Wind Design of Composite Plate Shear Walls/Concrete Filled (SpeedCore) Systems

Soheil Shafaei, Amit H. Varma, Jungil Seo, Devin Huber, and Ron Klemencic

ABSTRACT

Composite steel plate shear walls/concrete filled (C-PSW/CF), also known as the SpeedCore system, are a composite solution for the design of mid- to high-rise buildings. A C-PSW/CF system consists of steel plates (web and flange plates) and an infill concrete core. The composite interaction between steel plates and concrete core is developed by either tie bars or tie bars and shear studs. Generally, in low- to mid-rise buildings (less than 15 stories), planar (uncoupled) C-PSW/CFs are adequate for resisting lateral loading and deformations. Coupled C-PSW/CFs become more prevalent in mid- to high-rise buildings when increased lateral stiffness is desirable. The steel parts of SpeedCore system (steel plates, tie bars, and shear studs) are prefabricated in the shop and then transported to the site for assembly and erection. The erected empty steel module serves as falsework and formwork during construction and concrete casting, which reduces construction schedule considerably. This is a major advantage of SpeedCore systems, especially when compared with conventional reinforced concrete core systems. This paper presents the wind (nonseismic) design requirements and procedures for planar uncoupled and coupled C-PSW/CF systems. It includes a design example of a 15-story building located in Chicago, Illinois, using both uncoupled and coupled systems.

Keywords: composite plate shear walls/concrete-filled; SpeedCore system; uncoupled C-PSW/CF; coupled C-PSW/CF; wind design; nonseismic design, cyclic behavior of C-PSW/CF.

INTRODUCTION

The selection and proper specification of a lateral force-resisting system (LFRS) is one of the most critical design decisions for mid- to high-rise buildings. One popular lateral system used in both steel and reinforced concrete construction is a coupled reinforced concrete (RC) core wall system. As the overall structural height increases, the required lateral stiffness increases. For "signature" tall buildings—for example, 432 Park and Stenway Tower in New York City—the RC concrete wall thickness and reinforcement ratio can become very high (greater than 2%), leading to rebar congestion, concrete placement issues, and potential schedule delays. Composite plate shear walls/concrete filled (C-PSW/CF), also known as the SpeedCore system, offer a steel-concrete composite solution to such concrete core walls. Planar (uncoupled) or coupled C-PSW/CF systems offer several advantages compared to traditional RC core walls in mid- to high-rise buildings, including:

1. Steel modules are prefabricated in the shop and assembled together once at site, which reduces the construction schedule associated with assembling rebar cages.
2. Empty steel modules of C-PSW/CF, including steel web and flange plates, tie bars, and shear studs, serve as falsework and formwork during the construction and concrete casting.
3. Steel web and flange plates serve as permanent formwork and steel reinforcement of the composite wall after concrete sets, which further helps reduce the construction schedule by eliminating an additional step for form removal.
4. Rebar congestion issues are reduced by eliminating rebar cages.
5. Easier concrete placement is allowed by using conventional or self-consolidating concrete (SCC).
6. Core walls can be constructed at the same pace as steel framing. Unlike RC construction, there is no need to construct the core much further ahead of the steel framing. This reduces the number of trades that need to be coordinated at the site and also reduces construction tolerance issues with embed plates connecting the steel framing to the core.

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7. Higher reinforcement ratios (if needed) are achieved by using steel plates.

C-PSW/CF systems can have some challenges as well. For example:

1. The minimum steel plate thickness must be limited to ⅜ in. due to practical considerations of handling, fabrication, and transportation.
2. The steel fabrication costs may be larger than those in RC construction.
3. In-situ welding requirements may be a limitation in some locations due to costs or safety.
4. Design criteria and guidance are not yet available to the profession.

This paper provides an introduction as well as clarity on the nonseismic design of uncoupled (planar) and coupled C-PSW/CF systems when subjected to wind loading. Similar guidance for uncoupled C-PSW/CF systems subjected to seismic loading is available in AISC *Seismic Provisions* (2016a) Section H7 and Agarwal et al. (2020). Seismic design requirements and procedure for coupled systems are also provided in the FEMA P-2082 (FEMA, 2020) provisions and through expected changes in the 2022 edition of the AISC *Seismic Provisions* (AISC, 2022a) and the 2022 edition of ASCE 7 (ASCE, 2016; Bruneau et al., 2019; Broberg et al., 2022). Similarly, revisions to the 2022 edition of the AISC *Specification* (AISC, 2022b) will aim to address wind design of C-PSW/CF systems. However, no other literature is currently readily available for the wind design of uncoupled and coupled C-PSW/CF systems, and this paper serves to provide the interim guidance for this subject.

BACKGROUND

Composite plate shear walls/concrete filled (C-PSW/CF) consist of steel modules that are filled with plain concrete. Figure 1 shows a typical representation of a planar (uncoupled) C-PSW/CF panel. The modules consist of two exterior steel web plates that are connected to one another using tie bars, threaded rods, or other steel shapes, and there are steel flange plates (closure plates) at the end. The composite interaction between the steel plates and infill concrete is developed by tie bars or by a combination of tie bars and shear studs. Note that shear studs are not a specific requirement of the system, but they can be used to reduce the total number of ties in a C-PSW/CF.

The empty steel modules of C-PSW/CF consisting of steel plates (webs and flanges), tie bars, and shear studs are prefabricated in the shop and then shipped to the site for final assembly and concrete casting. The modules get shipped to the site "empty" (no concrete fill), are assembled and connected together to other panels into a final configuration, and then concrete is placed into the panels. The empty modules are stacked to a certain height, typically two to three stories, for each placement of concrete, and the process is repeated as the structure is constructed. A photo of an assembled empty module with coupling beams being placed at a site is shown in Figure 2. Generally, the empty steel modules come without painting, but after assembly, they might be fireproofed or painted, if needed (Anvari et al. 2020).

The steel plates act compositely with the hardened concrete using different configurations of tie bars, tie bars and shear studs, or other types of anchorage (i.e., steel shapes). These regularly spaced steel anchors and tie bars restrain local buckling of the steel faceplates, provide

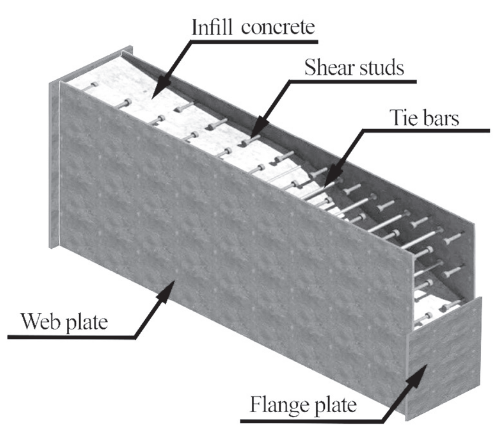


Fig. 1. Typical details of planar (uncoupled) C-PSW/CF (Shafaei et al., 2021a).

horizontal shear resistance, and develop composite action between the steel plates and the concrete infill (Zhang et al., 2014). Additionally, the tie bars (1) provide out-of-plane shear resistance, (2) provide confinement for concrete, and (3) resist potential delamination failure mode through the plain concrete (Sener and Varma, 2014).

C-PSW/CF systems do not include any conventional reinforcing steel of the plain concrete infill. The steel plates serve as the primary reinforcement for resisting all in-plane forces and moments as well as out-of-plane flexure. Out-of-plane shear resistance is provided by the concrete infill and the tie bars (Sener and Varma, 2014) and interfacial shear resistance by the steel anchors and tie bars (Zhang et al., 2014; Sener et al., 2016).

The lateral load behavior of planar C-PSW/CF systems, including their lateral force-deflection behavior and ductility, has been experimentally and numerically evaluated by Shafaei et al. (2021a, 2021b). Research on the lateral load behavior of coupled C-PSW/CF core wall structures is limited with studies ongoing by the authors. The behavior of C-PSW/CF subjected to cyclic lateral loading was investigated by Ramesh (2013). The study focused on (1) the stability of the C-PSW/CF module for construction loads and (2) the cyclic performance of welded details and other structural elements. The specimens were subjected to limited inelastic deformations prior to failure of welded details. The findings were limited to specific wall and faceplate thicknesses, tie bar ratio and spacing, and T-shaped intersections. Additional research is needed and is ongoing to investigate the behavior of planar and coupled C-PSW/CF as explained in the following section.

COMPOSITE PLATE SHEAR WALLS/CONCRETE FILLED (C-PSW/CF)

In mid- to high-rise buildings, planar (uncoupled) or coupled C-PSW/CF systems can be selected based on the architectural plan to resists the lateral loads. Figure 3 shows several examples of typical planar and coupled C-PSW/CF core wall structures.

