in axial tension, using ASTM A500 Grade C HSS6×4× $\frac{5}{16}$ and six $\frac{3}{4}$ -in.-diameter ASTM F3125 Grade A325 bolts. The end plate is ASTM A572 Grade 50 with $w_p = 10$ in., $a = 1.5$ in., and $b = 1.5$ in., as shown in Figure 10.

LRFD Solution:

From AISC *Manual* Table 1-11, the geometric properties for HSS6×4× $\frac{5}{16}$ are:

$$H = 6 \text{ in.}$$

$$t = 0.291 \text{ in.}$$

From AISC *Manual* Table 2-5, the plate ultimate strength for ASTM A572 Grade 50 is:

$$F_{up} = 65 \text{ ksi}$$

Using load factors from ASCE 7 (2022), the required tensile strength is:

$$\begin{aligned} P_u &= 1.2P_D + 1.6P_L \\ &= 1.2(20 \text{ kips}) + 1.6(65 \text{ kips}) \\ &= 128 \text{ kips} \end{aligned}$$

The applied load per bolt, with no prying, can then be computed:

$$\begin{aligned} T_r &= \frac{128 \text{ kips}}{6 \text{ bolts}} \\ &= 21.3 \text{ kips/bolt} \end{aligned}$$

From AISC *Manual* Table 7-2, the available tensile strength per $\frac{3}{4}$ in. ASTM F3125 Grade A325 bolt is:

$$T_c = 29.8 \text{ kips/bolt}$$

Because $T_c \geq T_r$, the design can proceed. Calculate b' using Equation 10:

$$\begin{aligned} b' &= b + t \\ &= 1.5 \text{ in.} + 0.291 \text{ in.} \\ &= 1.79 \text{ in.} \end{aligned} \tag{10}$$

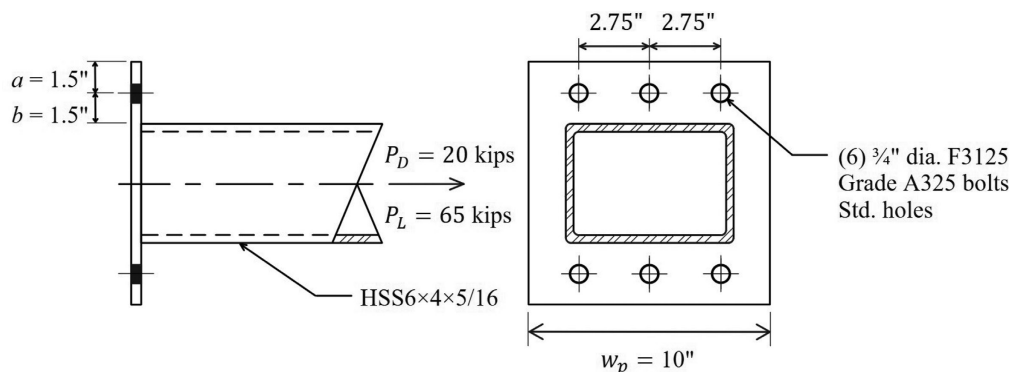


Fig. 10. Design example for a two-sided rectangular HSS end-plate connection.

Calculate the geometric parameters b' , p , ρ , d_h :

$$\begin{aligned} a' &= a \\ &= 1.5 \text{ in.} \end{aligned}$$

$$\begin{aligned} p &= \frac{w_p}{n_b/2} \\ &= \frac{10 \text{ in.}}{6/2} \\ &= 3.33 \text{ in.} \end{aligned}$$

$$\begin{aligned} \rho &= b'/a' \\ &= 1.79 \text{ in.}/1.5 \text{ in.} \\ &= 1.19 \end{aligned}$$

$$\begin{aligned} d_h &= d_b + 1/16 \text{ in.} \\ &= 3/4 \text{ in.} + 1/16 \text{ in.} \\ &= 13/16 \text{ in.} \end{aligned}$$

Check that $p \leq p_{max}$ using Equation 12:

$$\begin{aligned} p &\leq \frac{H + 4 \text{ in.}}{n_b/2} \\ 3.33 \text{ in.} &\leq \frac{6 \text{ in.} + 4 \text{ in.}}{6/2} \\ 3.33 \text{ in.} &\leq 3.33 \text{ in.} \end{aligned} \tag{12}$$

From AISC 360, Section J3.4, a User Note provides the preferred minimum bolt spacing as:

$$\begin{aligned} 3d_b &\leq 2.75 \text{ in.} \\ 2.25 \text{ in.} &\leq 2.75 \text{ in.} \end{aligned}$$

From AISC 360, Table J3.4, the minimum edge distance from hole center for a $3/4$ in. bolt is:

$$1 \text{ in.} \leq 1.5 \text{ in.}$$

Calculate δ :

$$\begin{aligned} \delta &= 1 - \frac{d_h}{p} \\ &= 1 - \frac{13/16 \text{ in.}}{3.33 \text{ in.}} \\ &= 0.756 \end{aligned}$$

Calculate t_c , the plate thickness to avoid prying, using Equation 16:

$$\begin{aligned} t_c &= \sqrt{\frac{4T_c b'}{\phi p F_{up}}} \\ &= \sqrt{\frac{(4)(29.8 \text{ kips})(1.79 \text{ in.})}{(0.90)(3.33 \text{ in.})(65 \text{ ksi})}} \\ &= 1.05 \text{ in.} \end{aligned} \tag{16}$$

Calculate β using Equation 3:

$$\begin{aligned}\beta &= \frac{1}{\rho} \left(\frac{T_c}{T_r} - 1 \right) \\ &= \frac{1}{1.19} \left(\frac{29.8 \text{ kips}}{21.3 \text{ kips}} - 1 \right) \\ &= 0.332\end{aligned}\quad (3)$$

Calculate α^* using Equation 4:

$$\begin{aligned}\alpha^* &= \min \left[\frac{1}{\delta} \left(\frac{\beta}{1-\beta} \right), 1 \right] \\ &= \min \left[\frac{1}{0.756} \left(\frac{0.332}{1-0.332} \right), 1 \right] \\ &= \min[0.659, 1] \\ &= 0.659\end{aligned}\quad (4)$$

Calculate t_p , the required plate thickness, using Equation 5:

$$\begin{aligned}t_p &= t_c \sqrt{\frac{T_r}{T_c} \left(\frac{1}{1 + \delta \alpha^*} \right)} \\ &= (1.05 \text{ in.}) \sqrt{\frac{21.3 \text{ kips}}{29.8 \text{ kips}} \left[\frac{1}{1 + (0.756)(0.659)} \right]} \\ &= 0.724 \text{ in.}\end{aligned}\quad (5)$$

Use a $\frac{3}{4}$ in. ASTM A572 Grade 50 end plate.

EXAMPLE PROBLEM—ANALYSIS

Given:

Find the available tensile strength of the connection presented in the previous example, with an end-plate thickness of $\frac{3}{4}$ in.

Solution:

Calculate α' using Equation 6:

$$\begin{aligned}\alpha' &= \frac{1}{\delta(1+\rho)} \left[\left(\frac{t_c}{t_p} \right)^2 - 1 \right] \\ &= \frac{1}{0.756(1+1.19)} \left[\left(\frac{1.05 \text{ in.}}{0.75 \text{ in.}} \right)^2 - 1 \right] \\ &= 0.572\end{aligned}\quad (6)$$

Calculate Q using Equation 7, for $0 < \alpha' \leq 1$:

$$\begin{aligned}Q &= \left(\frac{t_p}{t_c} \right)^2 (1 + \delta \alpha') \\ &= \left(\frac{0.75 \text{ in.}}{1.05 \text{ in.}} \right)^2 [1 + (0.756)(0.572)] \\ &= 0.735\end{aligned}\quad (7)$$

Calculate $T_{c,adj}$ using Equation 8:

$$\begin{aligned} T_{c,adj} &= T_c Q \\ &= (29.8 \text{ kips})(0.735) \\ &= 21.9 \text{ kips} \end{aligned} \quad (8)$$

Calculate T_a , the connection available tensile strength, using Equation 9:

$$\begin{aligned} T_a &= T_{c,adj} n_b \\ &= (21.9 \text{ kips})(6) \\ &= 132 \text{ kips} \end{aligned} \quad (9)$$

Because $T_a = 132 \text{ kips} > P_u = 128 \text{ kips}$, the design is satisfactory.

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SYMBOLS

B_D	Bias factor of dead load
B_L	Bias factor of live load
C_R	Load ratio correction factor
C_ϕ	Calibration coefficient
D	Dead load effect
F_m	Mean value of fabrication factor
F_{ub}	Bolt ultimate stress (ksi)
F_{up}	Plate ultimate stress (ksi)
F_{yp}	Plate yield stress (ksi)
G	Safety margin
$\overline{G}$	Mean value of the safety margin
H	HSS dimension parallel to bolt lines (in.)
L	Live load effect
M_m	Mean value of material factor
P_D	Tensile dead load (kips)
P_L	Tensile live load (kips)
P_m	Mean value of professional factor
P_u	Required tensile strength (kips)
Q	Prying adjustment factor

R	Resistance
R_n	Nominal resistance
S	Load effect
$S_{n,i}$	Nominal load effect
T_a	Available tensile strength of connection (kips)
T_c	Available bolt tensile strength (kips)
$T_{c,adj}$	Available bolt tensile strength, adjusted for prying effects (kips)
T_{nu}	Predicted tensile strength of connection using the plate ultimate stress (kips)
T_{nx}	Measured tensile strength of connection from experimentation (kips)
T_{ny}	Predicted tensile strength of connection using the plate yield stress (kips)
T_r	Applied load per bolt, with no prying (kips)
V_D	Coefficient of variation of dead load
V_F	Coefficient of variation of fabrication factor
V_L	Coefficient of variation of live load
V_M	Coefficient of variation of material factor
V_P	Coefficient of variation of professional factor
V_R	Coefficient of variation of resistance
V_Q	Coefficient of variation of load effect
a	Plate edge to bolt centerline distance (in.)
a'	Plate edge to sagging yield line distance (in.)
b	Bolt centerline to HSS face distance (in.)
b'	Bolt centerline to hogging yield line distance (in.)
d_b	Bolt shank diameter (in.)

d_h	Bolt hole diameter (in.)
n	Number of specimens
n_b	Number of bolts
p	Bolt tributary width (in.), probability
p_F	Probability of failure
p_{max}	Maximum allowable bolt tributary width (in.)
t	HSS thickness (in.)
t_c	End-plate thickness to avoid prying (in.)
t_p	End-plate thickness (in.)
w_p	End-plate width (in.)
α^*	Flexibility factor
α'	Design control factor
α_R	Separation factor
β	Prying ratio
β^+	Reliability index
γ_D	Dead load factor
γ_i	Constituent load factor
γ_L	Live load factor
δ	Prying effect factor
ρ	Yield line ratio
ρ_R	Bias coefficient of resistance
σ_G	Standard deviation of the safety margin
ϕ	Resistance factor

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Shear Lag Effects in Tension Members with Short Longitudinal Welds

JACKSON T. MUNCY and MARK D. DENAVIT

ABSTRACT

Shear lag can reduce the strength of tension members and must be considered when calculating tensile rupture strength—for example, by using the shear lag factor. The shear lag factor defined in Case 4 of AISC *Specification* Table D3.1 (2022) applies to tension members where the load is transmitted by longitudinal welds only. This shear lag factor was developed through analysis of physical experiments, but predominantly with experiments on specimens that failed by weld rupture, potentially leading to unnecessarily low values of the shear lag factor. As a result, the behavior of tension members with short longitudinal welds, for which the shear lag factor can be much less than unity, is uncertain. In this work, previously published experiments are reevaluated, and new experiments are reported. The results shed light on the behavior of tension members with short longitudinal welds. Despite current design provisions indicating that tensile rupture is an important limit state for tension members with short welds, no evidence that tensile rupture or shear lag influences strength of such members was identified. For plate tension members, use of a shear lag factor of $U = 1$ was found to be acceptable because other applicable limits states that better represent the physical behavior control. These results show the need for further research to better characterize the behavior of welded tension members and to simplify design provisions.

Keywords: tension members, welded connection, tensile rupture, shear lag factor.

INTRODUCTION

The nonuniform stress distribution that occurs in a tension member adjacent to a connection, especially when not all elements of the cross section are directly connected, is commonly referred to as the shear lag effect (Easterling and Gonzalez Giroux, 1993). Shear lag effects can decrease strength, and accurate accounting of them in design ensures safety and efficiency. For design of tension members according to the AISC *Specification* (2022), the available strength for the limit state of tensile rupture in the net section depends on the shear lag factor, U , which is determined according to AISC *Specification* Table D3.1. This table lists several cases. Case 4 applies to plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. The shear lag factor for Case 4 is defined as:

$$U = \frac{3l^2}{3l^2 + w^2} \left(1 - \frac{\bar{x}}{l} \right) \quad (1)$$

where l is the length of the connection, w is the spacing between welds, and $\bar{x}$ is the eccentricity of the connection. Figure 1 shows dimensions l and w . A footnote to AISC *Specification* Table D3.1 defines l as $(l_1 + l_2)/2$ and specifies that l_1 and l_2 shall not be less than 4 times the weld size.

