Proceedings of the
Annual Stability Conference
Structural Stability Research Council
Baltimore, Maryland, April 10-13, 2018

Stability of Steel Modules During Construction

Soheil Shafaei[1], April Wang[2], Amit H. Varma[3], Brian Morgen[4]

Abstract

Concrete-filled composite plate shear walls (CF-CPSW) are being considered for high-rise buildings because of their potential for modularity, construction speed, and structural efficiency. The system is composed of a concrete core sandwiched between two steel faceplates. The steel plates are connected to each other by tie bars, rods, or steel shapes, and composite interaction between steel faceplates and infill concrete is developed by these tie systems and headed stud anchors (if included). The empty steel modules–which are composed of steel faceplates, tie bars, and shear studs–are fabricated in a shop and shipped to the site for erection. The erected steel modules serve as formwork and falsework during construction and concrete casting. The stability of the steel modules during construction, while supporting construction loads, the weight of the surrounding steel frames, and the floor systems during concrete casting, is vital.

This paper presents the results of numerical and analytical studies conducted to evaluate the stiffness and stability of empty steel modules (of CF-SPSW) to resist gravity loads during construction. The stability of empty modules is governed by their effective shear stiffness (GA_{eff}), which in turn depends on the relative flexural stiffness (EI/L) of the faceplates and the tie bars. The finite element method can be used to determine the effective shear stiffness (GA_{eff}) and critical buckling stress (σ_{cr}) of the steel module. Additionally, the effective shear stiffness and the critical buckling stress can also be estimated conservatively using simple equations developed using a mechanics-based approach and proposed in this paper for design purposes.

When the ratio of the steel faceplate flexural stiffness (EI_p/S, where s is the tie spacing) to the tie bar stiffness (EI_t/d, where d is the tie diameter) is less than 25, the buckling of empty steel modules is somewhat independent of the load eccentricity, end conditions, and module height. For the design of steel modules, it is not recommended for this ratio of EI_p/S to EI_t/d to be greater than 25, because the empty modules become extremely flexile and the critical buckling stress becomes less than 1000 psi.

[1] Graduate Research Assistant, Lyles School of Civil Engineering, Purdue University, <sshafaei@purdue.edu>
[2] Graduate Research Assistant, Lyles School of Civil Engineering, Purdue University, <wang990@purdue.edu>
[3] Professor, Lyles School of Civil Engineering, Purdue University, <ahvarma@purdue.edu>
[4] Principal, Magnusson Klemencic Associates, <bmorgen@mka.com>

1. Introduction

Concrete-filled composite plate shear walls (CF-CPSW) are being considered for high rise building construction in the US and abroad due to their potential advantages in terms of modularity, construction schedule and structural performance. CF-CPSW consist of a concrete core sandwiched between two steel faceplates located on the surfaces. The steel faceplates are tied to each other using tie bars, rods, or steel shapes, and anchored to the concrete core using these tie systems and headed stud anchors, as shown in Fig. 1 (Varma et al. 2014 and 2015, Bruhl et al. 2015, Sener and Varma 2014, and Sener et al. 2015).

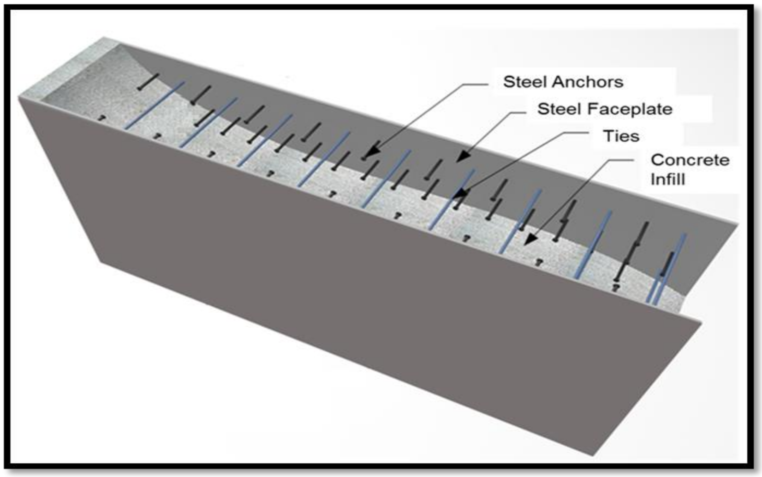

Figure 1: A typical CF-CPSW and its component

The steel modules−consisting of the faceplates, ties, and studs−are fabricated in the shop and shipped to the field for the erection. The erected steel modules serve as formwork and falsework for construction activities and concrete casting. They support considerable construction loads, the weight of the surrounding steel framework, and concrete floors during casting. The modularity of steel construction (before concrete construction activities) is the primary appeal and advantage of CF-CPSW systems (Varma et al. 2015). The stability of the empty module is a primary design requirement and prerequisite for the feasibility of the CF-CPSW system.

Ramesh et al. (2013) did research on empty steel modules and investigated the effects of concrete casting on stresses induced in the tie bars. They also conducted a load test on a two-story empty steel module to investigate the effects of eccentric axial loading on behavior and buckling. According to the study, steel modules can provide acceptable stability under construction loads .