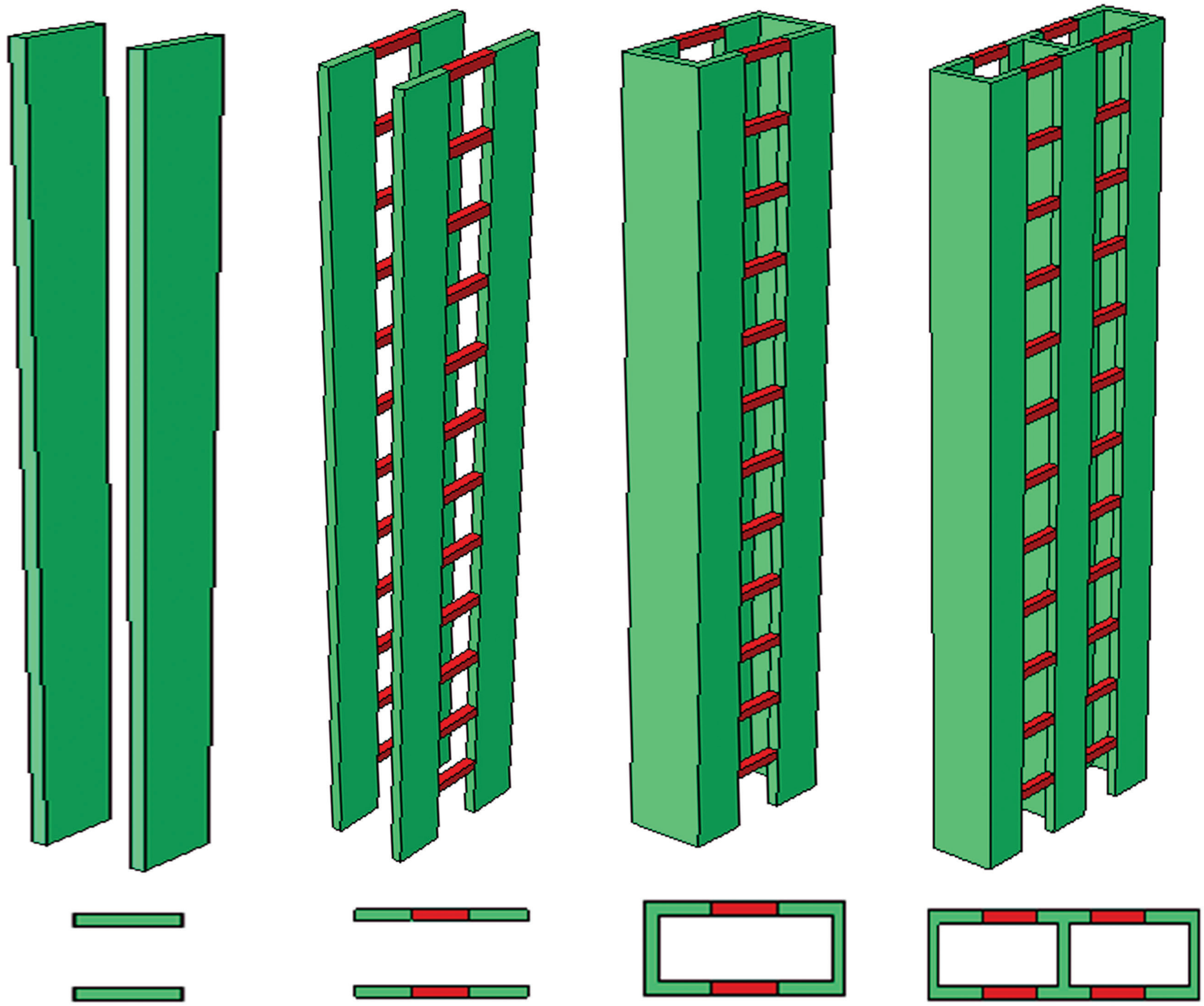
As shown in Figure 3, coupled systems consist of two (or more) individual C-PSW/CF connected together by coupling beams (link beams) along the height of the structure. Individual C-PSW/CF with planar, C, U, I, or T shapes are utilized to make coupled CPSW/CF core wall systems.

In higher seismic areas, the use of composite coupling beams is more common due to the large forces to be transferred and to meet ductility requirements for performing a capacity-based design. However, coupling beams using all-steel rolled shapes or built-up shapes can be more readily utilized in nonseismic areas where they can be sized to resist only the applied lateral loading, as discussed in subsequent sections.

At the time of publication, research by the authors is ongoing to investigate behavior of coupling beams in both seismic and nonseismic (i.e., wind-governed) regions. Configurations being investigated for the coupling beam research include standard wide-flange structural shapes, steel-only built-up sections, and composite steel-concrete built-up box shaped beams.

There are no specific ductility requirements or performance criteria for connections in nonseismic design (wind-governed design) of C-PSW/CF systems. Different types of

Fig. 2. Assembled coupled C-PSW/CF module being lifted into place at Rainier Square Project in Seattle, Washington (photo courtesy of Keith Evans/ENR).



(a) Planar walls (b) Coupled planar walls (c) Coupled C-shaped walls (d) Coupled C-shaped and I-shaped wall

Fig. 3. Variations of composite plate shear walls/concrete-filled.

welded or bolted connections at the section, member, and structure levels can be designed according to the AISC *Specification* (2016b) Chapter J requirements. Tie bar-to-steel plate, flange-to-web plate, and wall segment connection are examples of connections at the section levels. Connections at the member level can be coupling beam-to-wall and wall splice connections, and connections at the structure level can be wall-to-foundation and floor-to-wall connections. In wind design of C-PSW/CF systems, the connections at the member or structure levels can be designed based on the required strength using AISC *Specification* Chapter J provisions.

LATERAL LOAD BEHAVIOR OF C-PSW/CF SYSTEMS

Figure 4 illustrates idealized responses of planar and coupled shear wall systems under wind lateral loading. The lateral load behavior of planar C-PSW/CF is totally different from coupled C-PSW/CF and is described herein.

Planar C-PSW/CF Response

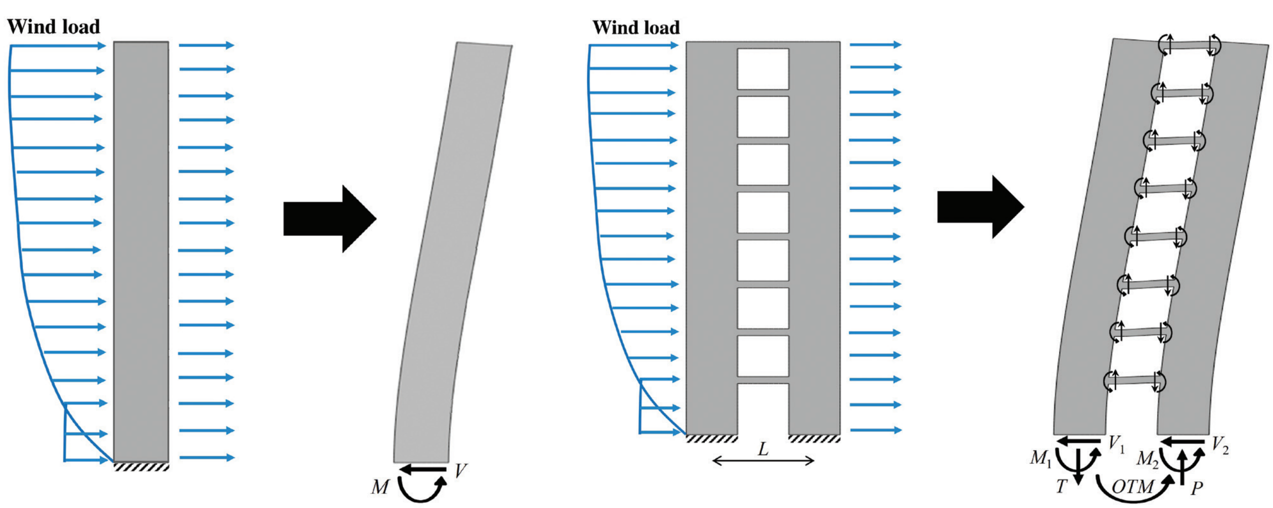
Planar C-PSW/CF structures are being considered in seismic and wind design of low- to mid-rise building (less than 15 stories) considering the architectural design. When a planar C-PSW/CF system is used, the planar wall resists the lateral load like a cantilever beam, as shown in Figure 4(a). The lateral load response of a planar C-PSW/CF system is governed by in-plane flexural behavior.

Coupled C-PSW/CF Response

In seismic and wind design of mid- to high-rise buildings utilizing C-PSW/CF systems, coupled configurations are typically used. Figure 4(b) illustrates an idealized coupled shear wall system deformed under wind lateral loads, which cause a global system overturning moment (OTM) at the base. The coupled wall system resists the OTM by flexural moment resistances in the individual walls, M_1 and M_2, as well as by developing an axial force couple [$\Sigma V_{beam,j}$ at the lever arm, L, as shown in Figure 4(b)]. The contribution of the axial force couple, $\Sigma V_{beam,j}$, to resisting the OTM is defined as the coupling ratio (CR) and it can be calculated using Equation 1. A CR of 0% implies that no coupling beams are present or the two individual shear walls are disconnected. A CR of 100% is the theoretical case where the coupling beam length is zero.

$$CR = \frac{L\sum V_{beam,j}}{L\sum V_{beam,j} + M_1 + M_2} = \frac{L\sum V_{beam,j}}{OTM} \tag{1}$$

The axial force couple, $\Sigma V_{beam,j}$, is applied to the individual shear wall (in addition to gravity loads). The axial force, either compression, P, or tension, T, acting on the individual shear wall influences the behavior of the wall (El-Tawil et al., 2010). For example, the axial compression force acting on the individual shear wall reduces the ductility of the wall due to concrete crushing failure, and the axial tension force acting on the individual shear wall decreases the flexural stiffness and strength of the wall. Hence, in design of coupled systems, each individual wall (tension or compression wall) should be designed separately.



(a) Planar C-PSW/CF *(b) Coupled C-PSW/CF*

Fig. 4. Idealized shear wall systems deformed under wind loading.

The ratio of the coupling action is directly correlated with the flexural stiffness of coupling beams and C-PSW/CF. Hence, higher coupling beam flexural stiffness will result in higher coupling action and *CR*. Often, the coupling beam consists of concrete-filled built-up box sections or rectangular HSS since they have similar characteristics in cross section with a C-PSW/CF. However, steel coupling beams can be used for nonseismic applications (wind design).

DESIGN REQUIREMENTS FOR PLANAR AND COUPLED C-PSW/CF SYSTEMS

When this paper was in development, specific and prescribed design requirements for seismic composite plate shear walls existed in various literature, and these references are noted whenever possible. With respect to seismic design requirements, provisions for uncoupled C-PSW/CF currently exist in AISC *Seismic Provisions* (2016a) Section H7. An additional section, Section H8, is currently under development for coupled systems and is expected to be in the 2022 edition of AISC *Seismic Provisions*. Also currently under development are updates to the 2022 edition of the AISC *Specification* that would apply to building design not governed by seismic loads.