Case 4 uses the shear lag factor from Case 2 (i.e., $U = 1 - \bar{x}/l$) as a base but modifies it to account for the length of the longitudinal welds, l , and the width between them, w . When the length of the welds is short relative to the width between the welds, the resulting shear lag factor can be very low. Note, however, that AWS D1.1 (2020), clause 4.9.2, states “If longitudinal fillet welds are used alone in lap joints of end connections of flat bar or plate members, the length of each fillet weld shall be no less than the perpendicular distance between them”—thus effectively limiting the ratio between l and w .

The current form of the shear lag factor for Case 4 (i.e., Equation 1) was introduced in the 2016 edition of the AISC *Specification* (2016) and is based on research by Fortney and Thornton (2012). Prior to this study, Case 4 applied only to plates, and the shear lag factor was defined as $U = 1.0$ when $l/w \geq 2$, $U = 0.87$ when $2 > l/w \geq 1.5$, and $U = 0.75$ when $1.5 > l/w \geq 1.0$. The shear lag factor was undefined when $l/w < 1$, implying that the weld length should not be less than the plate width. Fortney and Thornton proposed a shear lag factor applicable to all member types, cases with unequal weld lengths, and cases with weld lengths less than the weld spacing (i.e., $l/w < 1$). Fortney and Thornton developed their recommendations based on comparisons to experimental results. Three studies were included in their evaluation (Easterling and Gonzalez Giroux, 1993; Gibson

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and Wake, 1942; SSWC, 1931). Of the 175 specimens evaluated by Fortney and Thornton, 151 were reported by the Structural Steel Welding Committee of the American Bureau of Welding (SSWC, 1931), and 149 of these specimens failed by weld rupture and not tensile rupture. Fortney and Thornton asserted that data from these specimens are valid for evaluating shear lag effects.

To illustrate the pertinent strength calculations, consider the plate tension member with a welded connection shown in Figure 2. Five limit states must be considered to determine the available strength of the tension member: tensile yielding in the gross section, tensile rupture in the net section, shear yielding adjacent to the welds, shear rupture adjacent to the welds, and weld rupture. The shear yielding and shear rupture limit states can be considered as the base metal check for the weld. The strength for each of these limit states, except for gross section tensile yielding, depends on the length of the weld. Strength equations are shown in Table 1, where t is the thickness of the plate and t_w is the throat of the weld. The design strength (i.e., with LRFD resistance factors applied) of the tension member for

each limit state is shown as a function of weld length, l , in Figure 3.

Interestingly, the weld strength and tensile rupture strength are nearly equal for weld lengths less than 6 in. (i.e., the width of the plate). This may be a consequence of the shear lag factor being derived predominantly from experiments where weld failure occurred. It is questionable that tension members with these very small weld lengths would experience a net section tensile rupture failure, one where the strength is proportional to the cross-sectional area of the tension member, instead of a weld or base metal failure. If the weld and base metal checks are sufficient to capture the strength of members with short welds, including potential shear lag effects, then using the shear lag factor, U , to also capture the strength of members with short welds as part of the tensile rupture limit state is redundant and not representative of physical behavior.

The objective of this work is to shed new light on the strength of tension members connected with short longitudinal welds. Assumptions made in prior research are reevaluated by examining the results of previously published

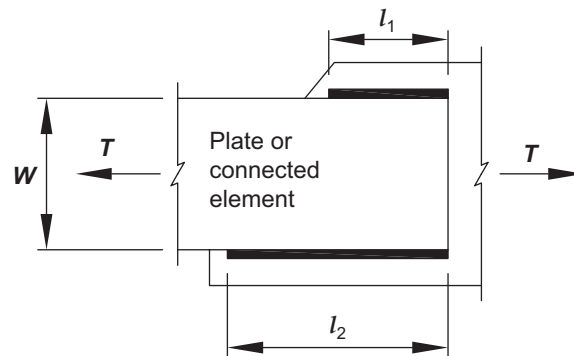


Fig. 1. Diagram for Case 4 in AISC Specification Table D3.1 (2022).

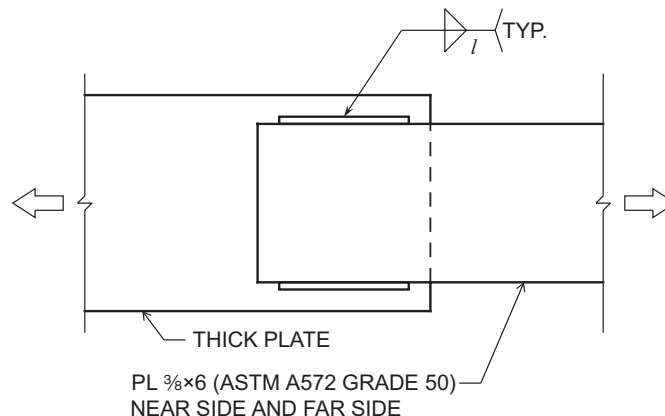


Fig. 2. Double-plate tension member for example calculations.

Table 1. Limit States and Strength Equations for Longitudinally Welded Double Plate Tension Member		
Limit State and AISC Specification Reference	Nominal Strength Equations	Resistance Factor and Safety Factor
Tensile yielding Section D2	$P_{n,TY} = F_y A_g$ $A_g = 2wt$	$\phi_t = 0.90$ $\Omega_t = 1.67$
Tensile rupture Section D2	$P_{n,TR} = F_u A_e = F_u A_n U$ $A_n = 2wt$ See Equation 1 for U	$\phi_t = 0.75$ $\Omega_t = 2.00$
Shear yielding Section J4.2	$P_{n,SY} = 0.60 F_y A_{gv}$ $A_{gv} = 4lt$	$\phi = 1.00$ $\Omega = 1.50$
Shear rupture Section J4.2	$P_{n,SR} = 0.60 F_u A_{nv}$ $A_{nv} = 4lt$	$\phi = 0.75$ $\Omega = 2.00$
Weld rupture Section J2.4	$P_{n,WR} = F_{nw} A_{we} = 0.60 F_{EXX} A_{we}$ $A_{we} = 4lt_w$	$\phi = 0.75$ $\Omega = 2.00$

physical experiments as well as new physical experiments described in this work. Comparisons are made to current design provisions and recommendations are made for further work to improve the tensile rupture provisions in the AISC Specification.

PREVIOUS EXPERIMENTAL RESULTS

A database of previously published physical experiments on longitudinally welded tension members was compiled to form a benchmark against which design equations can

be compared. The data was taken from five prior studies (SSWC, 1931; Gibson and Wake, 1942; Easterling and Gonzalez Giroux, 1993; Mannem, 2002; Orloff et al., 2020). The members tested in these experimental studies include single and double plates, single and double angles, and double channels. Only specimens with two parallel longitudinal welds were included in the database. Specimens with a transverse weld or only one longitudinal weld were excluded. Specimens that did not achieve their full strength (e.g., machine capacity limits were reached) were also excluded from the database. Altogether, the database developed for

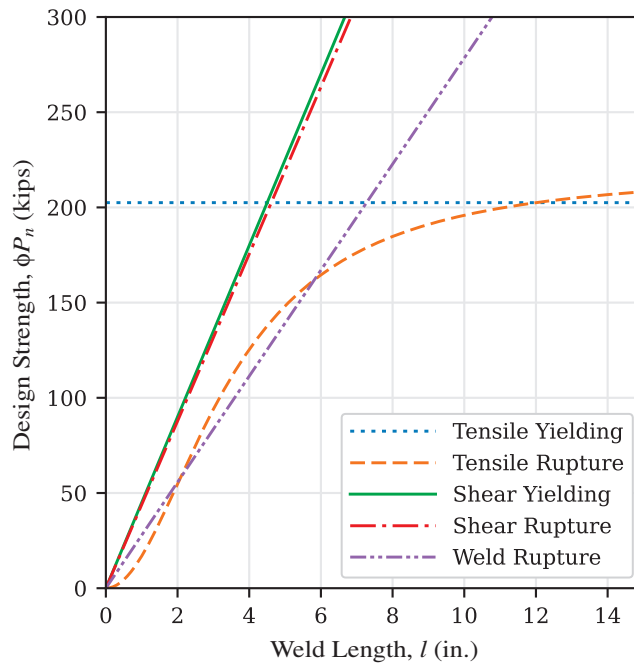


Fig. 3. Design strength of example double-plate tension member as a function of weld length.

Table 2. List of Source References for the Database and Count of Specimens in the Database by Experimental Failure Type			
	Experimental Failure Type		
Reference	Member	Weld	Total
SSWC (1931)	9	289	298
Gibson and Wake (1942)	3	7	10
Easterling and Gonzalez Giroux (1993)	12	0	12
Mannem (2002)	23	3	26
Orloff et al. (2020)	8	12	20
Total	55	311	366

this work includes 366 previously tested specimens as listed in Table 2 by reference and failure type. Most of the specimens failed by weld rupture, while 55 were reported to have failed in the member. The database is published in a publicly available data repository (Muncy and Denavit, 2025).

The Structural Steel Welding Committee (SSWC) of the American Bureau of Welding was organized to obtain “reliable information upon which to base safe unit working stresses in the designing of welded structures.” The report by the committee (SSWC, 1931) describes more than 2,000 physical experiments, 298 of which are included in the database (151 single-plate tension members and 147 double-plate tension members). The SSWC report was also the source of most of the data evaluated by Fortney and Thornton (2012); however, Fortney and Thornton only evaluated data from the 151 single-plate tension member experiments. Given the purpose of the testing, the vast majority of specimens reported by the SSWC failed by weld rupture. Only 9 of the 298 specimens reported by SSWC and included in the database developed for this work experienced a member failure.

Gibson and Wake (1942) investigated the strength of welded angle tension members with a primary goal of determining if unbalanced weld arrangements are as effective as balanced weld arrangements (i.e., those for which the center of resistance of the welds aligns with the center of gravity of the angle). Gibson and Wake tested 54 specimens in 15 different arrangements, most of which failed by weld rupture. Six single-angle arrangements and four double-angle specimens are included in the database. Gibson and Wake tested three specimens of each single-angle arrangement and reported only the average, which was taken as a single specimen in the database. Fortney and Thornton (2012) included only the four single-angle arrangements with balanced welds in their evaluation.

Easterling and Gonzalez Giroux (1993) investigated shear lag effects in welded tension members to inform updates to the AISC *Specification*. They tested 27 tension members, including double-plate, double-angle, and double-channel specimens, 12 of which were included in the database.

Fortney and Thornton (2012) included these specimens in their evaluation plus four additional double-tee specimens reported in Gonzalez Giroux’s master’s thesis (Gonzalez, 1989).

The remaining two studies were not evaluated by Fortney and Thornton (2012). Mannem (2002) and Orloff et al. (2020) both investigated shear lag effects in welded steel angles and plates. Mannem tested 49 specimens, 26 of which are included in the database, while Orloff et al. tested 20 specimens, all of which are included in the database.

The database includes recorded data such as weld dimensions and experimental strength as well as calculated values analyzing the specimen’s strength compared to AISC *Specification* equations. The following fields are included in the database:

- Reference (i.e., author and year of publication)
- Specimen name
- Specimen member type
- Weld length, l (in.)
- Weld spacing, w (in.)
- Net area, A_n (in.²)
- Connection eccentricity, $\bar{x}$ (in.)
- Measured yield stress, F_y , and tensile strength, F_u , of tension member (ksi)
- Nominal weld strength, F_{EXX} (ksi)
- Weld throat dimension, t_w (in.)
- Experimental failure type (i.e., member or weld)
- Experimental strength, P_{exp} (kips)
- Plate thickness, t (in.) for plate specimens only

The net area for double component specimens (i.e., double-plate, double-channel, or double-angle) was taken as the combined net area for both shapes. Likewise, the

experimental strength of double-component specimens was taken as the total force in both components at failure.

The data encompasses a range of ratios of weld length to weld spacing (i.e., l/w) and connection eccentricity to weld length (i.e., $\bar{x}/l$). These parameters are important because they define the shear lag factor in Equation 1. Figure 4 shows the range of these ratios for specimens in the database. The specimens are separated into two subfigures by experimental failure type. Each marker represents a single specimen. The shape and color of the marker indicates the member type. The connection eccentricity for all plates was assumed to be zero as is typical for design (Fortney and Thornton, 2012). Apart from the plate tension members, the eccentricity ratio, $\bar{x}/l$, ranges from 0.069 to 0.465. The length to width ratios ranges from 0.667 to almost 4.0. Specimens with lower values of $\bar{x}/l$ or higher values of l/w tended to fail in the member, while specimens with higher values of $\bar{x}/l$ or lower values of l/w tended to fail in the weld. Specimens with the highest values of l/w tend to have relatively low values of $\bar{x}/l$ and vice versa (i.e., few specimens were tested with high l/w and high $\bar{x}/l$).