Corus' "Bi-Steel Design and Construction Guide" Chapter 9 contains methodologies to calculate the effective critical buckling load and shear stiffness for certain module configurations. These methodologies rely on a design table, Table U, which provides maximum mid-span deflection due to shear effects under a 1 kN/m^2 uniform distributed load. The design table values are generated from a computer program. The manual states that "Deflections generated by the program have been validated by tests. However, stresses remain theoretical values based on bending theory" (Bi-Steel 1999).

By contrast, there is a significant lack of knowledge and some misinformation regarding the stability and strength of the steel module before and during concrete placement. In high-rise building construction, the empty steel module is expected to support three-to-four stories of the surrounding steel framework before and during concrete placement. As such, the stability and strength of the steel module is critical to the safe and successful application of these CF-CPSW systems in practice.

In this study, detailed 3D finite element models of empty steel modules are developed and used to determine their critical buckling stress by conducting eigenvalue buckling analysis. Different boundary conditions (pinned-pinned, fixed-pinned, and fixed-fixed) are considered, while conducting the finite element analyses. Additionally, simple mechanics-based models are used to develop analytical equations for conservatively estimating the stiffness and buckling capacity of the steel modules. Finally, the finite element models and analytical equations are compared with available experimental results.

2. Finite element method

This section discusses the empty module 3D finite element models developed and analyzed using the commercial program, Abaqus. All the steel plates and tie bars were modeled using higher-order (quadratic) C3D20R elements (20-node solid elements with reduced integration). These higher-order elements were selected because of well-known issues such as shear locking and hourglassing with of lower-order (linear) C3D8 (8-node solid), C3D8R (reduced integration), and C3D8I (incompatible mode) elements. Initial studies indicated that using these lower-order elements resulted in convergence issues due to the localized interaction between the elements of the steel plates and orthogonal tie bars. Similar mesh convergence issues were also noted for models with S4 and S4R (4-node shell) elements for plates and B31 or B32 (beam) elements for tie bars. These convergence issues were due to the drilling degree-of-freedom in shell elements, which are modeled inadequately using artificial stiffness in commercial software, and activated in this model by the shell-to-beam connections.

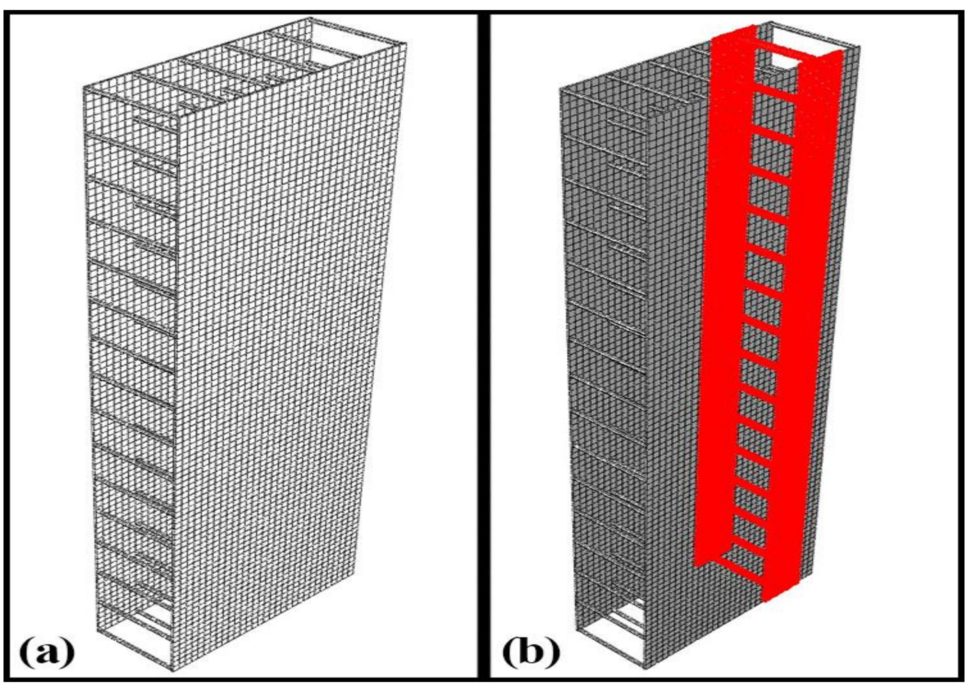

Figure 2: Overall finite element model and the extracted unit width

Fig. 2(a) shows a typical finite element model of an empty module of CF-CPSW, Fig. 2(b) highlights a unit width of the module used for further analytical investigations accounting for translational symmetry. Fig 3 illustrates the typical loading and boundary conditions (pinned-pinned) used for the extracted unit width of the empty steel module. Boundary conditions associated with translational symmetry were modeled along the edges of the steel plates.

Several such finite element models of the unit widths of empty modules were developed and analyzed as part of this research. Three practical wall thicknesses (T_{sc}) of 18, 24, and 36 in. were considered. Two different tie spacings (S) equal to wall thickness (T_{sc}) and wall thickness divided by two ($T_{sc}/2$) were selected. The thicknesses of steel faceplates were in the range of 3/8 in. to 1-1/16 in. The steel faceplate thickness and tie bar diameters were selected based on the associated plate reinforcement ratios ($\rho = 2t_p/T_{sc}$) and the tie bar reinforcement ratios ($\rho_{tie} = \pi d_{tie}^2/S^2 4$). The ratios of ρ and ρ_{tie} were limited to the practical range of 1.5 – 6% and 0.2 – 0.6%, respectively (Varma et al. 2014). The lengths (or column heights) of the finite element models were equal to ten times the wall thickness (T_{sc}).