Presented in the following sections are design requirements for uncoupled and coupled C-PSW/CF systems to aid engineers in sizing various wall components. It is noted herein where these requirements exist in current provisions in the AISC *Seismic Provisions* or AISC *Specification*. Other recommendations provided, but not within the current specifications, are also noted. Many of these recommendations will become specific requirements in the 2022 editions of the AISC specifications, but they are still under review. Note, there are certain requirements with respect to both seismic and wind design that are the same regardless of what region the structure is located within. Similarly, between planar and coupled C-PSW/CF systems, some design requirements stay constant.

DESIGN REQUIREMENTS FOR C-PSW/CF

The following sections present requirements proposed in the 2022 AISC *Specification* for nonseismic design of C-PSW/CF.

Section Detailing of C-PSW/CFs

As proposed in the AISC *Specification*, the opposing steel web plates of C-PSW/CF have equal nominal thicknesses and are connected to each other using ties consisting of bars, structural shapes, or built-up members. For filled composite plate shear walls, the steel plates are anchored to the concrete using tie bars or a combination of tie bars and steel anchors (steel headed studs). The minimum steel reinforcement ratio of C-PSW/CF is 1%, and the maximum is 10% of the wall gross cross section. C-PSW/CF have steel flange plates (closure plates) or boundary elements at the end of the wall cross-section.

Slenderness Requirement

As proposed for the AISC *Specification*, the slenderness ratio of the steel plates is defined as b/t_p, where b is the largest spacing between rows of tie bars or stud anchors (if included). This slenderness ratio has a significant influence on the local buckling and compressive strength of the steel plate. Zhang et al. (2014, 2020) have experimentally and numerically evaluated the effects of steel plate slenderness ratio on the local buckling and axial compressive strength of composite walls. They established the local buckling classification criterion, shown here as Equation 2. When the plate slenderness ratio satisfies the inequality of Equation 2, then it can develop the nominal yield stress, F_y, before local buckling. It is important to note that the critical buckling stress of slender plates can be estimated using equations provided by Zhang et al. (2020), which are not included here.

$$\frac{b}{t_p} \leq 1.2\sqrt{\frac{E}{F_y}} \tag{2}$$

where

E_s = elastic modulus of steel plate, ksi

F_y = yield stress of steel plate, ksi

b = largest clear distance between rows of tie bars or stud anchors, in.

t_p = steel plate thickness, in.

Tie Bar Requirement

As proposed for the AISC *Specification*, C-PSW/CF need to maintain the structural integrity and prevent concrete splitting of the wall section. For nonseismic applications, there is no minimum tie bar strength requirement, as the tie bar requirement of the empty module governs the design. The empty steel module requirements apply to the steel shell prior to concrete casting. These requirements relate to the required effective shear stiffness, GA_{eff}, of the empty module, which governs the structural behavior and stability during construction and concrete casting (Varma et al. 2019). This limit also ensures that concrete casting pressure and other construction loads do not excessively deform the steel modules prior to concrete curing. This limit is presented in Equations 3 and 4.

$$\frac{s}{t_p} \leq 1.0\sqrt{\frac{E_s}{2\alpha+1}} \tag{3}$$

$$\alpha = 1.7\left(\frac{t_{sc}}{t_p} - 2\right)\left(\frac{t_p}{d_{tie}}\right)^4 \quad (4)$$

where

d_{tie} = effective diameter of the tie bar, in.

s = largest clear spacing of ties, in. (vertical)

t_p = steel plate thickness, in.

t_{sc} = thickness of composite plate shear wall, in.

Stiffness of C-PSW/CF

As proposed for the AISC *Specification*, the flexural, axial, and shear stiffnesses of composite plate shear walls account for the extent of concrete cracking corresponding to the required strength. The effective flexural, axial, and shear stiffnesses are calculated using Equations 5, 6, and 7, respectively.

$$EI_{eff} = E_s I_s + 0.35 E_c I_c \quad (5)$$

$$EA_{eff} = E_s A_s + 0.45 E_c A_c \quad (6)$$

$$GA_{v.eff} = G_s A_{sw} + G_c A_c \quad (7)$$

where

A_c = area of concrete, in.^{2}

A_s = area of steel section, in.^{2}

A_{sw} = area of steel plates in the direction of in-plane shear, in.^{2}

E_c = modulus of elasticity of concrete
= $w_c^{1.5}\sqrt{f_c'}$, ksi

E_s = modulus of elasticity of steel
= 29,000 ksi

G_c = shear modulus of concrete
= $0.4E_c$

G_s = shear modulus of steel
= 11,200 ksi

I_c = moment of inertia of the concrete section about the elastic neutral axis of the composite section, in.^{4}

I_s = moment of inertia of steel shape about the elastic neutral axis of the composite section, in.^{4}

w_c = weight of concrete per unit volume ($90 \le w_c \le 155$ lb/ft^3)

Strength of C-PSW/CF

As proposed for the AISC *Specification*, the nominal flexural strength, $M_{n,wall}$, of planar C-PSW/CF can be calculated using the plastic stress distribution method or using a cross-section fiber-based analysis. A strength reduction factor, ϕ_b, of 0.90 is considered in the calculation of the design flexural strength, $\phi M_{n,wall}$, of C-SPW/CF. In the calculation of the nominal flexural strength using the plastic stress distribution method, it is assumed the steel plates reach the yield stress, F_y, in both tension and compression and the infill concrete core develops compression stress equal to $0.85f_c'$.

Alternatively, the nominal flexural strength, $M_{n,wall}$, can be computed using fiber cross-section analysis. In fiber analysis, the influence of the axial force, P, from gravity load and induced by the coupling action on the wall flexural strength should considered. Hence, the flexural strength of individual wall subjected to either axial tension or compression force is directly calculated using a fiber analysis.

As proposed for the AISC *Specification*, the nominal in-plane shear strength of C-PSW/CF, $V_{n,wall}$, is computed using Equation 8. In the equation, F_y is the nominal yield stress of the steel wall plate, and A_{sw} is the steel web plate area. A strength reduction factor, ϕ, of 0.90 is used to calculate the design shear strength, $\phi V_{n,wall}$, of C-PSW/CF.

$$V_{n,wall} = \frac{K_s + K_{sc}}{\sqrt{3K_s^2 + K_{sc}^2}} A_{sw} F_y \quad (8)$$

where

$$K_s = G_s A_{sw} \quad (9)$$

$$K_{sc} = \frac{0.7(E_c A_c)(E_s A_{sw})}{4E_s A_{sw} + E_c A_c} \quad (10)$$

The recomendations given herein for the axial load strength of CPSW/CF systems are based on the following requirements being met:

1. The steel plates comprise at least 1% of the total composite cross-sectional area.
2. The steel plates satisfy the slenderness requirements noted previously.

As proposed for the AISC *Specification*, the nominal compressive strength of axially loaded composite plate shear walls is determined for the limit state of flexural buckling. The value of flexural stiffness, EI_{eff}, from Equation 5 is used along with the section axial compressive strength, P_{no}, calculated using Equation 11. The unsupported length for flexural buckling of composite walls is typically assumed to be equal to the story height.

$$P_{no} = F_y A_s + 0.85 f_c' A_c \quad (11)$$

As proposed for the AISC *Specification*, the design tensile strength of axially loaded C-PSW/CF is determined by the limit state of steel plate yielding in accordance with Equation 12.

$$P_n = A_s F_y \quad (12)$$

DESIGN REQUIREMENTS FOR COUPLING BEAMS

For nonseismic applications, composite steel-concrete or steel beams (wide flange steel beams or built-up sections) can be used for the coupling beams of coupled C-PSW/CF systems. Composite coupling beams can be concrete-filled built-up box sections or rectangular HSS sections. For seismic design, the coupling beams are proportioned to be flexure critical (flexural governing behavior); however, for wind design, the coupling beams are designed to resist the demand forces and moments. The requirements for composite coupling beams are based on those in the AISC *Specification* Chapter I as identified in the following subsections.

Section Detailing of Filled Composite Coupling Beams

As required by AISC *Specification* Section I2.2a, the cross-sectional area of the steel section should comprise at least 1% of the total composite cross section. This minimum area of steel requirement is based on AISC *Specification* Section I2 requirements for composite members.

Slenderness Requirement

As proposed for the AISC *Specification*, in composite coupling beams, width-to-thickness of steel plate limits are critical since the structural performance of coupled C-PSW/CF system is directly related to the behavior of the coupling beams. Per AISC *Specification* Section I1.4 (Table I1.1b), the width-to-thickness limits are used to classify the coupling beams as compact, noncompact, or slender for flexure. For wind (nonseismic) design, there is no requirement to select exclusively a compact section as compared to seismic design.

Stiffness of Filled Composite Coupling Beams

As required by AISC *Specification* Section I2.2b, the effective flexural stiffness, EI_{eff}, of the filled composite coupling beams is calculated using Equation 13. The term C_3 in Equation 13 is the coefficient to calculate the effective rigidity of a filled composite member in compression, and it is calculated using Equation 14. For the axial stiffness, EA, the gross area of the coupling beam is used. If the structural analysis is based on the direct analysis method, flexural and axial stiffness values of the composite concrete-filled coupling beams are reduced to $0.64EI_{eff}$ and $0.8EA$.

$$EI_{eff.CB} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \tag{13}$$

$$C_3 = 0.45 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \le 0.9 \tag{14}$$

Strength of Filled Composite Coupling Beams

As required by the AISC *Specification* Section I3.4b, the nominal flexural strength of composite coupling beams, $M_{n,CB}$, is determined according to AISC *Specification* Section I3.4b using the plastic stress distribution method. A strength reduction factor, ϕ, of 0.90 is considered in the calculation of the design flexural strength of composite coupling beams, $\phi M_{n,CB}$.