Most of the specimens exhibited significant member yielding prior to failure. This can be seen in the load-deformation response of the specimens for which that data was reported and also by comparing the experimental strength to the tensile yielding strength, $A_g F_y$ (using measured values). The ratio of experimental strength to tensile yielding strength is shown in Figure 5. Only one specimen reported to have failed in the member, 2443-Q from SSWC (1931), failed at a load less than $A_g F_y$. Many specimens reported to have failed in the weld had a peak load less than $A_g F_y$, but most had a peak load above this limit. These results indicate that even when rupture and a sudden loss of load was observed in the experiment, it was often preceded by significant yielding.

The strength of the specimens according to the 2022 AISC *Specification* (AISC, 2022) is compared to the experimental strength in Figure 6. Specimens reported to have failed in the member are compared to the tensile rupture strength, while specimens reported to have failed in the weld are compared to the weld rupture strength. Tensile rupture strength was computed using measured material properties, while weld rupture strength was calculated using nominal material properties (due to a lack of reporting on measured weld material properties). Both were computed without resistance or safety factors.

The average test-to-predicted ratio in Figure 6(a) is above unity. The four specimens shown in Figure 6(a) that have a test-to-predicted ratio less than 0.95 were reported by the SSWC (1931). For three of the specimens (i.e., 2263-Q, 2463-C, and 2463-Q), the failure occurred above the yield strength, $F_y A_g$, while the fourth specimen (i.e., 2443-Q) failed at 89% of the yield strength. Overall,

the test-to-predicted ratio tends to increase as the weld length-to-width ratio decreases, particularly when l/w is less than 2.0. Test-to-predicted ratios greater than unity mean that the specimen exceeded the expected strength calculated according to the AISC *Specification*. Significantly surpassing this expected strength could imply either the tensile rupture equation or the shear lag factor equation underestimates the strength of the specimen. Since the specimens with higher length-to-width ratios have relatively accurate predictions, it is likely that the shear lag factor is the cause of the underestimated strength for the specimens with a lower length-to-width ratio. The shear lag factor equation (i.e., Equation 1) for these specimens is strongly dependent on the weld length-to-width ratio. These results could indicate that the shear lag factor is overly sensitive to the weld length-to-width ratio.

The average test-to-predicted ratio in Figure 6(b) is also above unity, but many specimens have a test-to-predicted ratio less than unity. Notably, two single-angle specimens with $l/w = 2.5$ have a low test-to-predicted ratio of 0.36 [the markers for these two specimens overlap in Figure 6(b)]. These two specimens, UEA4 and UEA7, experienced “failures through the weld by pull out at the outer edge on the heel of the angle” (Mannem, 2002). Tearing at the heel of the angle at the net section was also observed. The weld failures occurred at a load 10% greater than $F_y A_g$ and the computed tensile rupture strength (both approximately 103 kips). Therefore, while weld rupture was the limit state causing loss of load-carrying capacity for these specimens, other damage occurred, and strength equations for limit states associated with the damage conservatively underpredict the experimental strength. The other specimens with test-to-predicted ratio less than unity in Figure 6(b) do not necessarily indicate that the current provisions are unsafe because the scatter is considered in reliability analyses performed to determine resistance and safety factors (e.g., Dowswell and Cox, 2024). However, the lack of a clear trend between l/w and the weld test-to-predicted ratio in Figure 6(b) indicates that shear lag is not a major factor in the strength of longitudinal welds in end connections.

The strength data shown versus the weld length-to-width ratio in Figure 6 is shown versus the eccentricity ratio in Figure 7. There is an upward trend in Figure 7(a) where the test-to-predicted ratio for double-angle specimens increases with increasing eccentricity ratio. However, the trend is not observed for single-angle specimens. The two single-angle specimens with low weld test-to-predicted ratios discussed previously are seen again in Figure 7(b), but no other obvious trends are identified from Figure 7(b).

Overall, the previous experimental results indicate that tensile rupture strength equations in the AISC *Specification* (2022) underestimate the true strength of longitudinally welded tension members, likely due to a lower

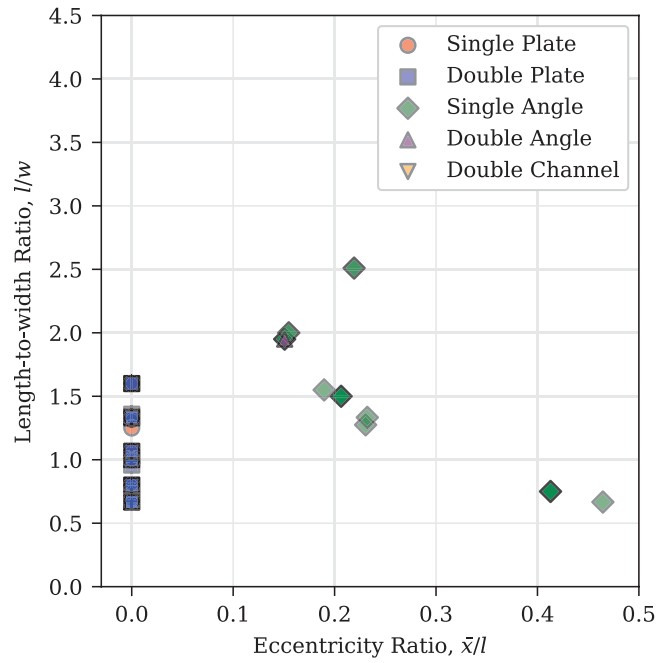
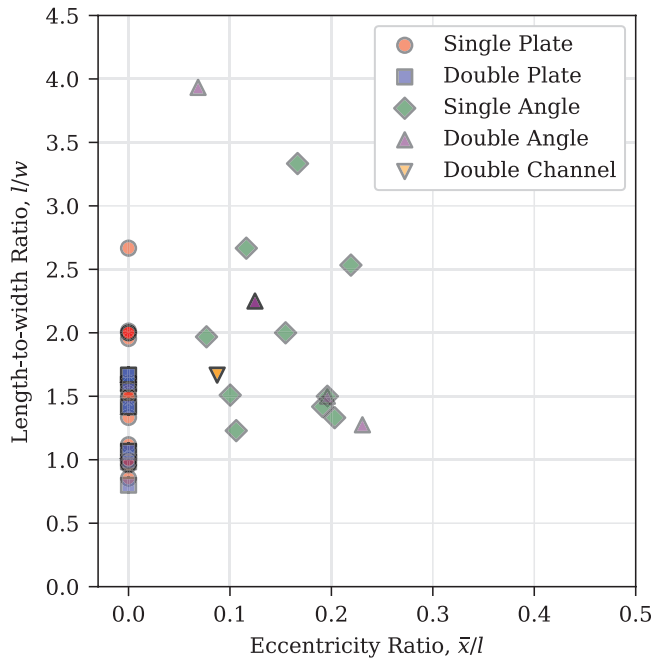


Fig. 4. Length-to-width ratio versus eccentricity ratio for specimens in the database.

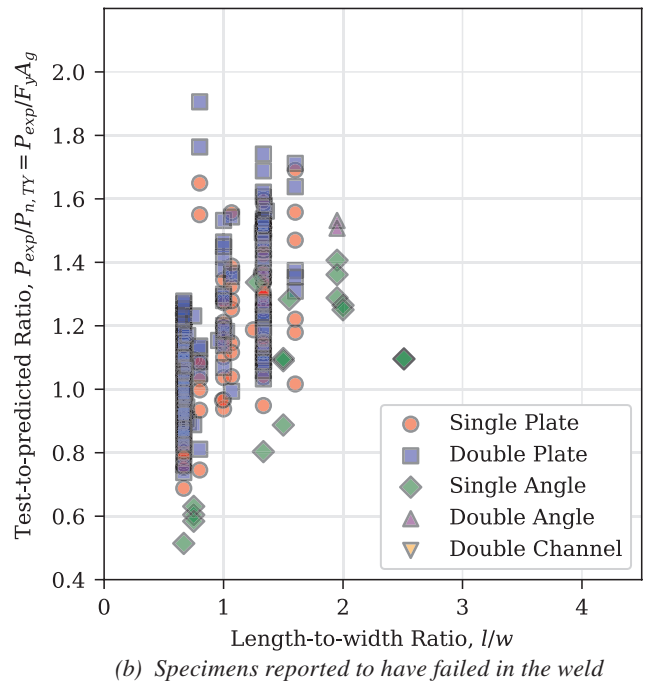
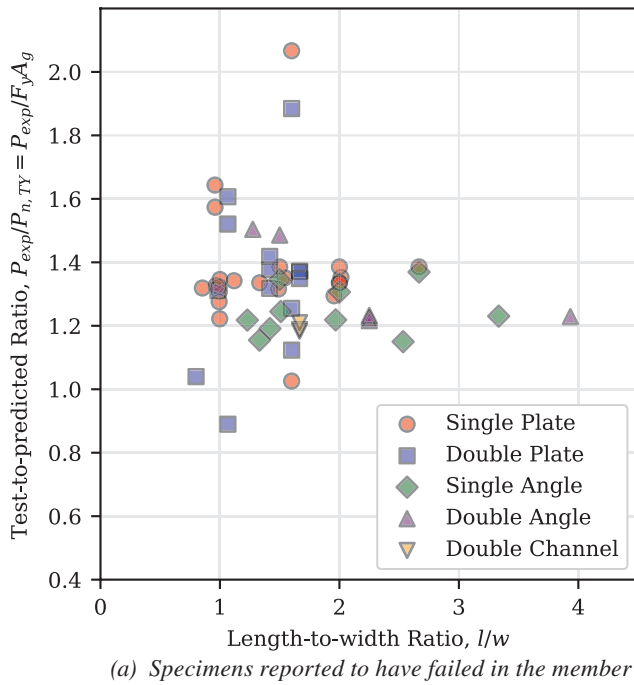
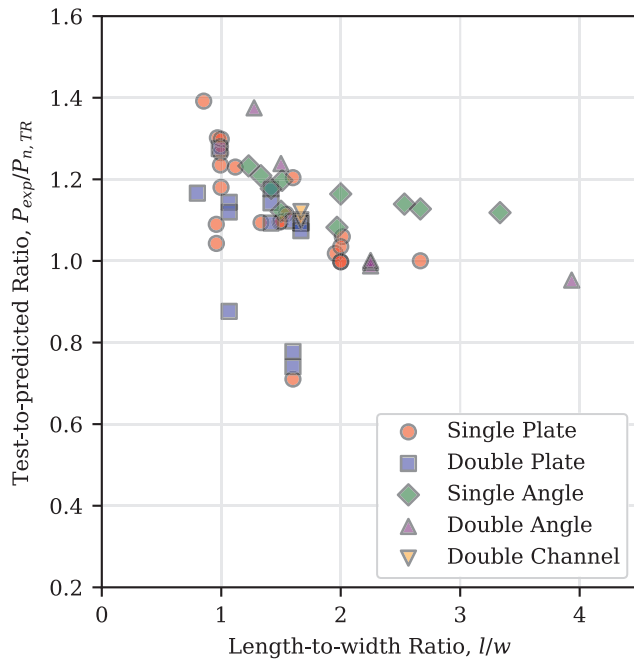
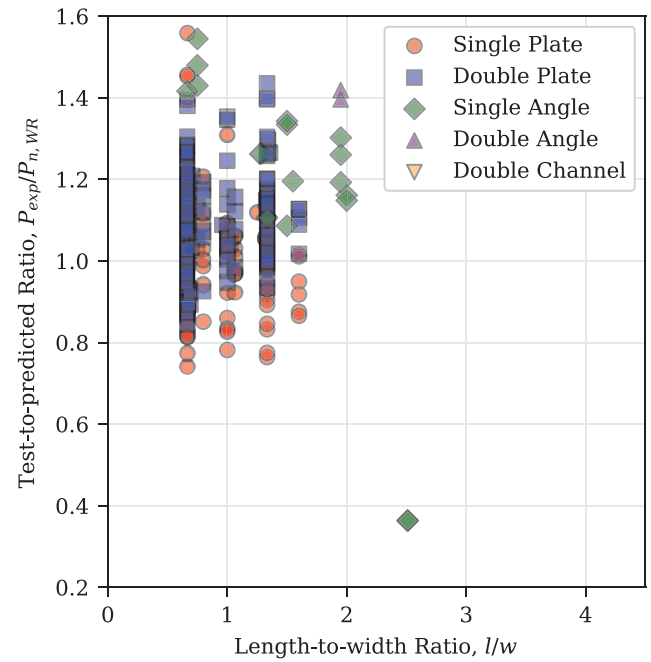


Fig. 5. Comparison of experimental strength to calculated yield strength for specimens in the database.