Table 1 includes the details of the 22 finite element models that were developed and analyzed. The models were subjected to axial loading simulating 1000 psi stress in the steel faceplates, and eigenvalue buckling analyses were conducted to obtain the critical buckling stress and buckling mode. Different boundary conditions (pinned-pinned, fixed-pinned, and fixed-fixed) were considered for each model. The resulting critical stress values (σ_{cr}) are included in Table 1, and the typical buckling modes are shown in Fig. 4(a), (b), and (c) for the different boundary conditions. These buckling modes indicate that the shear deformation of the empty modules is the critical deformation associated with the buckling mode. It is also important to note that the critical buckling stress values are well within the elastic range of behavior of typical structural steel materials with yield strength of 50,000 psi or more.

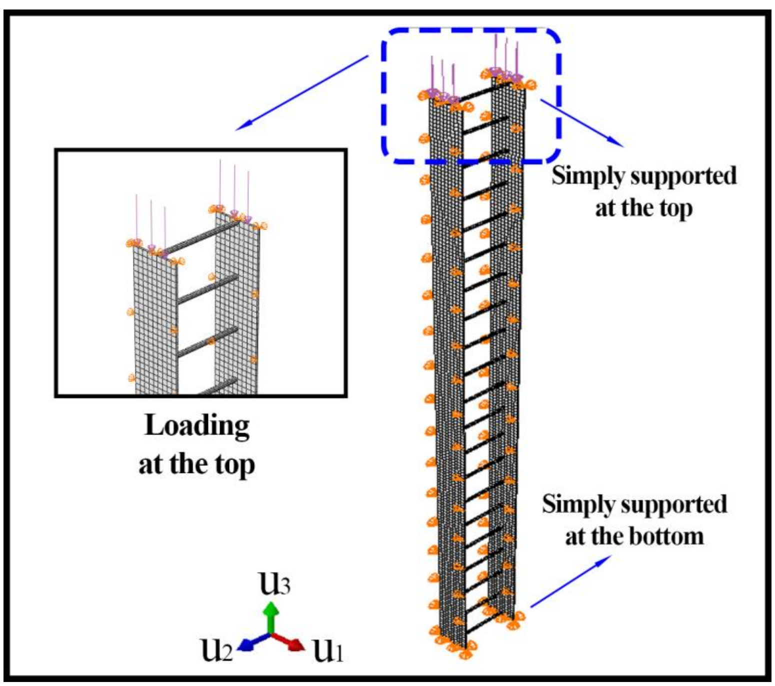

Figure 3: A typical loading and boundary conditions.

Table 1: Critical buckling stress of finite element models of empty module with different boundary conditions.

Models	T_{sc} (in)	S_{tie} (in)	t_p (in)	d_{tie} (in)	σ_{cr} (psi) Pinned-pinned	σ_{cr} (psi) Fixed-pinned	σ_{cr} (psi) Fixed-Fixed
Model 1	18	9	3/8	7/16	769	876	1064
Model 2	18	9	3/8	5/8	2613	2699	2800
Model 3	18	9	3/8	13/16	6200	6261	6300
Model 4	18	9	9/16	7/16	723	981	1451
Model 5	18	9	9/16	5/8	2128	2365	2758
Model 6	18	9	9/16	13/16	5220	5421	5703
Model 7	18	18	3/8	15/16	2349	2413	2493
Model 8	18	18	9/16	15/16	2379	2583	2888
Model 9	24	12	1/2	5/8	961	1066	1236
Model 10	24	12	1/2	7/8	3038	3119	3204
Model 11	24	12	1/2	1-1/16	5670	5731	5830
Model 12	24	12	3/4	5/8	864	1120	1579
Model 13	24	12	3/4	7/8	2483	2714	3082
Model 14	24	12	3/4	1-1/16	4790	4992	5307
Model 15	24	24	1/2	1-3/16	2024	2092	2200
Model 16	24	24	3/4	1-3/16	2015	2227	2571
Model 17	36	18	3/8	15/16	1307	1322	1336
Model 18	36	18	3/4	15/16	954	1058	1231
Model 19	36	18	3-4	1-5/16	2979	3060	3177
Model 20	36	18	1-1/16	15/16	861	1088	1492
Model 21	36	18	1-1/16	1 5/16	2513	2714	3046
Model 22	36	18	1-1/16	1 9/16	4555	4732	5010