As proposed for the AISC *Specification*, the nominal shear strength of composite coupling beams, $V_{n,CB}$, is calculated as the design shear strength of the steel section and the concrete infill according to Equation 15, where A_v is the area of steel webs and A_c is the cross-sectional area of the concrete. A strength reduction factor, ϕ, equal to 0.90 is considered in the calculation of the design shear strength of concrete-filled coupling beams, $\phi V_{n,CB}$.

$$V_{n.CB} = 0.6 A_v F_y + 0.06 K_c A_c \sqrt{f_c'} \tag{15}$$

Requirements for Steel Coupling Beams

The use of steel coupling beams is permitted in wind (nonseismic) applications of C-PSW/CF systems. The design of steel coupling beams for flexure and shear is conducted in accordance with AISC *Specification* Chapter F (Design of Members for Flexure), Chapter G (Design of Members for Shear), and Chapter J (Design of Connections), respectively.

CONNECTION REQUIREMENTS

The behavior and design of SpeedCore systems for nonseismic, or wind-governed, loading combinations have no explicit ductility or performance requirements for the connections beyond those of adequate strength for the design demands (required strengths) calculated from the analysis for the applicable factored load combinations. Welded and/or bolted connections are designed in accordance with the requirements of AISC *Specification* Chapter J.

Connections of the tie bars to the steel plate are designed to develop the yield strength of the tie bar in axial tension. This enables yielding of the tie bars before failure of the tie-to-plate connection.

Connections at the member and structure levels are designed in accordance with the calculated design demands (required strengths) at the corresponding levels and location. Some examples of the different types of connections in the systems include (1) coupling beam-to-composite wall connections, (2) composite wall-to-foundation or subgrade structure connections, and (3) splice connections in composite walls.

PRACTICAL AND CONSTRUCTION CONSIDERATIONS OF C-PSW/CF SYSTEMS

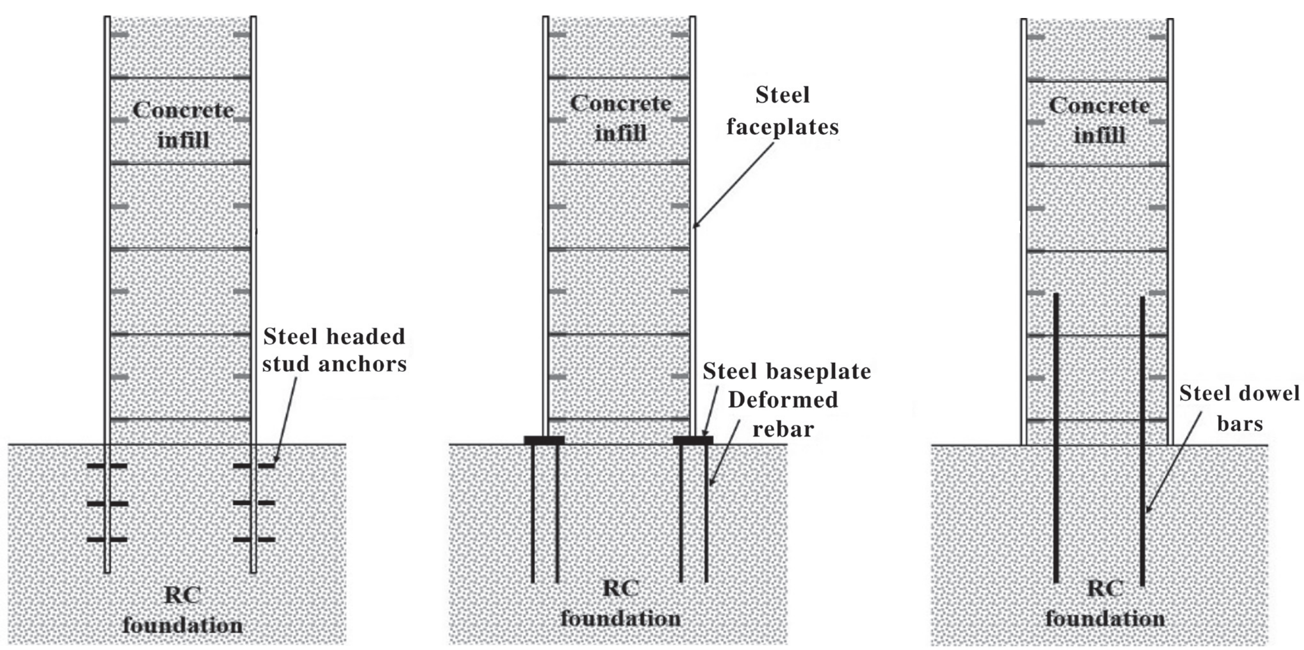
In many cases, the plate thicknesses for planar or coupled C-PSW/CF modules govern the design based on construction considerations related to welding requirements or the stability of the empty steel modules during the construction and concrete casting. Previous experience in using these panel type for the Rainier Square Tower in Seattle resulted in minimum plate thicknesses used of ½ in., regardless of demands on the panels. This was ultimately a fabricator and erector preference and is the limit they felt could be practically handled in their particular situation. Additionally, the governing equations presented earlier relate to limits for the empty steel modules, which highlighted that the empty module stability is tied closely to the effective shear stiffness of the empty modules.

There is no prequalified seismic concrete-filled composite coupling beam to wall connection, but research is ongoing on this topic by the authors. Designing the coupling beam-to-wall connection for seismic applications is challenging since the connection should provide specific ductility and rotation capacity. By contrast, for the nonseismic design, the design strength of the coupling beam-to-wall connection should only be equal to, or greater than, the required forces of the coupling beam.

In seismic applications, the structural performance (stiffness and ductility) of uncoupled or coupled C-PSW/CF systems is based on an assumption that the wall-to-foundation connections do not fail prior to the formation of plastic hinges in the walls and coupling beams. By contrast, the nonseismic design of the wall-to-foundation connection of a planar (uncoupled) or coupled C-PSW/CF system is done in accordance with the required force demand. That is, the foundation connections need only be strong enough to resist the required moments and forces at the base of C-PSW/CF.

Although the design strength of the wall-to-foundation connection should be equal to, or greater than, the required moments and forces, the rotational stiffness of the wall-to-foundation has a direct impact on the wind design of uncoupled and coupled C-PSW/CF. As uncoupled C-PSW/CF systems behave structurally like a cantilever beam, the assumed rotational stiffness of the wall-to-foundation connection is critical in estimating the overall drift of the structure. This type of structural behavior leads to lateral deformation limits governing most wind design cases of uncoupled C-PSW/CF systems. Comparatively, in coupled C-PSW/CF, the rotational stiffness of the foundation connection has an effect on the force or moment distribution along the height and lateral deformation, as the system resists the lateral loads by developing coupling action.

An investigation into wall-to-foundation connections for wind applications is ongoing by authors. Three potential wall-to-foundation connection options are illustrated in Figure 5 (JEA, 2005). As shown in Figure 5(a), the steel plates can be continued and embedded into the concrete foundation using stud anchors welded to the steel faceplates. The



(a) Embedding (b) Anchoring (c) Lap-splicing

Fig. 5. Three different anchoring techniques (adapted from JEA, 2005).

steel plates can also be welded to baseplates with anchor bars that are embedded into the concrete foundation, as shown in Figure 5(b). Alternatively, steel dowel bars can be used to transfer the forces from the C-PSW/CF to the reinforced concrete foundation, as shown in Figure 5(c). Selection of the anchoring option depends on the required strength for the connection and constructor preferences.

WIND DESIGN PROCEDURE FOR C-PSW/CF

The wind design procedure for uncoupled and coupled C-PSW/CF systems is described in the following sections. The requirements presented in the previous sections and applicable building codes are used to develop the wind design procedure.

Wind Design Procedure for Uncoupled C-PSW/CF

This section presents a design procedure for the wind application of uncoupled (planar) C-PSW/CF systems. Figure 6 shows a flowchart summarizing the general wind design procedure for planar C-PSW/CF. A description for each step of the uncoupled C-PSW/CF design process is presented next.

Step 1: General Information of the Considered Building

In this step, the initial design information such as building location, geometry, architectural requirements, number of stories, story height, floor dimension, building importance, material properties, etc., is collected.