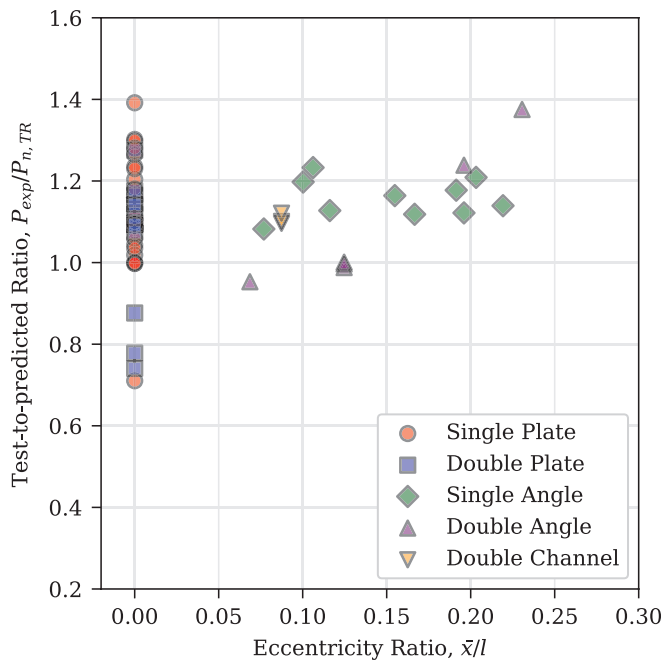


(a) Specimens reported to have failed in the member

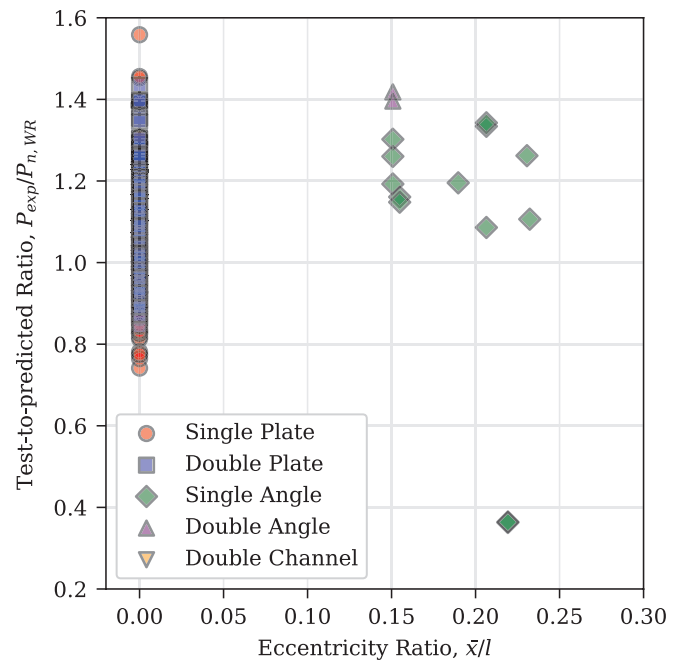


(b) Specimens reported to have failed in the weld

Fig. 6. Comparison of experimental strength to calculated strength shown versus length-to-width ratio for specimens in the database.



(a) Specimens reported to have failed in the member



(b) Specimens reported to have failed in the weld

Fig. 7. Comparison of experimental strength to calculated strength shown versus eccentricity ratio for specimens in the database.

than necessary shear lag factor. They also indicate that if shear lag effects reduce weld strength, then the reduction is minor. However, the data is limited, especially for tension members with very low weld length-to-width ratios for which shear lag effects are expected to be the most extreme. Therefore, additional testing, especially on specimens with weld length-to-width ratios less than 0.667, would be beneficial for better understanding the behavior of longitudinally welded tension members.

MATERIALS AND METHODS

Eight double-plate tension member specimens were designed, fabricated, and tested to failure to quantify their strength. A drawing of one of the specimens is shown in Figure 8. For each specimen, two $\frac{3}{8}$ -in.-thick steel test plates were welded to two $\frac{3}{4}$ -in.-thick steel support plates. Each support plate was bolted with two $\frac{7}{8}$ -in.-diameter

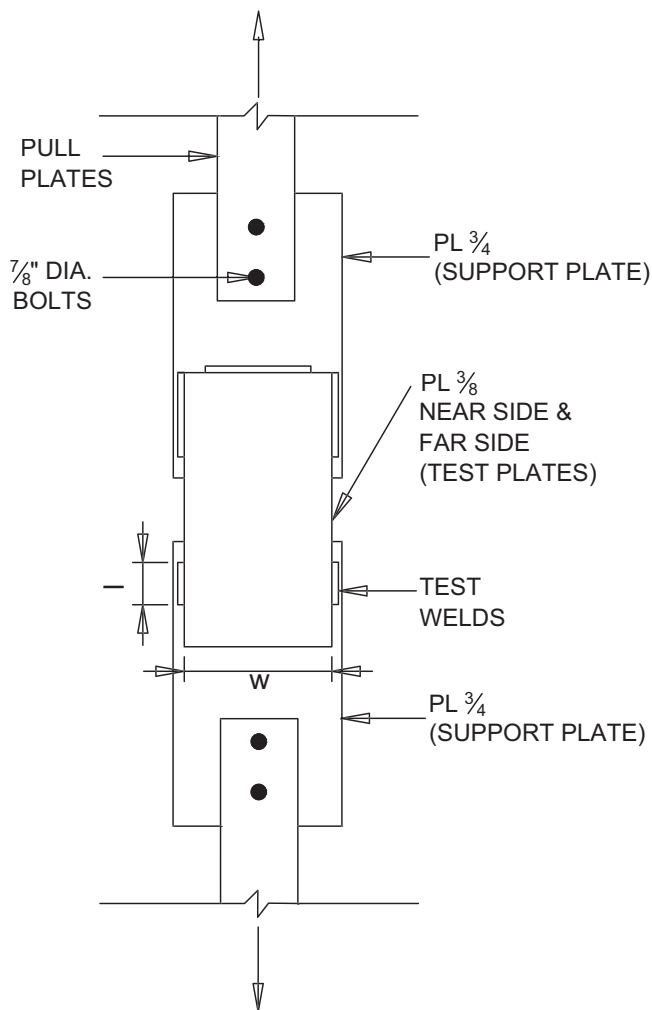


Fig. 8. Schematic of test specimen.

bolts to a pair of pull plates that were connected to a uni-axial test machine and applied tension to the specimen. A photograph of one of the specimens in the test machine is shown in Figure 9.

The test plates were all 13 in. long and $\frac{3}{8}$ in. thick but varied in width. Specimen names and dimensions are listed in Table 3. The first number in each specimen name indicates the width of the plate. The second number in each specimen name indicates the nominal length of the weld. All test plates were cut from a single piece of $\frac{3}{8}$ -in.-thick by 8-in.-wide by 20-ft-long steel, using a waterjet cutter. Three tensile coupons were also cut from the piece of steel and tested according to ASTM E8 (2022). The average yield stress was measured to be $F_y = 54.4$ ksi, the average ultimate strength was measured to be $F_u = 73.0$ ksi, and the average elongation was measured to be 26% (for an 8 in. gauge length). The thickness of the test plates was measured to be 0.372 in.

The welds were specified as $\frac{1}{4}$ in. and made in a single pass using a MIG process with ER70S-6 wire ($F_{EXX} = 70$ ksi). The weld lengths (tip-to-tip) and weld legs were

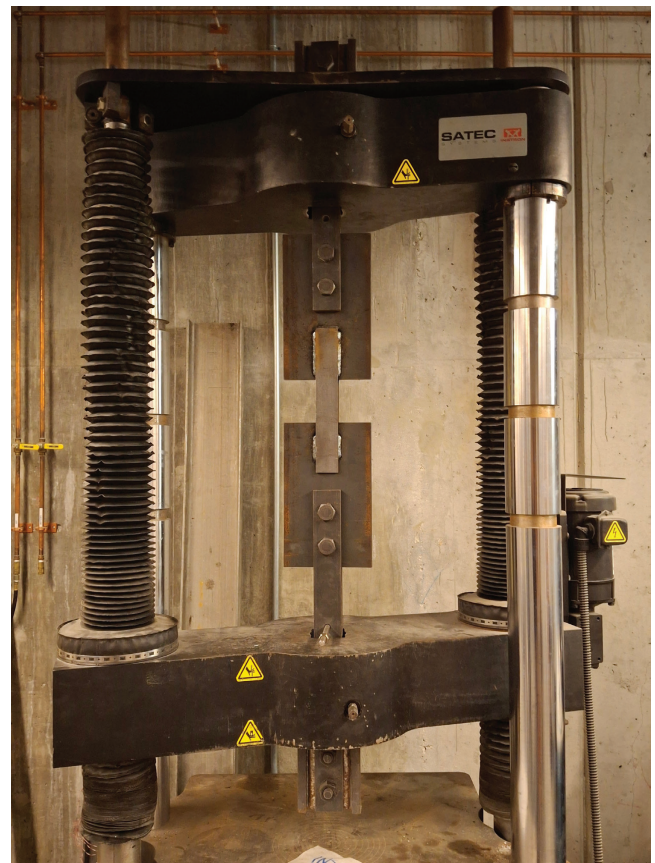


Fig. 9. Photograph of Specimen S22 in the testing machine prior to loading.

Table 3. Test Matrix								
Specimen —	t in.	w in.	l in.	t_w in.	l/w —	U —	$A_g = A_n$ in. ²	$A_{gv} = A_{nv}$ in. ²
S22	0.372	2	2.26	0.202	1.130	0.793	1.49	3.36
S23	0.372	2	3.22	0.188	1.610	0.886	1.49	4.79
S24	0.372	2	4.08	0.191	2.040	0.926	1.49	6.07
S32	0.372	3	2.20	0.191	0.733	0.617	2.23	3.27
S42	0.372	4	2.15	0.202	0.538	0.464	2.98	3.20
S52	0.372	5	2.24	0.205	0.448	0.376	3.72	3.33
S62	0.372	6	2.26	0.202	0.377	0.299	4.46	3.36
S72	0.372	7	2.35	0.207	0.336	0.253	5.21	3.50

measured using a caliper. The throat, t_w , was computed from the weld leg measurements as shown in Figure 10. Each specimen had four lines of test welds. The average of the four weld lengths and average of the four throats are reported in Table 3. Using measured dimensions, the specimens had weld length-to-width ratios ranging from 0.336 to 2.040.

Deformations of the specimens were measured using LED markers attached to the plates. Two markers were attached to the top support plate, and two markers were attached to the bottom support plate. An Optotrak position measurement machine recorded the three-dimensional position of the markers throughout the test. Connection deformation (i.e., elongation) was calculated from the position data after the test. Specimens were loaded in displacement control at a rate of the crosshead of 0.05 in./min or 0.10 in./min for specimen S24. Testing was stopped after weld rupture or tensile rupture occurred and the load-carrying capacity of the specimen dropped. Specimens S23 and S24 were stopped after necking initiated and the peak load had been achieved but prior to rupture.

RESULTS

A summary of the test results is presented in Table 4. Two failure modes were observed: tensile rupture of the test plates for specimens S22, S23, and S24 and weld rupture for all other specimens. Load-deformation results of all eight specimens are shown in Figure 11.

The specimens that failed by tensile rupture (i.e., S22, S23, and S24) exhibited an initially stiff response, followed by yielding with a plateau, then strain hardening. Necking occurred away from the weld at approximately mid-distance between the two support plates as shown in Figure 12. Fracture of the two test plates occurred simultaneously for specimen S22. Testing of specimens S23 and S24 was halted after necking initiated but before fracture of the test plates. No other damage to the specimen or welds was identified.