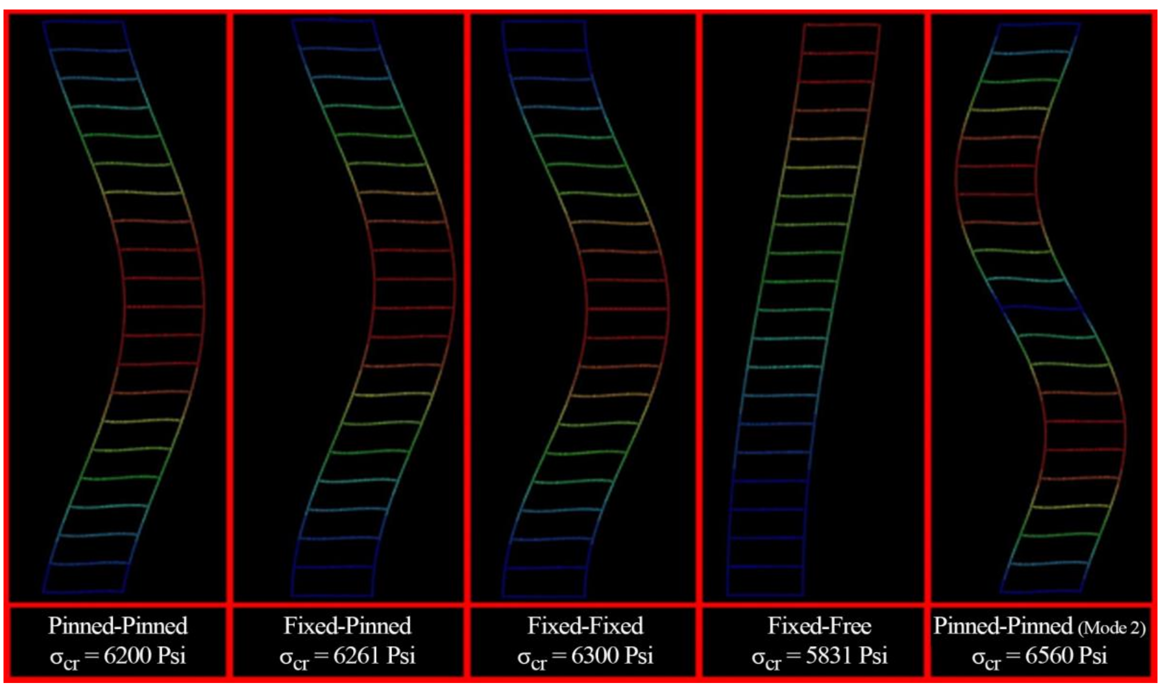

Figure 4: Results from eigenvalue buckling analysis of Model 3 with different boundary conditions

Fig. 5 plots the critical buckling stress (σ_{cr}) of the steel modules (reported in Table 1) with respect to the tie bar diameter. It includes the results for modules with pinned-pinned boundary conditions, wall thickness (T_{sc}) of 18 and 24 in, and tie spacing equal to $T_{sc}/2$. The figure clearly shows that the critical buckling stress values are within the range of 1000 to 6000 psi. The critical buckling stress (σ_{cr}) increases significantly with the diameter of the tie bars, which improves the shear stiffness (GA) of the empty module. The effects of changing the steel plate thickness are relatively small in the range of parameters considered.

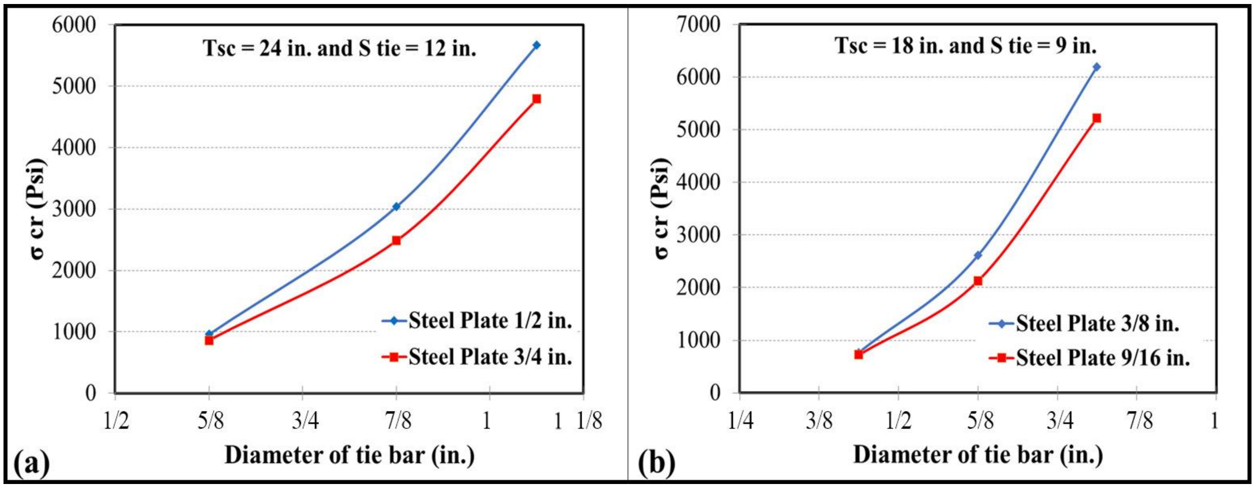

Figure 5: Eigenvalue buckling results of empty modules due to axial compression loading

Figure 4(a-e) also includes the results of eigenvalue analysis of Model 3 for different end conditions including the three mentioned earlier: (a) pinned-pinned, (b) fixed-pinned, (c) fixed-fixed, and two additional: (d)fixed-free, and (e) the second mode from pinned-pinned. The buckling stress is very similar for all these conditions. Additional analyses were conducted for members with different lengths, and for members with eccentric loading applied on only one plate (instead of both). All these analyses indicated that the critical buckling stresses were relatively similar (within 10% variation) irrespective of the end conditions, member length, and loading eccentricity (for practical ranges of length of 5 – 10 times the wall thickness). This raises the question about the fundamental behavior and buckling mode of empty steel modules, which was further investigated and explained using the mechanics-based model discussed in the following section.