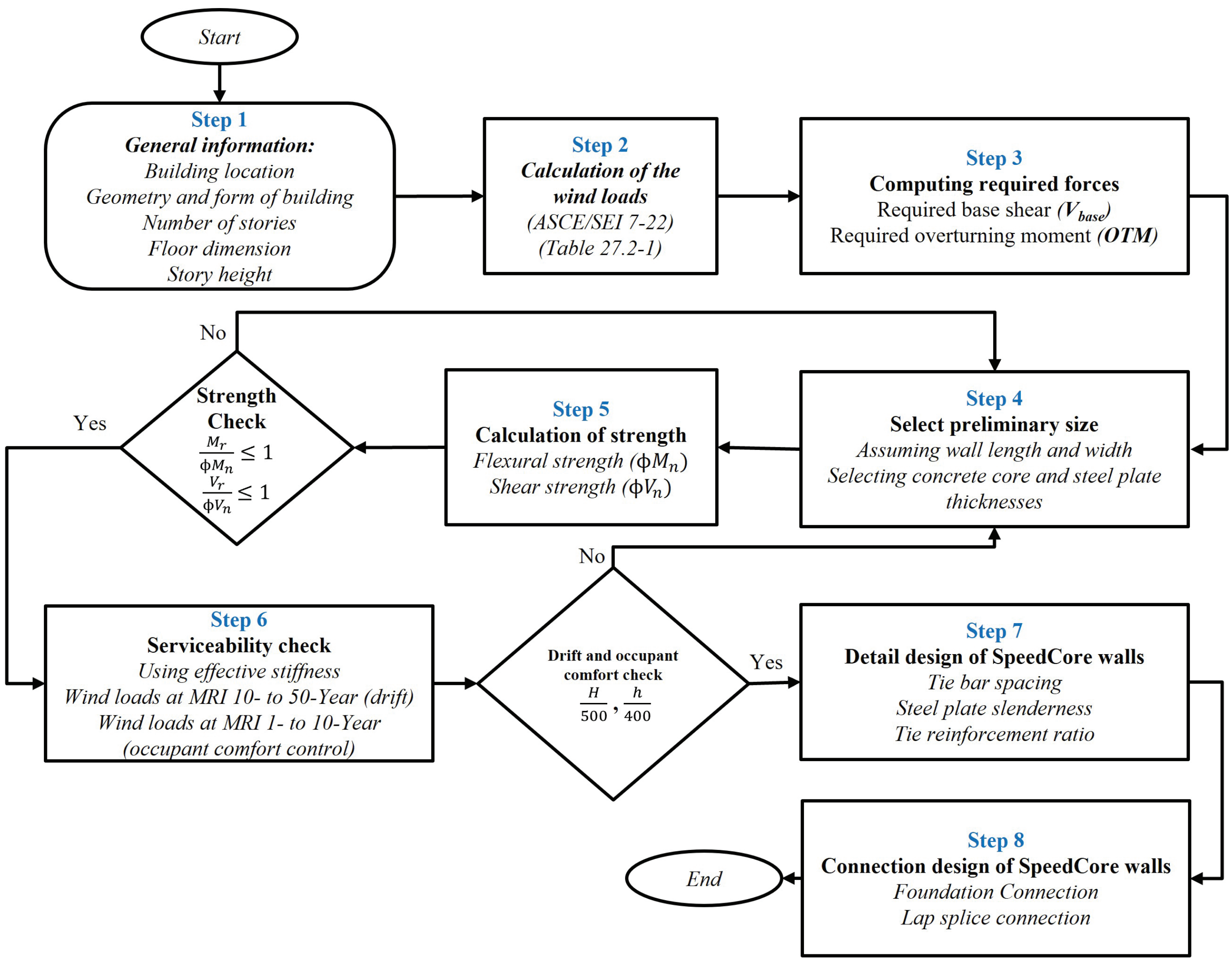


Fig. 6. Flowchart showing general wind design procedure for planar C-PSW/CF.

Step 2: Calculation of the Wind Loads

The C-PSW/CF are designed to resist the code-specified wind loads. In the absence of any wind tunnel testing or other special requirements for the given structure, design wind loads shall be determined based on the location and geometry (form)/building envelope according to ASCE 7, *Minimum Design Loads for Buildings and Other Structures* (2016). The wind loads are determined using the seven-step procedure (simplified procedure) provided in Table 27.2-1 of ASCE 7-16. Basic wind speed for the considered risk category is selected according to ASCE 7-16, Figure 26.5.1-B, to calculate wind loads for the strength design. Additionally, wind loads at mean recurrence interval (MRI) of 10-year or 50-year wind speed are also used to check the drift and occupant comfort checks. It is recommended that drift control of $H/500$ (roof total drift) and $h/400$ (maximum interstory drift) at 10-year MRI be used for the drift check of SpeedCore systems.

Step 3: Calculation of Base Shear and Overturning Moment

The design wind loads determined in Step 2 for the specific risk category are used to calculate the demand base shear and overturning moment (OTM). For uncoupled C-PSW/CF, the required base shear and OTM can be calculated by hand without conducting computational analysis.

Step 4: Select Preliminary Size for the Uncoupled C-PSW/CF

The size of uncoupled C-PSW/CF components is selected in this step considering the floor layout, architectural plan, and required base shear and moment. Components sized at this point include the total wall length, L_{wall}, wall thickness, t_{sc}, and steel plate thickness, t_p. No specific recommendations for the initial sizing are provided in this paper because there could be multiple drivers for sizing components, including architectural requirements, fabricator and erector preference, and designer experience with the system.

Step 5: Strength Check of Uncoupled C-PSW/CF

The flexural strength of the selected planar C-PSW/CF is calculated using one of the methods presented in the AISC *Specification* and compared with the required OTM. The shear strength is calculated using Equation 8 and is compared with the required base shear. If the strength check is not satisfied, the size of uncoupled C-PSW/CF should be revised, and the procedure restarts from Step 4. In design of mid- or high-rise buildings, the strength of planar C-PSW/CF is considerably higher than the demand forces as serviceability (drift limits) generally governs the design.

Step 6: Serviceability (Drift and Occupant Comfort) Check of Uncoupled C-PSW/CF

The lateral deflection is checked to evaluate the serviceability of the uncoupled C-PSW/CF system. The deflection limit for the wind loads is on the order of $1/600$ to $1/400$ of the building or story height (ASCE Task Committee on Drift Control of Steel Building Structures, 1988; Griffis, 1993). AISC Design Guide 3 (Fisher and West, 2003) recommends the roof lateral deflection due to wind loading be limited to $H/500$ using a 10-year mean recurrence interval (MRI) wind pressure, where H is the total height of the building, or $h/400$ for interstory drift, where h is the story height. A commercial finite element program can be used to calculate the roof displacement and interstory drift of uncoupled C-PSW/CF systems. The effective stiffnesses of planar walls presented in the AISC *Specification* are used to estimate the lateral deflection. The effect of the rotational stiffness of the wall-to-foundation connection is also considered in the calculation of lateral deflection. If the drift check is not satisfied, the size of uncoupled C-PSW/CF should be revised, and the procedure restarts from Step 4.

Step 7: Detail Design of Uncoupled C-PSW/CF

In this step, the tie bar spacing and tie bar diameters of planar C-PSW/CF are designed to satisfy the slenderness limits and tie reinforcement ratio requirements. Slenderness limits and tie bar requirements are presented in previous sections. It should be noted that the stability of the empty steel module governs the required tie bar diameter of C-PSW/CF.

Step 8: Connection Design of Uncoupled C-PSW/CF

Finally, after designing the uncoupled C-PSW/CF, the wall-to-foundation connection is designed based on the calculated required strengths. The composite wall splices are also designed based on the calculated demands (required strengths) at the corresponding locations. It is important to consider that when the wall-to-foundation connection is designed according to the demands, the correct rotational stiffness of the connection should be used in Step 6 to check the drift.

Wind Design Procedure for Coupled C-PSW/CF

This section describes the wind design procedure for coupled C-PSW/CF systems. Figure 7 shows a flowchart summarizing the general wind design procedure for coupled C-PSW/CF systems. Working through the noted steps will provide sizing for the walls and coupling beams. A description for each step of the coupled C-PSW/CF design process is presented next.

Step 1: General Information of the Considered Building

This step is similar to Step 1 for uncoupled C-PSW/CF. The building information is collected and compiled, which includes the building location, floor dimension, number of stories, and story height.

Step 2: Calculation of the Wind Loads

Step 2 for coupled C-PSW/CF is also similar to Step 2 for uncoupled C-PSW/CF discussed previously.

Step 3: Select Preliminary Sizes for C-PSW/CF and Coupling Beams

The C-PSW/CF and coupling beams cross-sections are preliminarily sized in this step considering floor layout and architectural design. This includes selecting the wall length, L_{wall}, wall thickness, t_{sc}, steel plate thicknesses, t_p, and infill concrete core thickness. In addition, the coupling beam depth, h_{CB}, width, b_{CB}, web plate thickness, $t_{pw.CB}$, flange plate thickness, $t_{pf.CB}$, and length, L, are also selected in this step. Recommendations for sizing are not included in this paper because engineering experience and judgement are needed for selecting initial sizes.

Step 4: Calculation of Required Forces

The distribution of forces along the height of coupled C-PSW/CF is directly influenced by the stiffnesses of the C-PSW/CF walls and coupling beams. A finite element model of the coupled C-PSW/CF system is developed using the effective stiffnesses for the composite walls and coupling beams. A commercial structural analysis program can be used to conduct the analysis and calculate the design demands (also referred to as required strengths).