The remaining five specimens (i.e., S32, S42, S52, S62, and S72) failed by weld rupture. A detailed view of the load-deformation response of these specimens is shown in Figure 11(b). The peak deformation was much less than the specimens that failed by tensile rupture. The specimens exhibited an initially stiff response, followed by

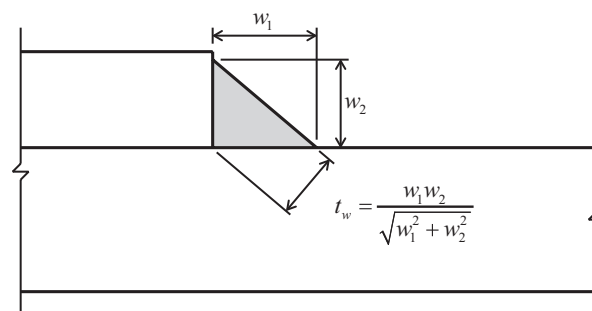


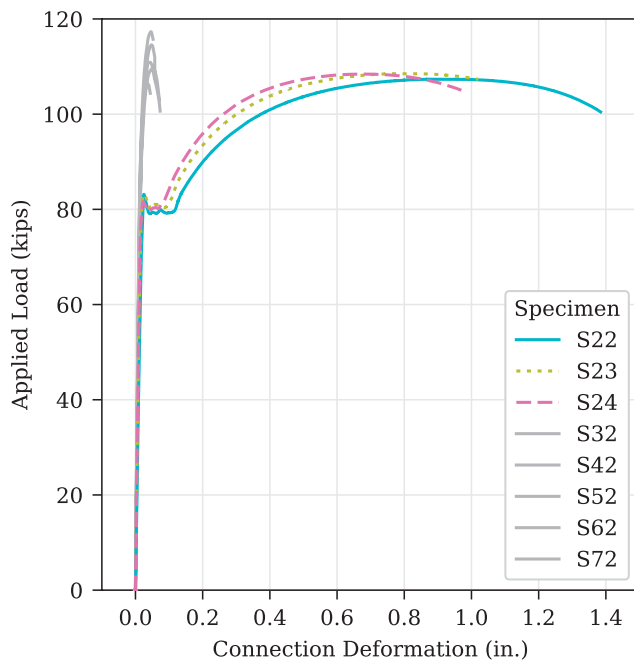
Fig. 10. Details of weld measurements.

Table 4. Experimental Results			
Specimen —	Peak Load, P_{exp} kips	Failure Mode —	$U_{exp} = P_{exp}/F_u A_n$ —
S22	107.3	Plate rupture	0.988
S23	108.2	Plate rupture	0.996
S24	108.2	Plate rupture	0.996
S32	114.3	Weld rupture	0.702
S42	110.6	Weld rupture	0.509
S52	109.1	Weld rupture	0.402
S62	116.9	Weld rupture	0.359
S72	106.3	Weld rupture	0.279

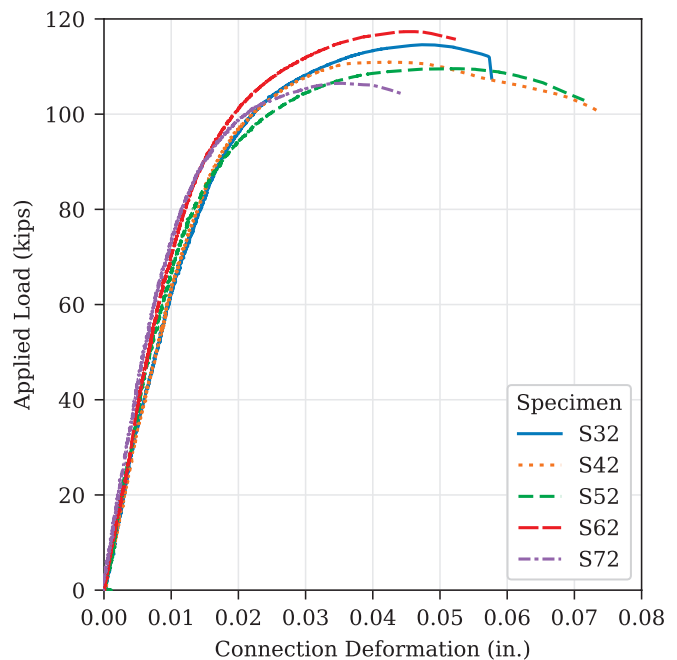
gradual softening, eventually reaching a peak load. After the peak load, the load dropped gradually until weld rupture occurred (as shown in Figures 13 and 14) and the connection lost load-carrying capacity. Failure of the welds occurred simultaneously and was generally through the throat at around 45°. Signs of minor shear yielding of the test plates in some locations adjacent to the weld were observed as shown in Figure 15, but no other damage to the test plates was observed.

DISCUSSION

Table 5 lists strengths for the specimens calculated according to the AISC *Specification* (2022) for each of the five limit states applicable to the test plates and welds: tensile yield (TY), tensile rupture (TR), shear yield (SY), shear rupture (SR), and weld rupture (WR). Equations for these limit states are shown in Table 1. For the strengths listed in Table 5, no resistance or safety factors were applied, measured values of F_y and F_u of the plate were used, measured



(a) All specimens



(b) Specimens that failed by weld rupture

Fig. 11. Measured load-deformation response.



Fig. 12. Photograph of specimens that failed in the plate.



Fig. 13. Photograph of specimens that failed in the weld.



Fig. 14. Photograph of weld failure for specimen S42.

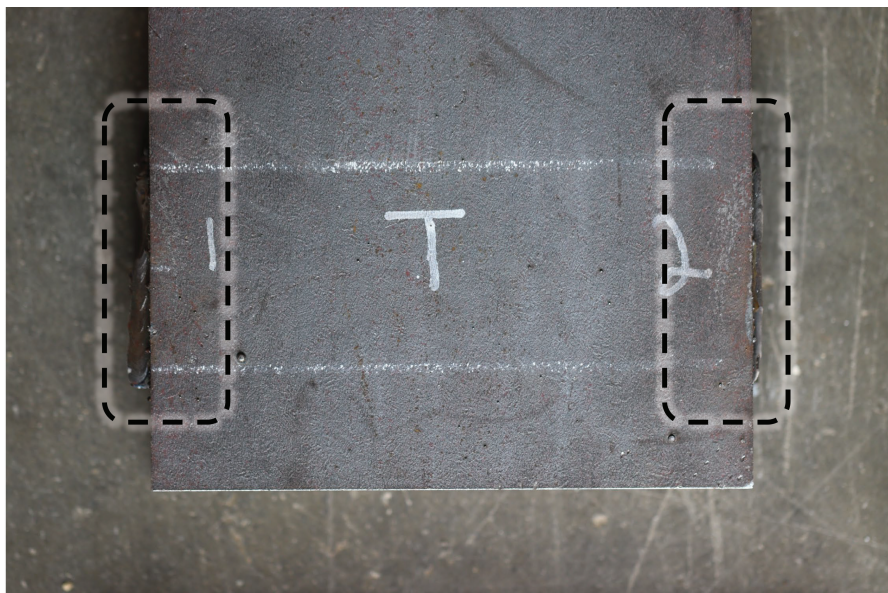


Fig. 15. Photograph of specimen S62 after testing with indications of shear yielding near the welds.

Table 5. Calculated Strengths for the Test Specimens

Specimen —	$P_{n,TY}$ kips	$P_{n,TR}$ kips	$P_{n,TR(U=1)}$ kips	$P_{n,SY}$ kips	$P_{n,SR}$ kips	$P_{n,WR}$ kips	P_n kips	$P_{exp}/P_{n,TR}$ —	$P_{exp}/P_{n,WR}$ —
S22	80.9	86.1	109	110	147	76.6	76.6	1.25	1.40
S23	80.9	96.2	109	156	210	102	80.9	1.12	1.06
S24	80.9	101	109	198	266	132	80.9	1.08	0.83
S32	121	101	163	107	143	70.7	70.7	1.14	1.62
S42	162	101	217	104	140	72.8	72.8	1.10	1.52
S52	202	102	272	109	146	77.1	77.1	1.07	1.42
S62	243	97.3	326	110	147	76.6	76.6	1.20	1.53
S72	283	96.1	380	114	153	81.8	81.8	1.11	1.30

plate and weld dimensions (as shown in Table 3) were used, and the nominal weld strength was used. The minimum calculated strength, P_n , is shown along with select test-to-predicted ratios.

According to the AISC *Specification* equations, the specimens were controlled by either weld rupture or tensile yielding with controlling strengths ranging from 70.7 to 81.8 kips. All eight specimens had an experimental strength higher than their respective controlling strength according to AISC *Specification* equations. The calculated tensile rupture strength ranged from 86.1 kips for specimen S22 to 102.1 kips for specimen S52. All eight specimens also surpassed their calculated tensile rupture strength. For five of the eight specimens the experimental strength exceeded or was within 3% of the calculated shear yielding strength.

For the three specimens that failed by tensile rupture (i.e., S22, S23, and S24), the experimental strengths were nearly identical. They were also close to the tensile rupture strength calculated using $U = 1$. The only difference in these three specimens are their respective weld lengths represented by the second number in their name. The ratio of experimental strength to calculated tensile rupture strength (i.e., $P_{exp}/P_{n,TR}$) decreases toward unity with increasing weld length. This is due to the shear lag factor computed using Equation 1 increasing toward unity. These results, and the fact that the rupture occurred away from the welds, indicate that there were minimal shear lag effects in these three failed specimens despite $U < 1$ according to the AISC *Specification*.

For the five specimens that failed by weld rupture, the true tensile strength of the plates is not known. However, since the plates did not fail, it can be concluded that the tensile rupture strength of these specimens is greater than the experimental strength. Furthermore, the experimental strength for these specimens is greater than the calculated tensile rupture strength. Therefore, the shear lag factor is conservative, but to an unknown extent.

It is common in previous research, including Fortney and Thornton (2012), to compute an experimental shear lag factor as the experimental strength divided by the product of the tensile strength and net area:

$$U_{exp} = \frac{P_{exp}}{F_u A_n} \quad (2)$$

The experimental shear lag factor can be computed for any specimen, including those not failing by tensile rupture. Fortney and Thornton (2012) argued that experimental shear lag factors computed for specimens failing by weld rupture were relevant to the shear lag factor for tensile rupture. However, they are relevant only as a lower bound of the true shear lag factor depending on how close the specimen was to tensile rupture failure when the weld ruptured.

Figure 16 shows that the experimental shear lag factor for the specimens tested in this study decreases significantly as the width between the welds increases (i.e., l/w decreases) and the trend is well captured by Equation 1. However, for the specimens that failed by weld rupture, there was no indication of distress in the net section that would indicate that tensile rupture was imminent. Furthermore, other limit states (e.g., weld rupture and shear yielding) limit the calculated strength such that the overall calculated strength of the specimen is conservative, even if $U = 1$.

The five specimens that failed by weld rupture failed at loads greater than calculated using measured weld dimensions and nominal weld material properties. The ratio of test strength to calculated weld strength is 1.46, which is substantial but not out of the ordinary for similarly calculated test-to-predicted ratios for longitudinally loaded welds (e.g., Dowswell and Cox, 2024). Interestingly, the specimens achieved roughly the same strength despite the distance between the welds being different. These five specimens all had weld lengths of 2 in. and had different

widths between welds, represented by the first number in their name. The lack of variation in strength indicates that a shear lag factor is not needed for weld failures. The lack of variation also indicates that violating the requirement in AWS D1.1, clause 4.9.2, that the width between welds must be less than the length of the welds will not necessarily result in lower-than-expected strength. While these five specimens are not permitted according to AWS D1.1, they exhibited no obvious weaknesses in this study.

Given that all specimens tested in this work were stronger than expected and could be safely designed with $U = 1$, it is worth evaluating the other experimental data on plate tension members using $U = 1$ as well. This comparison includes 336 experiments: the plate specimens in the database of previous experimental results plus those tested in this study. For each of the specimens, strengths were computed for the five limit states listed in Table 1, except that U was taken as 1 for these calculations. Measured material and geometric properties as reported by the experimentalists were used. Mannem (2002) did not report the size of the welds; therefore, weld strength was not calculated for their 16 plate specimens. The lowest strength from the five limit states was taken as the controlling strength (denoted as P_n). The ratio of experimental strength to the controlling calculated strength (i.e., the test-to-predicted ratio P_{exp}/P_n) is plotted against the length-to-width ratio in Figure 17. The shape of the markers indicates the failure mode that controls the calculated strength, and the color of the markers indicates the reference describing the experiments.

Across the 336 plate tension member specimens, only 18 specimens have a controlling test-to-predicted ratio less than unity, 17 of which failed in the weld and one that failed in the member. These relatively few test-to-predicted ratios less than unity are not necessarily indicative of unsafe provisions because resistance and safety factors are calibrated to account for the scatter. No other specimens had a test-to-predicted ratio less than unity, which is especially notable since this evaluation used $U = 1$ and not Equation 1 for tensile rupture. Furthermore, there are no obvious trends between the test-to-predicted ratio and l/w , indicating that use of $U = 1$ (and thus no explicit use of l/w) in the strength equations is appropriate.

The calculated strength for all specimens is either controlled by the tensile yielding, weld rupture, or shear yielding limit states. With $U = 1$, tensile rupture did not control for any of the specimens. Tensile rupture may still control in a design situation where resistance factors or safety factors are applied.