3. Mechanics Based Model

3.1 Shear stiffness of empty module

The finite element analysis results and buckling modes indicated that the stiffness of empty modules is governed by their Vierendeel truss type behavior and effective shear stiffness (GA_{eff}). A unit cell was defined as a characteristic portion of the unit width of the empty module shown in Fig. 3. The unit cell can be repeated in a chain with translational symmetry to make the unit width of the empty module, and the unit width can be repeated in a chain with translational symmetry to make the empty module of the CF-CPSW. The unit cell of the empty module was identified to consist of just one tie bar and the associated (tributary) steel plates (up to one-half of the tie spacing, S) on both sides. The unit cell can be idealized as a simple H or I-frame with the members defined by steel plates and tie bars as shown in Fig. 6. The member idealizing the steel plate has a rectangular cross-section with width equal to the tie spacing, S, and depth equal to the plate thickness, t_p. The member idealizing the tie bar has circular cross-section with diameter equal to the tie bar diameter, d_{tie}. Frame analysis conducted using slope-deflection or similar equations can be used to calculate the effective shear stiffness (GA_{eff}) of the unit cell.

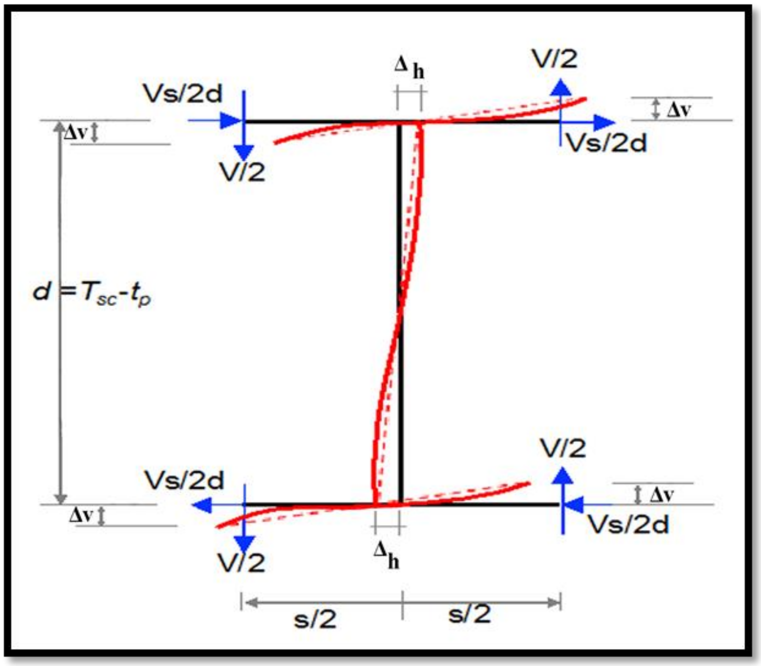

Figure 6: Free body diagram of unit cell or shear panel subjected to pure shear loading.

The effective shear stiffness (GA_{eff}) was calculated using Eq. 1 as the ratio of the effective shear force divided by the effective shear strain for the unit cell. In the equation, V is the applied vertical shear force, S is the spacing between ties, d is the center-to-center distance between the plates, and Δ_v and Δ_h are as identified in Fig.6.

$$GA_{eff} = \frac{V}{\frac{2\Delta_v}{S} + \frac{2\Delta_h}{d}} \tag{1}$$

The calculated value of GA_{eff} using slope-deflection equations is shown below:

$$GA_{eff} = 24\left(\frac{E \times I_t}{d \times S}\right)\frac{1}{\left(2 + \frac{1}{\alpha}\right)} = 24\left(\frac{E \times I_p}{S \times S}\right)\frac{1}{(2\alpha + 1)} \tag{2}$$

Where E is the modulus of elasticity, I_t and I_p are the moments of inertia of the members modeling the tie bar and steel faceplate respectively. α is the ratio of flexural stiffness of steel plate to tie bar:

$$\alpha = \frac{\left(\frac{E.I_p}{S}\right)}{\left(\frac{E.I_t}{d}\right)} = \frac{\left(\frac{st_p^3}{12S}\right)}{\left(\frac{\pi d_{tie}^4}{64d}\right)} \tag{3}$$

3.2 Bifurcation buckling

The bifurcation buckling of members with significant shear flexibility is discussed in Figure 7. When the member flexibility is dominated by shear deformations, then the critical buckling load, P_{cr}, is equal to the effective shear stiffness GA_{eff}. This well-known, but sometimes forgotten equation, was proposed by Engesser in 1889. This unique equation suggests that the critical buckling load does not depend (directly) on the boundary conditions, member length, bracing points, or even load eccentricity, which is quite remarkable. The critical buckling load depends just on the effective shear stiffness GA_{eff} of the member. The finite element results presented earlier attest this phenomenon for the empty steel module of CF-CPSW as well.