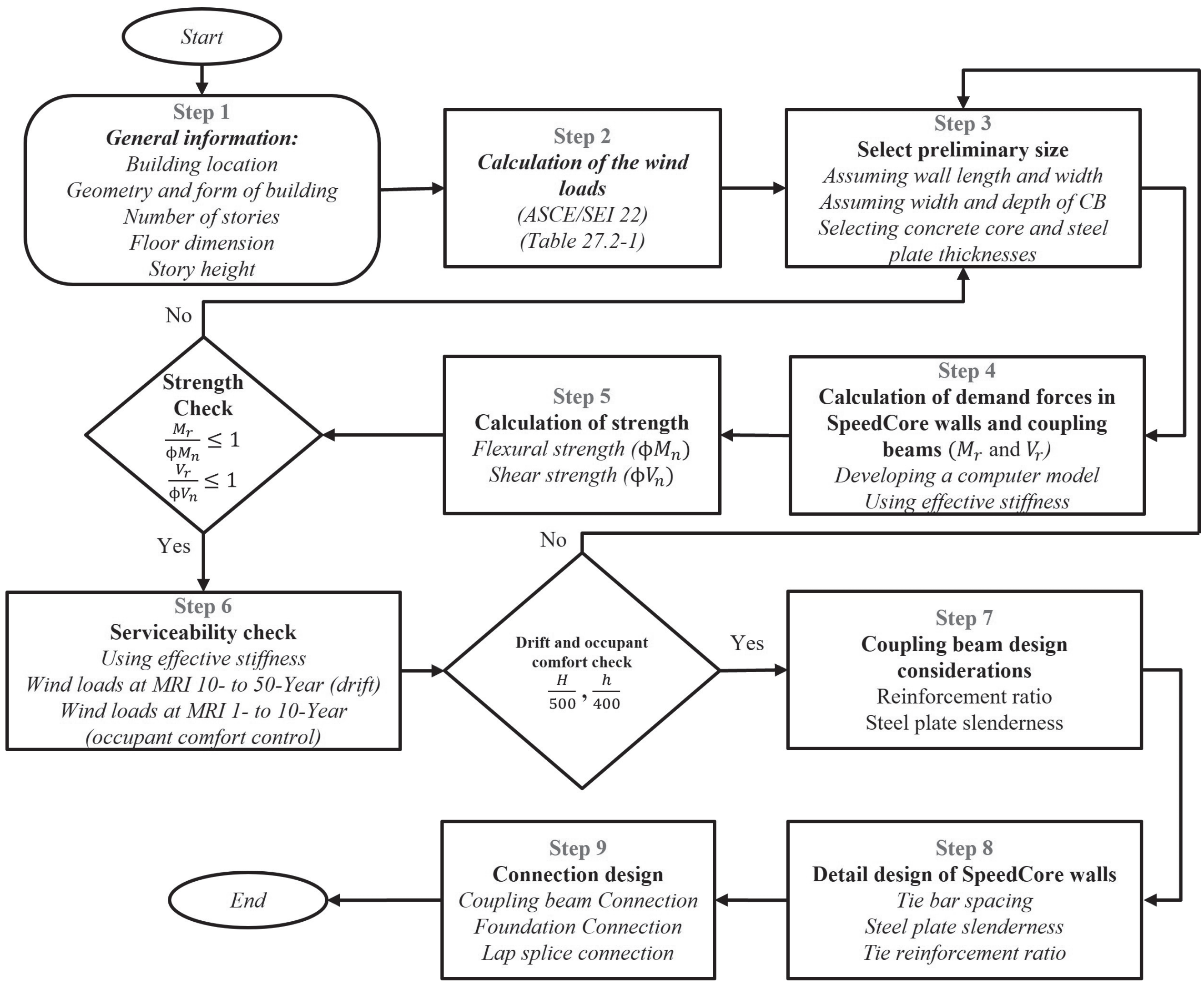


Fig. 7. Flowchart showing general wind design procedure of coupled C-PSW/CF.

Step 5: Strength Check of C-PSW/CF and Coupling Beams

The flexural strength of the selected C-PSW/CF is calculated and compared with the required OTM. The shear strength is calculated using Equation 8 and is compared with the required base shear. Additionally, the design flexural and shear strengths of coupling beams are calculated as outlined in the "Design Requirements for Coupling Beams." If the strength check is not satisfied, the size of coupling beams or C-PSW/CF are revised, and the procedure restarts from Step 4. In the design of mid- to high-rise buildings, the coupling beam strength typically governs the design, and the strengths of C-PSW/CF are considerably higher than the demand forces. In some cases, the lateral drift limits for wind loading govern the design of coupled C-PSW/CF.

Step 6: Serviceability (Drift and Occupant Comfort) Check of Coupled C-PSW/CF

A structural analysis is conducted in this step using a mean recurrence interval (MRI) of 10-year wind loads to check the drift limits. Appropriate effective stiffnesses of the composite walls and coupling beams are used in the structural analysis models of the coupled C-PSW/CF. The lateral deflection due to wind loading is limited to $H/500$, where H is the total height of the building, and the story drift is limited to $h/400$, where h is the story height. Although the rotational stiffness of the wall-to-foundation connection has a small effect on coupled systems due to the benefits of coupling action, the connection stiffness can be included to improve the estimation of lateral displacements. If the roof displacement or interstory drift requirements are not met, the sizes of members (C-PSW/CF or coupling beams) are revised, and the procedure restarts from Step 3.

Step 7: Detail Design of Coupling Beams

The section slenderness requirements and limits for the coupling beams are checked. These slenderness limits were identified previously. The geometric size and details of the coupling beams are used to assess coupling beam-to-wall connection possibilities.

Step 8: Detail Design of C-PSW/CF

Tie bar size (diameter) and spacing of planar C-PSW/CF are designed to satisfy the slenderness limits and tie reinforcement ratio requirements outlined previously.

Step 9: Connection Design

The composite wall-to-foundation (or subgrade structure) connections are designed based on the calculated demands (required strengths). The coupling beam-to-composite wall connections are designed based on the calculated demands (required strengths) at the corresponding levels. The composite wall splices are also designed based on the calculated demands (required strengths) at the corresponding locations.

Wind Design Example of 15-Story Planar and Coupled C-PSW/CF

The following design example presents the wind design procedure of planar (uncoupled) and coupled C-PSW/CF structures discussed in the previous sections. The structure considered in this design is a 15-story office building located in Chicago, Illinois (41.8847687° N, −87.6231634° W), where wind loads are expected to govern the design. Figure 8 illustrates the floor plan of the structure, which has a 200 × 120 ft footprint. The structure consists of two planar (uncoupled) C-PSW/CF in the East–West direction and two coupled C-CPSW/CF in the North–South direction. The first story height is 17 ft, and the typical story height is 14 ft. The total height of the structure is 213 ft. It is also assumed the floor loading is 120 psf (including superimposed dead loads), resulting in the total structural weight of 43,200 kips. In this example, the uncoupled C-PSW/CF system is designed first, followed by the coupled C-PSW/CF. The design assumed ASTM A992 steel for all steel shapes, ASTM A572, Grade 50 steel for all plates, and a Young's modulus of 29,000 ksi for all steel. The design also assumed normal weight concrete with a compressive strength of 6 ksi and Young's modulus of 4,415 ksi.

The wind loads for the considered building were determined using ASCE 7-16, Chapters 26–27. Table 1 shows the respective wind hazards and basic wind speeds for Chicago per ASCE 7-16. The wind pressures in the East–West direction are used to design planar (uncoupled) C-PSW/CF, and the wind pressures in the North–South direction are considered for the design of coupled C-PSW/CF. The main wind force-resisting system (MWFRS) of the building is designed for four different load cases presented in ASCE 7-16, Figure 27.3-8. The considered building satisfies the requirements in ASCE 7-16, Section D.1, as a torsionally regular building. Therefore, it only needs to be designed for Case 1, as shown in ASCE 7-16, Figure 27.3-8.

The wind loads were determined using the seven-step procedure provided in ASCE 7-16, Table 27.2-1. Table 2 presents a summary of wind pressures on the surface of the building determined using ASCE 7-16, Chapters 26–27, using the basic wind speed for risk category II. The wind pressures are multiplied with the exterior surface area (facades) of the building to calculate the wind loads. Wind loads calculated using wind speed for the given risk category are used for the strength design. Additionally, wind loads computed using a wind speed at the 10-year MRI are considered for checking the lateral deformation of the building to meet serviceability requirements.

Table 1. Basic Wind Speed for Buildings Located in Chicago	
Wind Hazard	**Basic Wind Speed**
MRI 10-year	74 mph
MRI 25-year	80 mph
MRI 50-year	85 mph
MRI 100-year	92 mph
Risk Category I	100 mph
Risk Category II	107 mph
Risk Category III	114 mph
Risk Category IV	119 mph

Table 2. Design Wind Pressure (Risk Category II) in the East–West and North–South Directions

Story No.	Story Height (ft)	K_z	q_z (psf)	External Pressure East–West Direction			External Pressure North–South Direction			Internal Pressure (psf)
				Windward (psf)	Leeward (psf)	Side (psf)	Windward (psf)	Leeward (psf)	Side (psf)	
15	213	1.63	40.7	28.3	–17.7	–24.8	28.9	–13.2	–25.2	±7.83
14	199	1.62	40.2	28.0	–17.7	–24.8	28.5	–13.2	–25.2	±7.83
13	185	1.59	39.7	27.7	–17.7	–24.8	28.2	–13.2	–25.2	±7.83
12	171	1.57	39.2	27.3	–17.7	–24.8	27.8	–13.2	–25.2	±7.83
11	157	1.55	38.6	26.9	–17.7	–24.8	27.4	–13.2	–25.2	±7.83
10	143	1.52	37.9	26.4	–17.7	–24.8	26.9	–13.2	–25.2	±7.83
9	129	1.50	37.3	25.9	–17.7	–24.8	26.4	–13.2	–25.2	±7.83
8	115	1.47	36.6	25.5	–17.7	–24.8	25.9	–13.2	–25.2	±7.83
7	101	1.44	35.8	24.9	–17.7	–24.8	25.3	–13.2	–25.2	±7.83
6	87	1.40	34.8	24.3	–17.7	–24.8	24.7	–13.2	–25.2	±7.83
5	73	1.36	33.8	23.5	–17.7	–24.8	23.9	–13.2	–25.2	±7.83
4	59	1.31	32.6	22.7	–17.7	–24.8	23.1	–13.2	–25.2	±7.83
3	45	1.25	31.1	21.6	–17.7	–24.8	22.0	–13.2	–25.2	±7.83
2	31	1.17	29.1	20.3	–17.7	–24.8	20.6	–13.2	–25.2	±7.83
1	17	1.05	26.2	18.3	–17.7	–24.8	18.6	–13.2	–25.2	±7.83

Wind Design of the Uncoupled C-PSW/CF System

This section presents the wind design of the planar (uncoupled) C-PSW/CFs in the East–West direction of the given building. The design of these uncoupled C-PSW/CFs is accomplished according to the design procedure presented in Section 6.

Step 1: General Information of the Considered Building

As shown in Figure 8, there are two uncoupled C-PSW/CF in the West–East direction to resist that wind loads. The length of the planar C-PSW/CF considered is 25 ft. The planar C-PSW/CF resist a small portion of gravity loads (dead load), as shown in Figure 8; therefore, the axial compression force due to gravity loads and C-PSW/CF self-weight is not considered in the design.