CONCLUSIONS

The shear lag factor defined in Case 4 of AISC *Specification* Table D3.1 (2022) implies that net section tensile rupture is a prominent failure mode for tension members connected by short longitudinal welds. However, the shear lag factor in Case 4 was developed using data from experimental specimens that failed by weld rupture. With short welds, it is logical that weld rupture or base metal strength controls

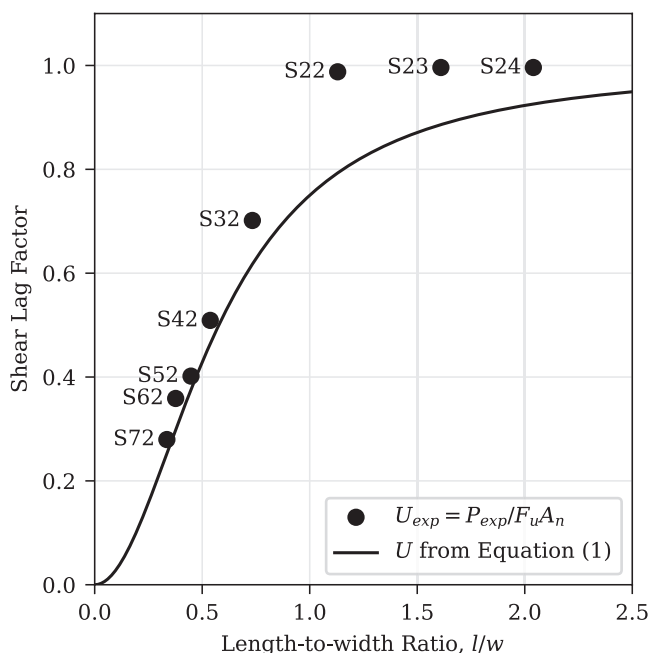


Fig. 16. Comparison of experimental shear lag factor to AISC shear lag factor ($\bar{x} = 0$).

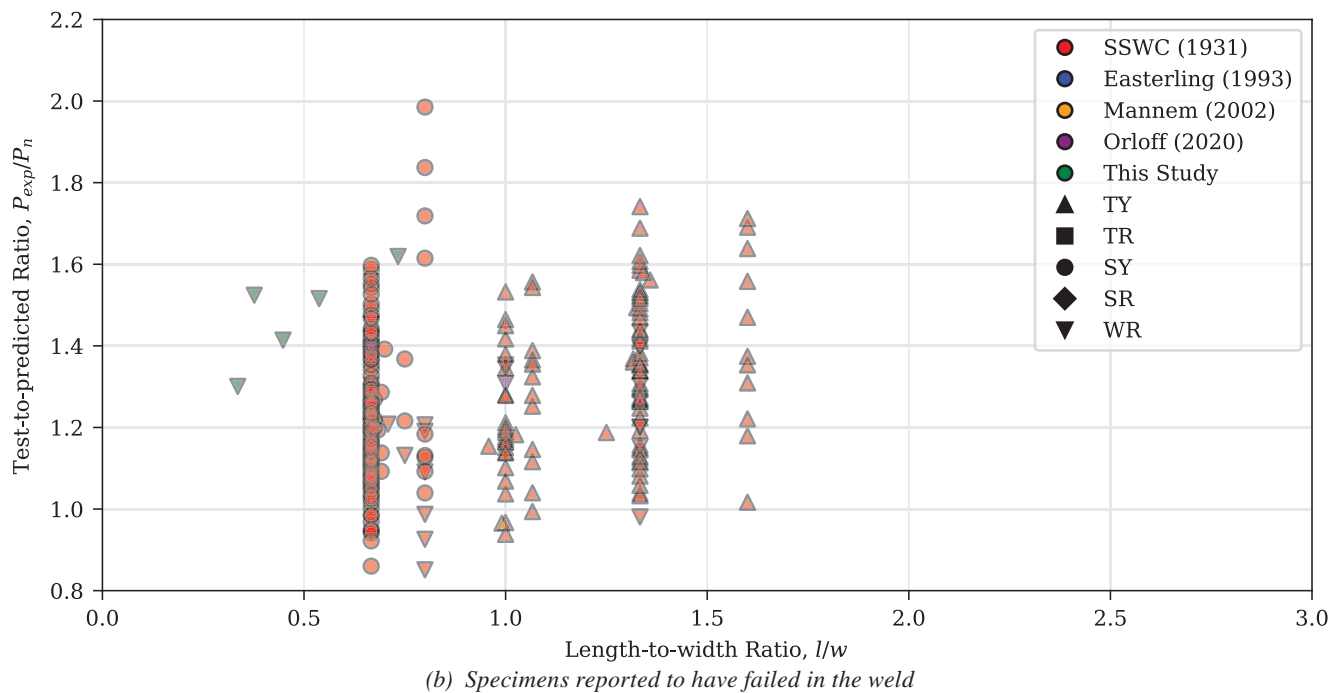
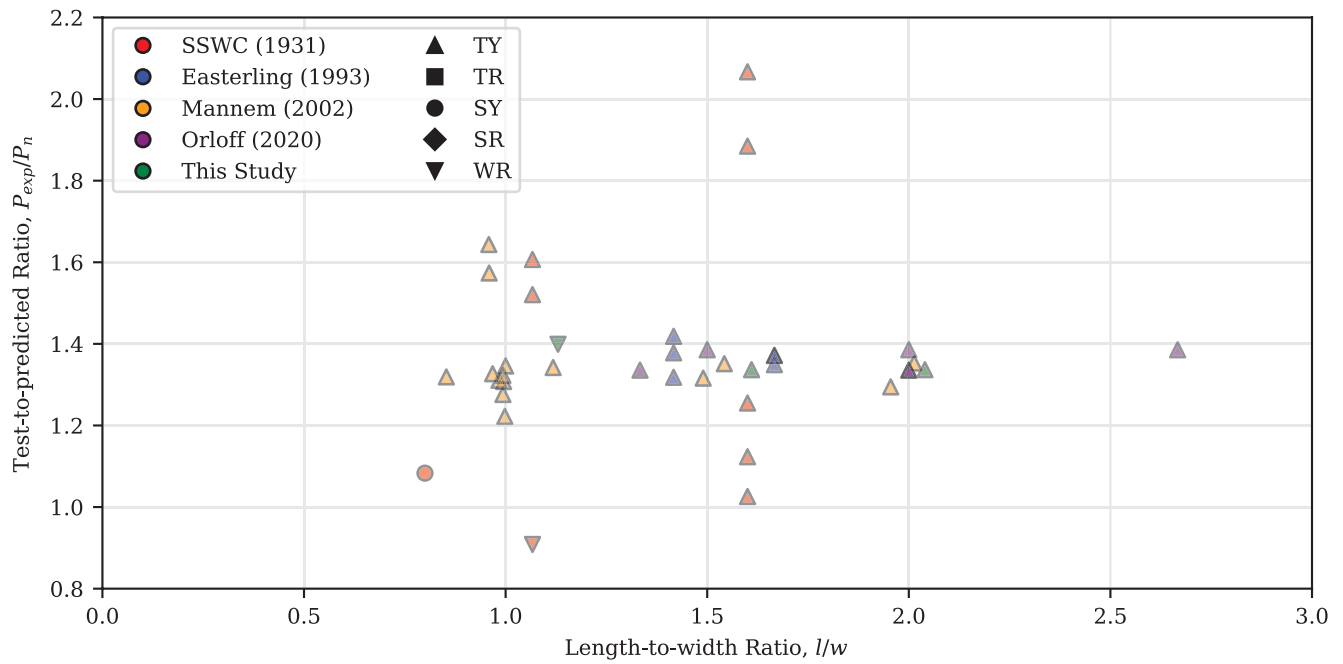


Fig. 17. Ratio of experimental strength to controlling calculated strength versus weld length-to-width ratio.

over member failures such as tensile rupture. To help clarify behavior and start building justification for simplifications to the AISC *Specification*, past experimental work was reviewed, and new experiments were performed in this study. The new experiments were of double-plate tension members and were unique in their use of very short welds with length-to-width ratios as low as 0.336. The primary conclusions of this study are as follows:

- Regardless of the weld length-to-width ratio or failure mode, all eight specimens failed at approximately 110 kips, indicating that shear lag had minimal impact on the tensile rupture strength and weld rupture strength of these specimens despite design provisions indicating otherwise.
- Using a shear lag factor of $U = 1$ for the eight plate specimens tested in this work results in calculated strengths that are safe and accurate in comparison with their experimental strengths.
- Using a shear lag factor of $U = 1$ for the hundreds of plate specimens tested in previous research results in calculated strengths that are safe and accurate in comparison with their experimental strengths.
- Use of $U = 1$ for single or double plates with short longitudinal welds is safe because other limit states, including weld rupture and shear yielding, control over tensile rupture.

These results indicate that simplifications to the tensile rupture provisions in the AISC *Specification* are possible and that the simplifications could enable design calculations that better represent the physical behavior of tension members. However, additional research is needed to fully justify such changes. Specifically, more data is needed for tension members other than plates, including angles and channels, especially those connected with short longitudinal welds.

ACKNOWLEDGEMENTS

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High Strength Structural Steel in Building Applications

JUDY LIU

INTRODUCTION

Ongoing research is taking steps forward in the adoption of high strength structural steel in building design and construction. This University of Cincinnati (UC) study is led by Dr. Rachel Cross, Associate Professor in the Department of Civil & Architectural Engineering & Construction Management. Dr. Cross's research interests include structural steel design and behavior, performance-based fire design of steel buildings, seismic analysis and design, and multi-hazard resilience. Dr. Cross is an AISC Terry Peshia Early Career Faculty Award recipient, recognized for contributions and high potential in structural steel research and teaching. Graduate student Kara Stall is one of AISC's inaugural Peggy A. McCartney/B&B Welding Company, Inc., PhD Fellows. The fellowship supports full-time PhD students committed to academia and participating in AISC-funded research—in this case, material characterization and implementation strategies for high strength structural steel in building applications. Selected highlights from the work to date are presented, along with a preview of future research tasks.

BACKGROUND AND MOTIVATION

The interest in high strength steel in building design and construction is clear, but current design guidance and specifications are not. Benefits include structural performance as well as weight and cost savings. Such benefits have been realized in bridge design, for example. However, the “success with high-strength steel in U.S. bridges has been slow to translate to the building market.” (AISC, 2019). In 2017, an Ad Hoc Task Group on High Strength Steel was charged with guiding AISC committees to improved clarity and pathways in the AISC *Specification for Structural Steel Buildings* (2022) for adoption and innovation in application of high strength steel in building construction. The Task Group (TG) reviewed technical literature on high strength structural steel (named HS3 by the TG); studied HS3

applications across building, bridge, and other industries domestically and internationally; examined new material adoption pathways and standards worldwide; and developed recommendations (AISC, 2019).

High strength steel properties and uses vary by sector. In building construction, HS3 is any steel with specified minimum yield stress, F_y , of 65 ksi or greater. HS3 processes include high strength low-alloy (HSLA) for rolled shapes, quenching and tempering or self-tempering (Q&T or QST), and thermo-mechanical control processes (TMCP). High strength is typically 80 ksi or greater in bridge construction today. Specifically, high-performance steel (HPS) has been developed for improved weldability and fatigue performance. HPS is commonly used in built-up shapes (from plate) that may be hybrid members, with a lower grade in the web plate compared to the flange plates. This concept that “key members in a structure may use different grades of steel to meet a particular need ... has seen limited use in bridge construction, but is a relatively new thought for building construction outside of seismic applications.” (AISC, 2019)

Obstacles to implementation include material behavior and questions about the validity of current design specifications. HS3 stress-strain behavior differs from mild steel in a lower yield to tensile strength ratio, a smaller or absent yield plateau, and a shorter strain hardening region, as shown in Figure 1. This may affect the tension member limit state of yielding on the gross cross section, for example. Whereas gross cross-section yielding may be clear for mild steel members, the limit states and strengths may change over relatively low deformation levels in high strength steel members. In addition, the modulus of elasticity of HS3 does not change relative to conventional steel, so use of the material is best realized in force-controlled applications. Members that are deflection-controlled would likely not benefit significantly by shifting to HS3 materials.

The practice of expressing residual stresses in terms of F_y presents another challenge. Studies on high-strength welded built-up and rolled shapes (e.g., Rasmussen and Hancock, 1995; Ban et al., 2012) show that residual stresses can be $0.2F_y$ or lower, even in welded sections. Simplified expressions—for example, $0.3F_y$ based on mild steel—may overestimate residual stresses and underestimate HS3 member capacity.

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This issue extends to element width-to-thickness limits. Compact, noncompact, and slender element criteria are also functions of F_y . These limits were developed considering steel with F_y less than or equal to 50 ksi. HS3's residual stress and strain hardening characteristics may affect local buckling and motivate changes to these limits (AISC, 2019).