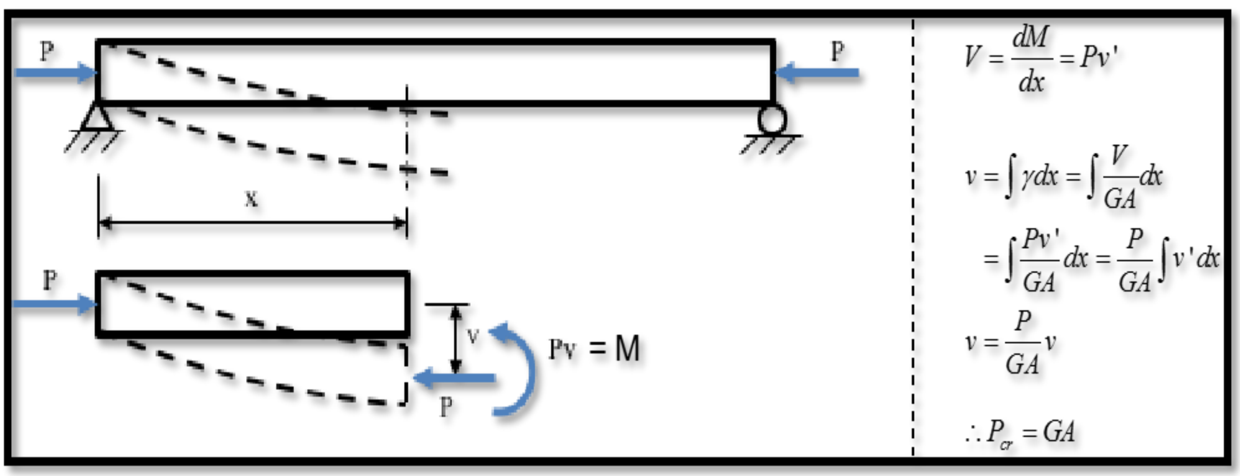

Figure 7: Bifurcation buckling of a shear flexibility dominated member.

Therefore, the critical buckling stress, σ_{cr}, can be calculated using Eq. 4. In the equation, the effective shear stiffness, GA_{eff}, of empty steel module can be estimated using Eq. 2.

$$\sigma_{cr}^{MBM} = \frac{P_{cr}}{S \times t_p \times 2} = \frac{GA_{eff}}{S \times t_p \times 2} \tag{4}$$

4. Comparison of critical buckling stress from different methods

Eq. 2 provides the mechanics-based equation for estimating the effective shear stiffness, GA_{eff}, of the unit cell and empty module. Using Eq. 4, the critical buckling stress can be calculated approximately in the absence of a detailed finite element analysis. The comparisons of the empty module critical buckling stress calculated using Eq. 4 and the results from the detailed finite element analyses are shown in Fig. 8. This figure, similar to Fig. 5, includes the results for modules with pinned-pinned boundary conditions, wall thickness (T_{sc}) of 18 and 24 in, and tie spacing equal to $T_{sc}/2$. As shown, the critical buckling stress calculated using Eq. 4 from the mechanics-based model is close but conservative with respect to the buckling stress calculated by the detailed finite element method.

As expected, the mechanics-based model is a simple but conservative estimate of the effective shear stiffness of the empty module. It becomes more conservative as the steel plate thickness increases. Once again, it is also important to note that the critical buckling stress is of the order of 1000 – 6000 psi, which is much smaller than the plate yield stress, and further confirmation of the fact that elastic buckling governs the behavior and design of empty modules.

The effective shear stiffness, GA_{eff}, is dependent on α (Eq. 3). It is the ratio of the steel plate flexural stiffness (EI_p/S) to the tie bar flexural stiffness (EI_t/d). Parametric studies, the details of which are not included here, indicate that α should be limited to values less than 25. When α values become greater than 25, the empty steel module becomes too flexible and the critical buckling stress can be smaller than even 1000 psi.