Step 2: Calculation of the Wind Loads

Basic wind speeds for risk category II and the 10-year MRI are used in calculation of the wind loads for the wind design (strength check) and serviceability control (drift check), respectively. For Chicago, the basic wind speed is 107 mph for Risk Category II, and the 10-year MRI is 74 mph, as shown in Table 1.

Step 3: Calculation of Base Shear and Overturning Moment

As there are two planar C-PSW/CF in the West–East direction, each wall resists one-half of the wind loads. The required base shear and overturning moment at the base of the planar C-PSW/CF are calculated either by hand calculations or from computer-aided structural analysis. The wind loads calculated for Risk Category II are used to calculate the demand base shear and OTM for uncoupled C-PSW/CF. The wall can be assumed like a cantilever beam subjected to wind loads, so the shear force and overturning moment are simply calculated by hand. The required base shear and moment for each planar wall are 490 kips and 6.96×10^5 kip-in., respectively.

Step 4: Select Preliminary Size for the Uncoupled C-PSW/CF

In this step, the preliminary size of the planar wall is selected. The uncoupled C-PSW/CF length and thickness are assumed to be 300 in. (25 ft) and 18 in., respectively. A steel plate thickness of ½ in. is selected mainly because of construction considerations, including welding requirements and the stability of empty modules during construction and concrete casting. Also, a thicker steel plate allows larger tie bar spacing, which results in more economical fabrication of modules. Figure 9 and Table 3 show the cross-section details of uncoupled C-PSW/CF in the West–East direction.

Step 5: Strength Check of Uncoupled C-PSW/CF

The design flexural and shear strengths, $\phi M_{n,wall}$ and $\phi V_{n,wall}$, respectively, of planar C-PSW/CF sections are calculated in this step. The flexural strength for the planar wall in this example is calculated for the cross section, assuming the plastic stress distribution method where all steel plates reach F_y and concrete in compression reaches $0.85f_c'$, while concrete in tension is assumed not to contribute to the strength. The shear strength of the wall is calculated using Equation 8. Table 4 presents the flexural and shear strength of each planar wall and the comparison with the demand. As shown in the table, the strength of the uncoupled C-PSW/CF is considerably higher than the required

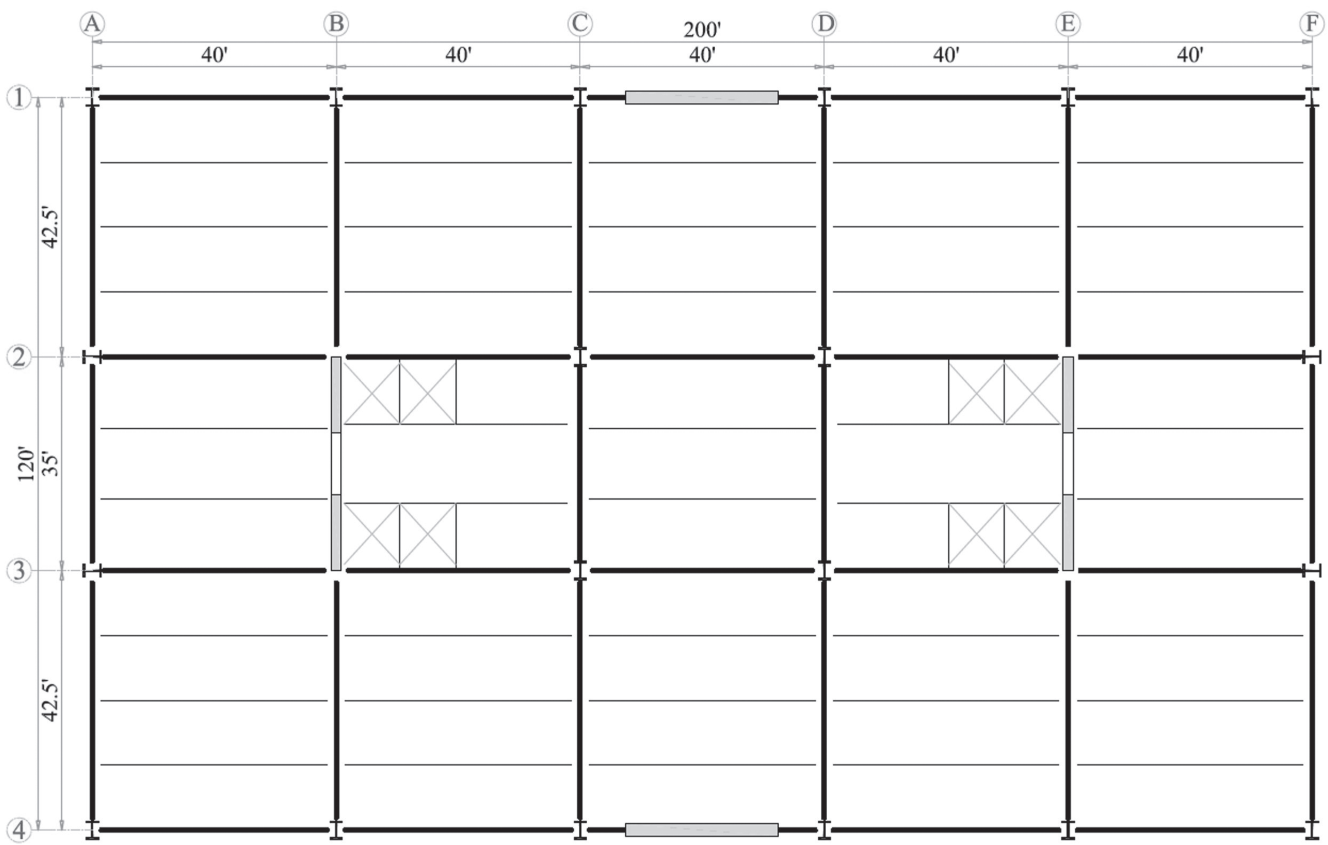


Fig. 8. Floor framing plan of the considered structure.

Table 3. Details of Uncoupled C-PSW/CFs in the West–East Direction				
Uncoupled C-PSW/CF	t_{pf}, in.	t_{pw}, in.	Length, in.	Wall Thickness, in.
	½	½	300	18

Table 4. Strength of Uncoupled C-PSW/CF and Comparison with Required Forces						
Uncoupled C-PSW/CF	$\phi M_{n,wall}$ kip-in.	$\phi V_{n,wall}$ kips	$M_{r,wall}$ kip-in.	$V_{r,wall}$ kips	$\frac{M_{r,wall}}{\phi M_{n,wall}}$	$\frac{V_{r,wall}}{\phi V_{n,wall}}$
	1.60×10^6	12,200	6.96×10^5	490	0.45	0.06

forces. It should be noted that the design of uncoupled C-PSW/CF structures is generally governed by drift limits; in other words, the flexural stiffness of planar C-PSW/CF governs the wind design. Again, the web and flange plate thicknesses of ½ in. are chosen due to construction considerations, including the welding of modules, stability of empty steel modules during erection, and concrete casting.

Step 6: Serviceability (Drift and Occupant Comfort) Check of Uncoupled C-PSW/CFs

Wind loads calculated using the wind speed at the 10-year MRI are used to check drift requirements. A finite element model, using a commercial software program (SAP2000) (CIS, 2017), of the planar C-PSW/CF can be utilized to check the serviceability and occupant comfort (drift limits). In order to estimate the lateral deflection of uncoupled C-PSW/CF, the effective stiffnesses of planar walls are used in the structural analysis. Additionally, the flexibility of the wall-to-foundation connection increases the lateral deflection; therefore, the rotational stiffness of foundation connection is considered in structural analysis for the drift check.

According to AISC *Specification* (2016b), Chapter B Commentary (Figure C-B3.3), a fully restrained connection shall provide a rotational stiffness at the support greater than or equal to 20 times the flexural stiffness, *EI*, divided by length of member. The required rotational stiffness, $K_{s.con}$, for a foundation connection to be considered fully restrained against rotation is calculated for a planar C-PSW/CF wall using Equation 16. The EI_{eff} given in the equation is the effective flexural stiffness of the uncoupled C-PSW/CF, and H is the total height of building.

$$K_{s.con} = \frac{20EI_{eff}}{H} \tag{16}$$

A two-dimensional finite element model of the planar C-PSW/CF for the given structure was developed using a commercial software program (using beam elements). The effective stiffness of the wall was used in the model, and both a completely fixed-base connection and a rotational spring with a stiffness calculated using Equation 16 were considered. Figure 10 shows the model output in terms of (1) the deformation shape, (2) story displacements over the height for both a fixed boundary condition (Fixed B.C.) assumption and rotational spring boundary condition (Spring B.C.), and (3) the interstory drift for both a fixed boundary condition assumption and rotational spring boundary condition. As shown in the figure, using a completely fixed boundary condition at the base of the planar wall results in a 4.1 in. roof displacement. Assuming a rotational spring at the base