Provisions related to plastic hinging and inelastic rotations may also need to be revisited. AISC 360, Appendix 1.3.2a, stipulates that the specified minimum yield stress, F_y , shall not exceed 65 ksi for members subject to plastic hinging (AISC, 2022). This restriction reflects a limitation of the studies used to develop plastic analysis and design provisions. Some small-scale bend test results supported a proposed increase to $F_y = 70$ ksi, but the AISC Task Committee reviewing the proposal decided that large-scale tests would be needed to demonstrate member-level ductility (AISC, 2019). Related HPS girder research (e.g., Barth et al., 2000; Green et al., 2002) demonstrated that the increase in yield stress and yield-to-tensile ratio, the decrease in strain-hardening modulus, and lower deformation ductility adversely affect the inelastic rotation capacity.

HS3 material and member behavior affect compression and flexural member design. For example, the existing column curves may not adequately predict HS3 member capacity. Also, high strength members may become more slender (globally and locally), requiring more careful attention to torsional and flexural torsional buckling behavior. For slender elements, the strength reduction approach may need to be adapted for higher strength steel given material behavior and residual stresses. Similar considerations exist for

flexural members, as local buckling and lateral-torsional buckling provisions are based on residual stress and assumptions about material behavior (AISC, 2019).

The Task Group conducted a survey of steel designers, which confirmed interest in HS3 as well as current barriers. All 27 respondents agreed they could use steel with yield strength from 65 to 145 ksi. Top choices for structural shapes were W-shapes, plate, and HSS. Trusses and composite columns were most popular applications, and the respondents saw the most potential for HS3 in high-rise buildings (more than 20 stories), arenas, convention centers, and stadiums. Some shared positive experiences, including a switch from 50 ksi to 65 ksi for columns, braces, and truss members to save over 100 tons of material on a high-rise residential project. Current barriers included technical issues already identified and other concerns. Survey participants noted lack of information about cost, market availability, and product applications, as well as limitations on use. Some commented on ductility, toughness, strength versus serviceability, and education needed for steel fabricators and erectors. Respondents also requested research and design guides (AISC, 2019).

With HS3 opportunities and challenges in mind, the Task Group looked to the example of high-performance steel in bridges. In 1992, a consortium developed high performance steel grades 50W, 70W, and 100W (50 ksi, 70 ksi, 100 ksi, respectively) with improved weldability and toughness. The 1998 Transportation Equity Act for the 21st Century partially funded use of new grades in highway bridges. Built-up bridge girders with 70W steel have seen fairly widespread

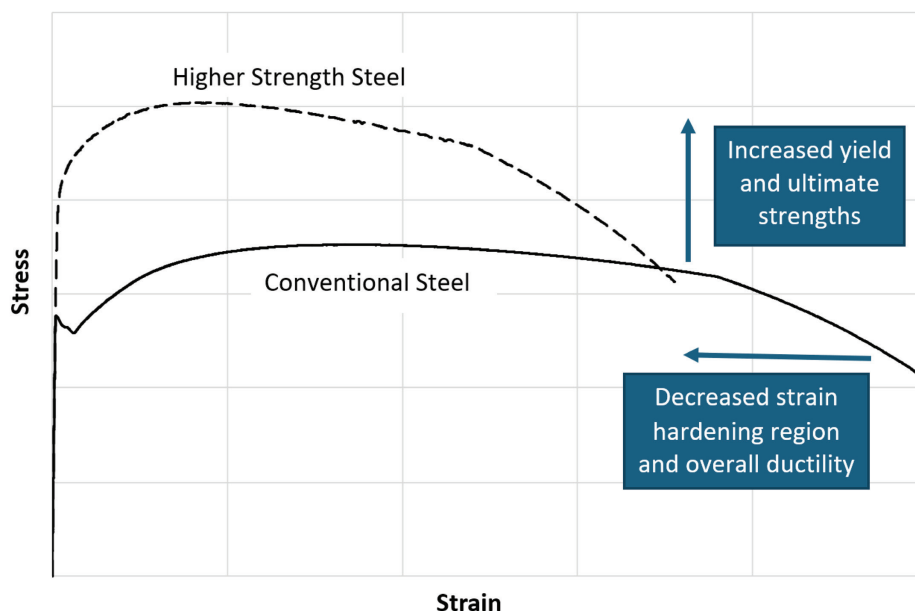


Fig. 1. Representative stress versus strain behavior of high strength steel relative to conventional steel.

adoption, along with some hybrid girder design using 50W webs and 70W flanges. Some of the primary takeaways for the Task Group were to focus on specific HS3 grades, utilize research, and recognize that implementation is not a simple swap of one grade for another; some “reengineering is required to reap the rewards” (AISC, 2019).

The Task Group developed recommendations across education, advocacy, fabricator needs, *Specification* Task Committee efforts, and research. Education efforts should include cost and availability information as well as success stories and case studies. Advocacy involves incentivizing demonstration projects, partnering with funding agencies and other organizations. To address fabricator needs, the Task Group recommended identifying fabricator challenges while developing and sharing best practices. The Task Group also recommended that the *Specification* Task Committees collaborate with counterparts in other countries to inform *Specification* updates and develop paths forward for new materials, specifically HS3. In research, development of performance criteria, investigating HS3 design solutions, and improved provisions are priorities. Research topics supporting improved specifications include imperfections, residual stress, column behavior and column curves (AISC, 2019).

Current Uses and Benefits

Building case studies have highlighted best uses and benefits for using HS3. Benefits may include smaller members with lower weight and costs, smaller foundations, and reduced carbon emissions. Some notable uses of high strength steel are in two New York City buildings—One World Trade Center’s perimeter steel moment frame and 217 West 57th Street’s steel mega-columns. 217 West 57th Street reported a 30% savings in weight of steel. Bay Adelaide Centre, East Tower, Toronto, used high strength steel in columns and short-span transfer girders, recording greater than 9% weight savings.

HS3 columns for 150 North Riverside, Chicago, resulted in 6% savings in total steel tonnage (Finnigan et al., 2015). The North Riverside building was the first use of 70 ksi rolled steel members in the United States. Columns with shop-welded moment connections were designed with 65-ksi steel, eliminating standard pre-heat requirements (AISC, 2018).

The Union Station Tower in Chicago, Illinois, was the first to use 80 ksi steel in the United States. The HS3 perimeter columns helped to reduce the building’s total steel weight by almost 20% compared to conventional steel (Yoders, 2020). With the smaller section sizes, the design team was able to achieve more tenant space. The case study supported a focus on force-controlled columns, maximizing benefits of the increased yield strength.

RESEARCH OBJECTIVES AND PLAN

This research builds on the growing interest in high strength steel for building applications and supports implementation through foundational work for improved design specifications and guides. The overarching goal is to close the knowledge gaps in material and member behavior while informing design with HS3. For material and member behavior, specific research objectives are to identify specific grades of interest, perform lab testing to investigate stress-strain behavior as well as residual stress and ductility assumptions, and evaluate the applicability of current column curves for HS3 wide-flange shapes. For design, the team will recommend changes as needed to the AISC *Specification* for HS3 members. The team will also identify key building components that would benefit from use of HS3 and evaluate material, cost, and carbon savings from using HS3 in place of conventional steel (50 ksi).

The research is organized into four major tasks across material behavior, member behavior, and design. Task 1, Development of Material Matrix and Selection of Testing Material, includes surveys as well as a review of the literature. Task 2, System-Level Comparison of HS3 and Conventional Steel, is based on designs of prototype buildings. Task 3, Material Characterization of HS3, includes tensile coupon and compression tests, residual stress measurements, and three-point bend tests. Task 3 results inform finite element models in Task 4, Development of HS3 Column Curves.

DEVELOPMENT OF MATERIAL MATRIX AND SELECTION OF TESTING MATERIAL

The research team conducted a literature review on HS3 properties and behavior, the state of practice with regards to HS3 case studies and current standards, and published studies informing the experimental program. They also conducted their own surveys on HS3 materials and applications. Selected portions of the literature review and surveys are highlighted.

The literature review informed the experimental program. The team studied standards and common practices for tensile testing, compression testing, residual stress measurements, and three-point bend testing. Multiple methods exist for residual stress evaluation. Residual stress measurement techniques include destructive (sectioning), semi-destructive (e.g., center hole drilling), and nondestructive (e.g., neutron diffraction) methods. Sectioning was chosen based on its common use and acceptance (Ziemian, 2010). Sectioning is also the method used in the two studies with published residual stress data for HS3 hot-rolled I-sections (Spoorenberg et al., 2013; Tankova et al., 2021). Residual stresses for the 65 ksi specimens in both studies demonstrated some dependence on the cross-section

geometry; Spoorenberg et al. (2013) tested heavy sections with flanges several inches thick.

The team developed an HS3 material matrix with mechanical properties, material use, production methods, and current producers. Mechanical properties included yield and tensile strength, elongation, and impact energy. Material use corresponded to rolled shapes, plate, etc. Production methods included HSLA, Q&T or QST, and TMCP.

Surveys on material use and applications were sent to designers, producers, fabricators, and erectors. The material survey asked about experience with each ASTM specification in the material matrix and with which grades of steel. Designers were asked if they used the material in gravity columns, lateral columns, beams, truss members, and/or built-up sections. Fabricators had the opportunity to note any difficulties or limitations with respect to fabrication with the specified material. Nineteen respondents included nine engineers, three fabricators, two producers, and a mix of project and quality managers. Only ASTM A913 (2019a) had been used by all engineers and in a variety of applications. All had used Grade 65, five used Grade 70, and one had used Grade 80.

The HS3 project survey requested information about current and past HS3 building projects. Name, location, and type of project (e.g., high-rise, long-span construction, stadium/arena); number of stories; and building use (e.g., office, industrial, residential) were collected. Designers were also asked for the grade of steel and for which members, their motivation for using HS3, the benefits provided by HS3, as well as any challenges. The team collected several responses for buildings ranging from 7 to 73 stories. The most common use of HS3 was in the gravity columns.

Working in consultation with the project advisory committee, the research team decided to focus on ASTM A913 Grade 80 rolled wide-flange shapes for this research. Results from the surveys indicated that designers were generally reticent to incorporate built-up members and would be more inclined to use rolled shapes. In addition, most interest was in column applications. Grade 80 was added to ASTM A913 in 2019 and to AISC 360 in 2022, but research on this material and its applicability to the existing member design equations is still limited. There are two different steel producers who supply this new material and agreed to collaborate. For these reasons, the research team and project advisory committee felt that studying ASTM A913 Grade 80 material would be most impactful toward implementation of high strength steels in building applications.

SYSTEM-LEVEL COMPARISON OF HS3 AND CONVENTIONAL STEEL

This task compares the use of HS3 and conventional steel in prototype buildings. The team is evaluating HS3 for

composite beams and columns and in columns of lateral systems (wind and seismic). However, the focus is on HS3 for the gravity columns, which are expected to have the greatest impact on cost and material savings.

These case studies utilize a diverse prototype building inventory. Prototype selection from the AISC Steel Solutions Center (AISC, n.d.) and other research projects (Gupta and Krawinkler, 1999; NEHRP, 2010) considered number of stories, building heights, story heights, and framing bay dimensions. The 3- to 35-story buildings vary in footprint—for example, 100 ft by 100 ft or 270 ft by 120 ft, and occupancy (residential, office, parking garage, etc.). Building use influences story height, which ranges from 10 ft to 15 ft, and typical bay dimensions—such as 30 ft by 30 ft or 62 ft by 30 ft. Three sample buildings are shown in Figure 2 to demonstrate the scope of building size and massing.

The prototype buildings were designed and compared. Baseline designs were created first for conventional steel ($F_y = 50$ ksi). There were two HS3 gravity column redesigns for each building with nominal 80 ksi and 100 ksi steels. (Note that 100 ksi rolled shapes are not currently produced but were designed nevertheless, assuming applicability of the existing column equations to illustrate the potential savings through use of high strength steel.) The designs were completed following the *International Building Code* (ICC, 2018). Column sizes and building weights were compared for reductions in steel tonnage. Building foundation designs are under way to quantify anticipated benefits of reduced foundation sizes. Material cost savings and carbon emissions reductions were also calculated.

Comparisons were made for HS3 yield strength, methodology, and number of stories. Three groupings were used for number of stories: 1–10 stories, 11–20 stories, and 21+ stories. Preliminary results showed average column tonnage savings (compared to conventional steel designs) of approximately 19 and 23% for buildings with 80 and 100 ksi columns, respectively. Buildings that fell into the 21+ story category recorded average column tonnage savings of approximately 25 and 31% for 80 and 100 ksi columns, respectively. An increase in yield strength corresponded to an increase in column tonnage savings.