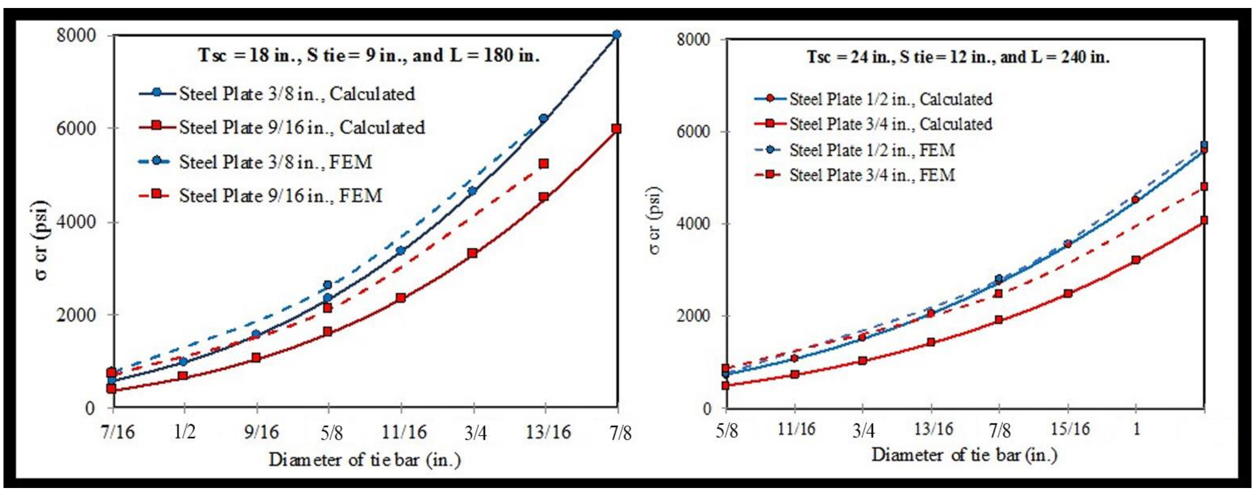

Figure 8: Comparison of critical buckling stress for empty steel modules

5. Experimental Verification

Ramesh et al. (2013) conducted a large-scale test to evaluate the buckling capacity of an empty module. The specimen was a 3/8-scale model of the prototype. The specimen wall thickness (T_{sc}) was equal to 9 in., the plate thickness (t_p) was equal to 3/16 in., the threaded tie bar diameter (d_{tie}) was equal to 3/8 in., and the tie spacing was equal to 4.5 in.

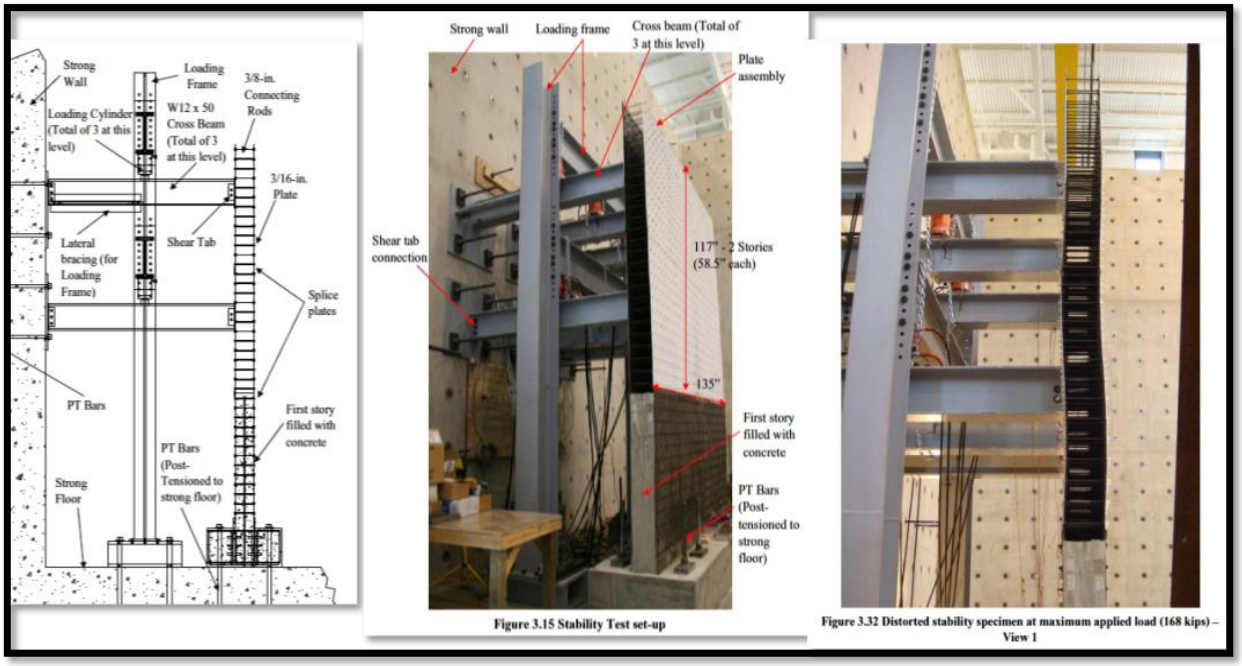

Figure 9: Experimental investigation by Ramesh et al. (2013) (a) Schematic of setup with specimen, (b) photo of specimen before loading, and (c) photo of buckled specimen after loading

Figure 9(a) shows the details of the test setup and the specimen. As shown, the first story of the specimen was already filled with concrete (which had set), and the two stories above it were empty. The loading was applied on only one plate of the empty module, and the loading points also served as brace points. Fig. 9(b) shows a photograph of the specimen before loading. The specimen wall length was 135 in. and the height was 117 in. Figure 9(c) shows the buckled shape of the specimen after loading (close to maximum loading). The deformed (buckling) shape of the specimen indicates behavior similar to the bifurcation buckling of empty modules (with significant shear deformation of tie bars) presented earlier.