Additional studies considered the influence of live load magnitude as well as story height to assess the impact on the HS3 in column applications. Composite columns using high strength steel were also studied. The use of HS3 in lateral-force-resisting systems [e.g., as columns in moment frames (MF) and concentrically braced frames (CBF)] is also being explored. At this time, it is not anticipated to use HS3 for the dissipative fuse elements of the seismic lateral-force-resisting systems (e.g., the braces in CBFs or beams in MFs), but the three-point bend tests to be conducted in the material testing portion of this project may inform these decisions. Findings of these comparative design studies will be presented in a future publication.

MATERIAL CHARACTERIZATION AND COLUMN CURVES

Work in progress includes the material characterization and development of HS3 column curves. Based on survey results, ASTM A913 Gr. 80 wide-flange shapes are the primary focus for these tasks.

Material Characterization

Laboratory tests are conducted to investigate material and member behavior. Specifically, the team is evaluating HS3 stress-strain behavior, residual stress distributions, and rotational ductility. The stress-strain and residual stress behavior are foundational to the development of column curves. The rotational ductility is necessary to assess the potential of the material in seismic applications.

The tensile coupons, compression specimens, and residual stress specimens were cut from the flanges and webs of the same group of W14 shapes: W14×283, W14×211, W14×176, and W14×398. The wide-flange selection was primarily based on availability from the two participating steel producers, but element slenderness and other cross-sectional properties were considered. The currently limited list of available shapes is expanding. Specified minimum yield strengths are 80 ksi for W14×283 and W14×211; 65 ksi for W14×398; and 50, 65, and 80 ksi for W14×176. The mix of yield strengths and shapes provided a conventional steel baseline and variations across yield strengths and cross sections.

The tension and compression tests together provide comprehensive information about HS3 stress-strain behavior. The tensile coupons were cut from flange and web locations

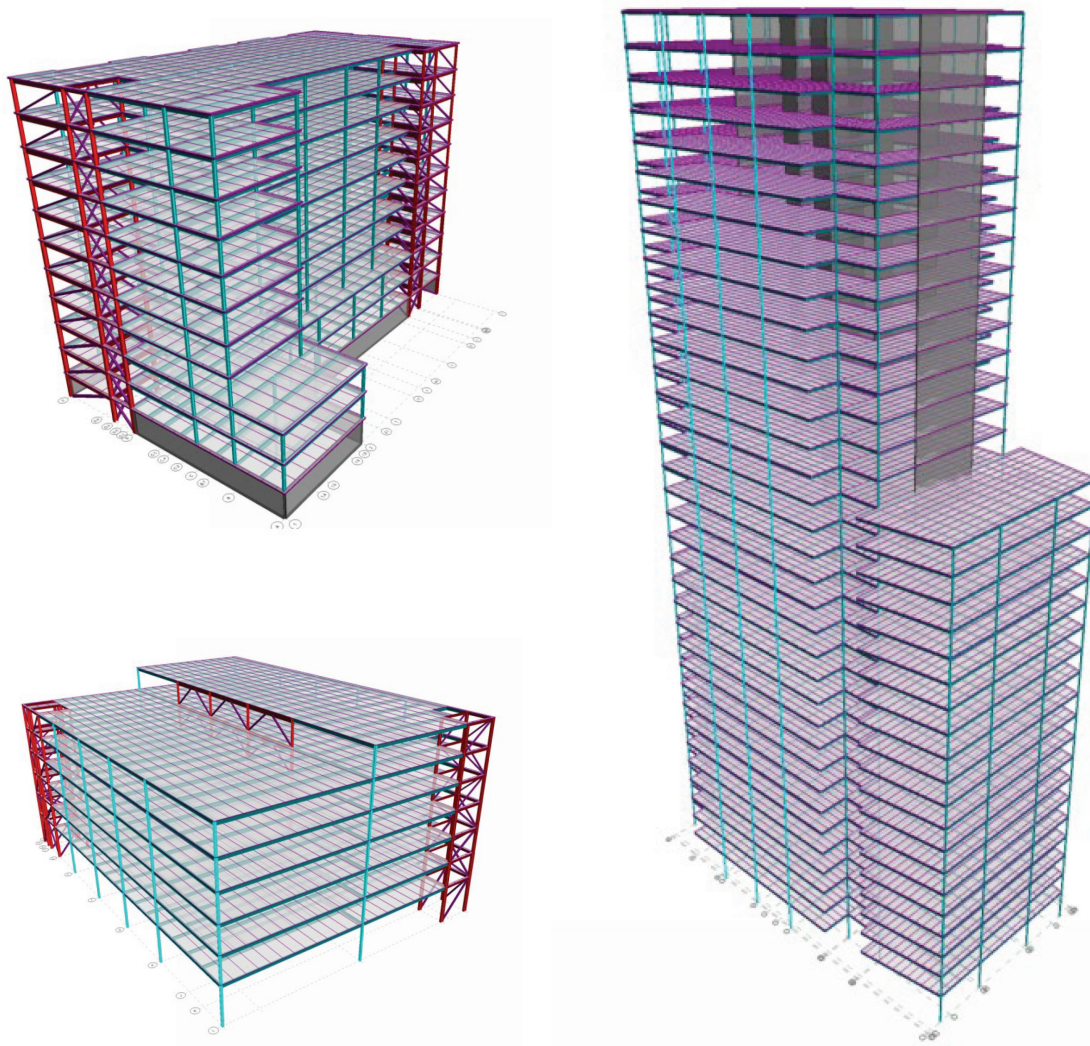


Fig. 2. Sample prototype buildings.

at dimensions and geometries that satisfy ASTM A370-24 (2024) requirements. For example, the flange coupons are cut from a location two-thirds from the flange centerline to the flange toe. Compression tests follow ASTM E9-19 (2019b) and have an approximate length to thickness ratio of 3.0. Compression test specimens were taken from the W14×211 and W14×176 flanges and web; specimens from the thicker W14×283 and W14×398 flanges would exceed the test machine capacity. Full-thickness specimens are needed to properly capture the quenched and tempered properties through the section. The tension and compression tests are conducted on a Tinius Olsen machine, following the appropriate loading protocol and procedures for measuring the stress-strain behavior (ASTM, 2024, 2019b). Photographs show the fractured coupon from one of the tensile tests (Figure 3).

The same W-shapes were used to assess residual stresses. The residual stress measurements for each wide-flange section are made using the sectioning technique (Ziemian, 2010, Stall et al., 2024). The original specimens are at least three times the W-shape's largest transverse dimension, plus the gage length of 10 in., plus 2 in. For the W14×398, the overall specimen length is 7 ft 6 in.; all others are 5 ft 6 in. Test pieces were cut from the original length before the strips were cut. The flanges and web are marked with 0.5-in.-wide strips (Figure 4). Stainless-steel locating points with indentations are epoxied to the ends of the strips to assist with measurements. Length measurements were taken prior to the test piece being cut, after the test piece was cut, and then after the strips were cut. Changes in length are used to calculate the residual stresses that have been released during the cuts. Stress distributions across the thicker flanges are captured by also sectioning through the flange thickness.

Rotational Ductility

Five three-point bend tests will be used to evaluate moment-rotation behavior and ductility. This portion of the study addresses knowledge gaps relevant to seismic applications and is not applied to the HS3 column curve development. Three 80 ksi specimens (W14×283, W14×211, and W14×176) will be compared to 50 and 65 ksi specimens (W14×176). Prior to testing, the wide-flange specimens are measured at nine locations along the length of the beam. Cross-sectional dimensions and imperfection measurements (camber and sweep) are taken. The 22 ft, simply supported test specimens are loaded at mid-span (Figure 5). Lateral bracing is provided at mid-span and near the supports. The beams are loaded past the formation of a plastic hinge to a peak on the moment-rotation curve. The peak is expected to be between 1.0 to 1.2 times the plastic moment, M_p . If possible, loading will continue until a drop in moment to below $0.8M_p$; local buckling may occur before targets are achieved.

Development of HS3 Column Curves

The experimental test results provide data for development of finite element models and column curves. The UC research team as well as researchers from other universities will use the stress-strain and residual stress measurement data in the development of finite element (FE) models for HS3 columns. These FE models are benchmarked to historical data and then analyzed using HS3 material characterization results. Effective stress-strain material models will be created and used in fiber-based models for a parametric study to create column curves.

Creating benchmarked finite element (FE) models is the first step in column curve development. Test data used



Fig. 3. Result of full-thickness tensile coupon testing.

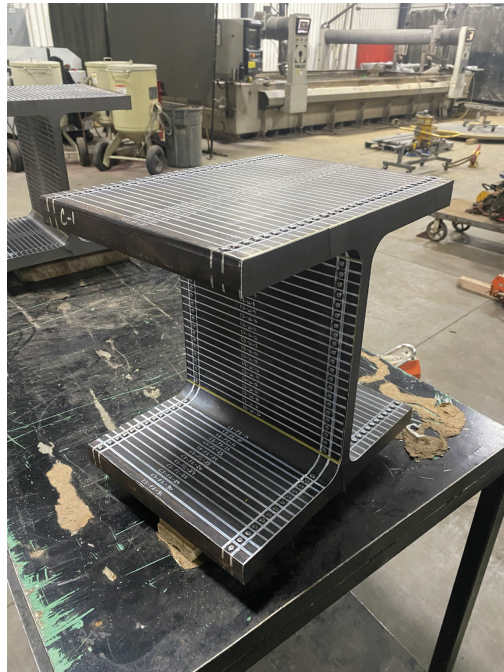


Fig. 4. A residual stress test piece before cutting the individual strips.

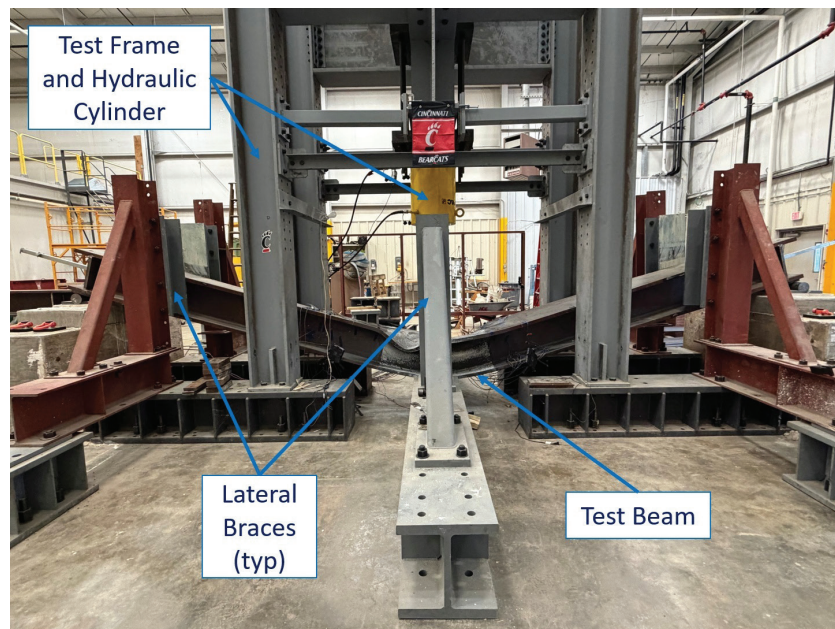


Fig. 5. Post-test photo of a three-point bend test.

for the first column curves (Bjorhovde and Tall, 1971) will also be used here. The published data include material stress-strain behavior, residual stresses, and geometric imperfections. Such data do not exist for hot-rolled HS3 columns. The column tests and models track load versus deflection through buckling and post-buckling. Analysis completed to date demonstrates that the FE models can predict the maximum compression loads within 4%.

Those benchmarked FE models will then be analyzed using the HS3 material characterization data. Models of the tested wide-flange shapes will incorporate the data from the tensile tests, compression tests, residual stress sectioning, as well as geometric imperfections. The HS3 modeling will be expanded to include additional wide-flange shapes for the development of phenomenological stress-strain curves (Alghossoon and Varma, 2023). For the HS3 column modeling, sections will include W12s as well as more W14s, W18s, and W27s, with flange ratios from 2.15 to 7.11 and web ratios from 5.21 to 18.9. The objective is to develop a generalized stress-strain curve capable of capturing local buckling behavior for any ASTM A913 (2019a) Grade 80 wide-flange section.

A parametric study will then be used to create an HS3 column curve. This study will utilize a fiber-based model, first benchmarked against the same data used for the FE models. Further benchmarking will compare fiber models to the wide-flange FE models used for the stress-strain curve development. Parameters include global slenderness ratios, as well as flange and web width-to-thickness ratios. Wide-flange members will be selected from available ASTM A913 (2019a) sections. The models may include Grades 50, 65, and 70, but priority will be given to sections currently manufactured in Grade 80.

Once the column curve work has been completed, the research team may recommend revisions to the AISC *Specification* for HS3 members. The prototype building designs will be revisited to demonstrate any additional savings with the revised column design procedure. Additional recommendations may stem from the prototype building work, with best practices for maximizing the benefits of HS3 members for structural performance, weight and cost savings, and reduced carbon emissions.

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