Ramesh et al. (2013) reported that the total peak load at buckling was equal to 168 kips. This can be used to calculate the critical buckling stress from the test, which is equal to 3.32 ksi, as shown below:

$$\sigma_{cr}^{Exp} = \frac{P_{Exp}}{L \times t_p \times 2} = \frac{168}{135 \times 0.1875 \times 2} = 3.32.ksi \tag{5}$$

The critical buckling stress for the specimen was calculated using the approach presented in previous sections, i.e, using Eq. 2 to estimate GA_{eff} and Eq. 4 to calculate the critical buckling stress, as shown below. The critical buckling stress was equal to 2.99 ksi, which is conservative (10% lower) with respect to the experimental result. This further confirms that the mechanics-based model is a conservative approach for estimating the buckling strength of empty modules.

$$\sigma_{cr}^{MBM} = \frac{P_{cr}}{S \times t_p \times 2} = \frac{GA_{eff}}{4.5 \times 0.1875 \times 2} = 2.99.ksi \tag{6}$$

A 3D finite element model of the unit width of the tested specimen was developed. The bifurcation buckling load from the eigenvalue analysis of the three-dimensional finite element model was equal to 6.49 kips. The corresponding critical buckling stress was 3.85 ksi, as calculated below, which is 16% larger than the experimental result. The probable reason for this discrepancy was that the specimens used threaded bars going through holes in the steel plates, which had additional (shear) flexibility than the fully-tied (welded) steel plate-to-tie bar connections used in the model:

$$\sigma_{cr}^{FEM} = \frac{P_{FEM}}{S \times t_p \times 2} = \frac{6.49}{4.5 \times 0.1875 \times 2} = 3.85.ksi \tag{7}$$

6. Design recommendation

The stiffness and stability of empty steel modules of CF-SPSW were numerically and analytically are investigated. 3D finite element models of the unit widths (of empty steel modules) were developed and analyzed to determine the critical buckling stress. Mechanics-based models of a unit cell (of the unit width of the empty steel module) were developed and analyzed to calculate the effective shear stiffness and critical buckling stress. The results from the finite element analyses and the mechanics-based models were evaluated and compared leading to the following conclusions:

- Effective shear stiffness of the empty module, GA_{eff}, governs the behavior and stability under gravity loads.

- The 3D finite element method along eigenvalue analysis is the most efficient way to calculate critical buckling stress for steel modules with applicable boundary conditions and loading.

- In the absence of a detailed finite element model, the effective shear stiffness, GA_{eff}, and critical buckling stress, σ_{cr}, for empty steel modules can be calculated conservatively using Eq. 2 and Eq. 4 developed based on the mechanics-based approach.

- The effective shear stiffness and the critical buckling stress of the empty steel module depends on α, which is the ratio of the steel faceplate flexural stiffness (EI_p/S) to the tie bar flexural stiffness (EI_t/d) in the unit cell model.

- Values of α greater than 25 are not recommended for design, because they would lead to modules that are extremely flexible, and with critical buckling stress lower than 1000 psi.

- When a reasonable value is selected for α (for example, less than 25) is used, the critical buckling stress and compression capacity of the empty steel module do not vary significantly with boundary conditions, load eccentricity, and unsupported lengths. The reason is that the effective shear stiffness, GA_{eff}, of steel module governs buckling behavior and strength.

Acknowledgments
The project was supported by Charles Pankow Foundation (CPF Research Grant #06-16) and American Institute of Steel Construction (AISC). However, any opinions, findings, conclusions, and recommendations presented in this paper are those of the authors and do not necessarily reflect the view of the sponsors.

References

ABAQUS (2016). ABAQUS Version 6.16 Analysis User's Manuals. Providence, RI, USA: Dassault Systemes Simulia Corporation; 2016

Bazant Z. P. (2003), "Shear Buckling of Sandwich, Fiber Composite and Lattice Columns, Bearings, and Helical Springs: Paradox Resolved." Journal of Applied Mechanics, Vol. 70, pp. 75-83

Bi-Steel Design and Construction Guide. (1999). Authors: Bowerman, H.G, Gough, M.S. and King, C.M., British Steel Ltd, Scunthorpe, London, 1999

Bruhl, J., Varma, A.H., and Johnson, W.H. (2015). "Design of Composite SC Walls to Prevent Perforation from Missile Impact." International Journal of Impact Engineering, Vol. 75, Jan., pp. 75 – 87.

Engesser, F., (1889), ''Die Knickfestigkeit gerader Sta¨be,'' Z. Architekten und Ing. Verein zu Hannover, 35, p. 455.

Engesser, F., (1889), ''Die Knickfestigkeit gerader Sta¨be,'' Zentralblatt des Bauverwaltung., 11, p. 483–486.

Ramesh, S., Kreger, M., and Bowman, M. (2013). "Behavior and design of earthquake-resistant dual-plate composite shear wall systems." Final report to the Charles Pankow Foundation.

Sener, K., and Varma, A.H. (2014). "Steel-Plate Composite SC Walls: Experimental Database and Design for Out-of-Plane Shear." Journal of Constructional Steel Research, Elsevier Science, Vol. 100, pp. 197-210.

Sener, K., Varma, A.H., and Seo, J. (2015). "Experimental and numerical investigation of the shear behavior of steel-plate composite (SC) beams without shear reinforcement." Engineering Structure, Vol. 127, pp. 495-509.

Varma, A.H., Malushte, S., Sener, K., and Lai, Z., (2014). "Steel-Plate Composite (SC) Walls for Safety Related Nuclear Facilities: Design for In-Plane Force and Out-of-Plane Moments." Nuclear Engineering and Design, Special Issue on SMiRT-21 Conference, Vol. 269, pp. 240-249

Varma, A. H., Malushte, S. R., Lai, Z., (2015) "Modularity & innovation using steel-plate composite (sc) walls for nuclear and commercial construction." 11th International Conference on Advances in Steel and Concrete Composite Structures Tsinghua University, Beijing, China, December 3-5,