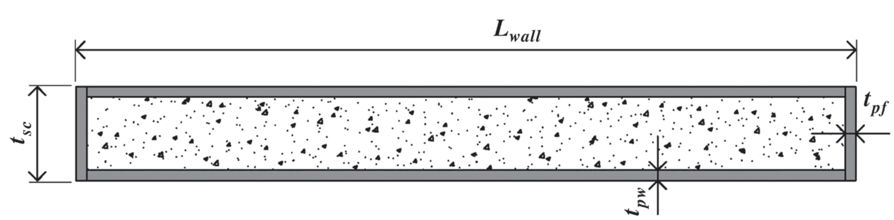
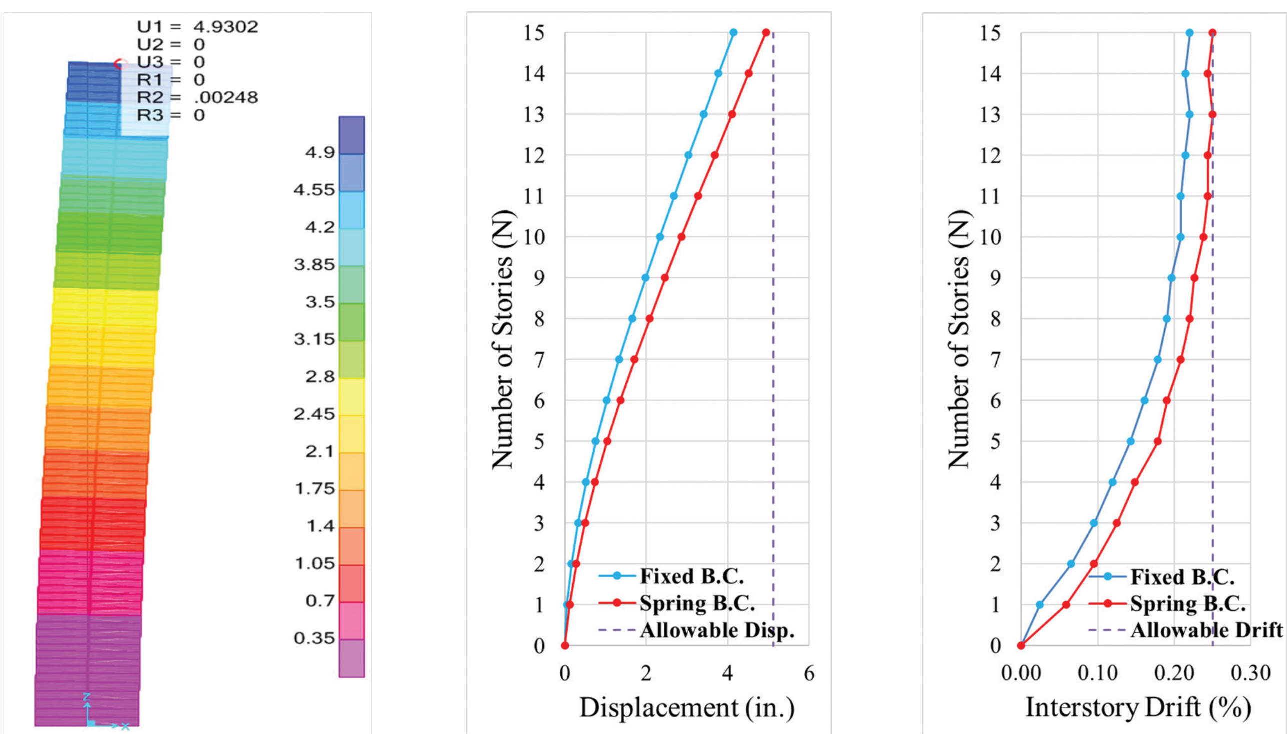


Fig. 9. Cross section of uncoupled C-PSW/CF.

with rotational stiffness calculated by Equation 16 gives a roof displacement of 4.9 in. In this case, considering the rotational stiffness of foundation connection increases the roof displacement by 19%; this highlights the importance of considering the foundation stiffness in checking drift limits for planar C-PSW/CF systems. Lateral drift ($H/500$) and interstory drift ($h/400$) requirements are 5.11 in. and 0.25%, respectively. Hence, the structure meets drift requirements based on the criteria checked. Also, as has been shown in this analysis, the rotational stiffness assumed for the foundation connection has a direct influence on lateral displacements. Therefore, the wall-to-foundation connection stiffness should be considered in the wind design of uncoupled C-PSW/CF structures.

While not a specific design step, it is important to discuss the assumed parameters that can impact drift calculations and, hence, the design of planar C-PSW/CF systems. A sensitivity study to further investigate the effect of flexural stiffness of uncoupled C-PSW/CF on the lateral load response was conducted. A 2D nonlinear finite element model of the uncoupled C-PSW/CF in the given structure was developed using the Abaqus finite element software (2016) and compared with the SAP2000 commercial software model (CIS, 2017). Layered composite shell elements (S4R) were used to model composite walls and three-dimensional truss elements (T3D2) were used for steel flange plates in the Abaqus model. Nonlinear material properties were considered for both steel and concrete. The detailed description of the development of C-PSW/CF model was presented in previous works written by Shafaei et al. (2019) and Bruneau et al. (2019).

Figure 11(a) shows the lateral deformation of 2D nonlinear finite element models of uncoupled C-PSW/CF subjected to wind loads at the 10-year MRI. Figure 11(b) shows the calculated lateral displacements of the uncoupled wall considering four different models using SAP2000 and Abaqus. In one model using SAP2000, the uncracked flexural stiffness, EI_{uncr}, of the wall was used, which resulted in a roof displacement of 2.3 in. Another 2D finite element model with Abaqus was developed using elastic material properties of steel and concrete (no concrete cracking in tension), resulting in a roof displacement of 2.45 in. There is a small difference between the Abaqus and SAP2000 models because the SAP2000 model has beam elements (considering only the flexural behavior), but the elastic Abaqus model has composite shell elements (2D finite element model), which considers the shear deformation of the planar C-PSW/CF as well. Using the effective flexural stiffness



(a) Lateral deformation shape (b) Story displacement (c) Interstory drift

Fig. 10. Uncoupled C-PSW/CF subjected to wind loads at a 10-year MRI.

of the wall in SAP2000 model results in a roof displacement of 4.9 in., while the Abaqus 2D finite element model using nonlinear material properties of steel and concrete (considering the concrete cracking in tension) gives a roof displacement of 4.2 in. Note that for completeness, other wind speeds were checked in the models and are shown in Figure 12.

The results of these analyses show that using the effective flexural stiffness for a planar wall along the height is a conservative approach when investigating serviceability requirements (drift check). Using the effective flexural stiffness in the higher stories of the building, where concrete may not have cracked, is generally a conservative assumption. In the wind design of tall buildings, it is recommended that the cracking moment for the planar C-PSW/CF be calculated and that the uncracked flexural stiffness be used for the walls, where the required moment is lower than the cracking moment.

Step 7: Detail Design of Uncoupled C-PSW/CF

This step determines the tie bar diameter and spacing requirements based on the stability of the empty steel modules. The tie spacing for the uncoupled C-PSW/CF considered 12 in., which results in a plate slenderness ratio of 24. The plate slenderness ratio is lower than the required value, as shown in Equation 17.

$$\frac{S_{tie}}{t_p} = 24.0 \quad < \quad 1.2\sqrt{\frac{E_s}{F_y}} = 28.9 \tag{17}$$

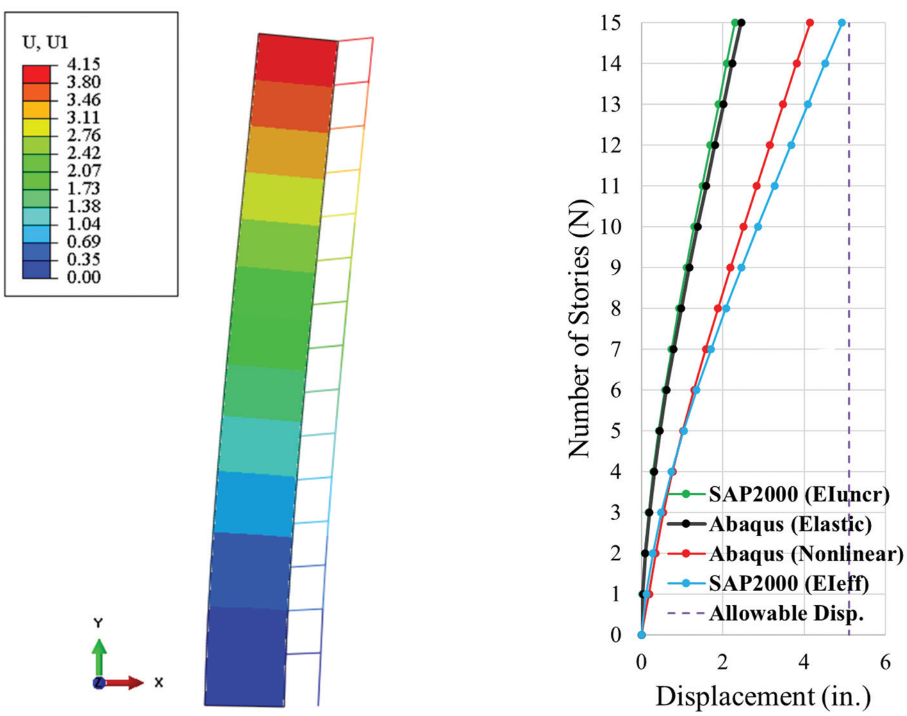
The tie bar diameter and spacing requirements should also be checked for the stability of the empty steel modules during erection and concrete casting. Tie bars with ⅝ in. diameters are selected for the uncoupled walls. This gives an α value of 23.7 and slenderness requirement of 24.5. As shown in Equations 18 and 19, the tie bar requirements for the empty steel module are satisfied.

$$\alpha = 1.7\left(\frac{t_{sc}}{t_p} - 2\right)\left(\frac{t_p}{d_{tie}}\right)^4 = 23.7 \tag{18}$$

$$\frac{S_{tie}}{t_p} = 24.0 \quad < \quad 1.0\sqrt{\frac{E_s}{2\alpha+1}} = 24.5 \tag{19}$$

Step 8: Connection Design of Uncoupled C-PSW/CF

The wall-to-foundation and wall-to-wall connections are designed based on the required forces. In this paper, the connection design (wall-to-foundation and wall-to-wall connection design) is not covered, as it is beyond the scope of this study.



(a) Lateral deformation of 2D finite element model *(b) Lateral displacements calculated by different models*

Fig. 11. Uncoupled C-PSW/CF subjected to wind loads at a 10-year MRI.

Wind Design of Coupled C-PSW/CF

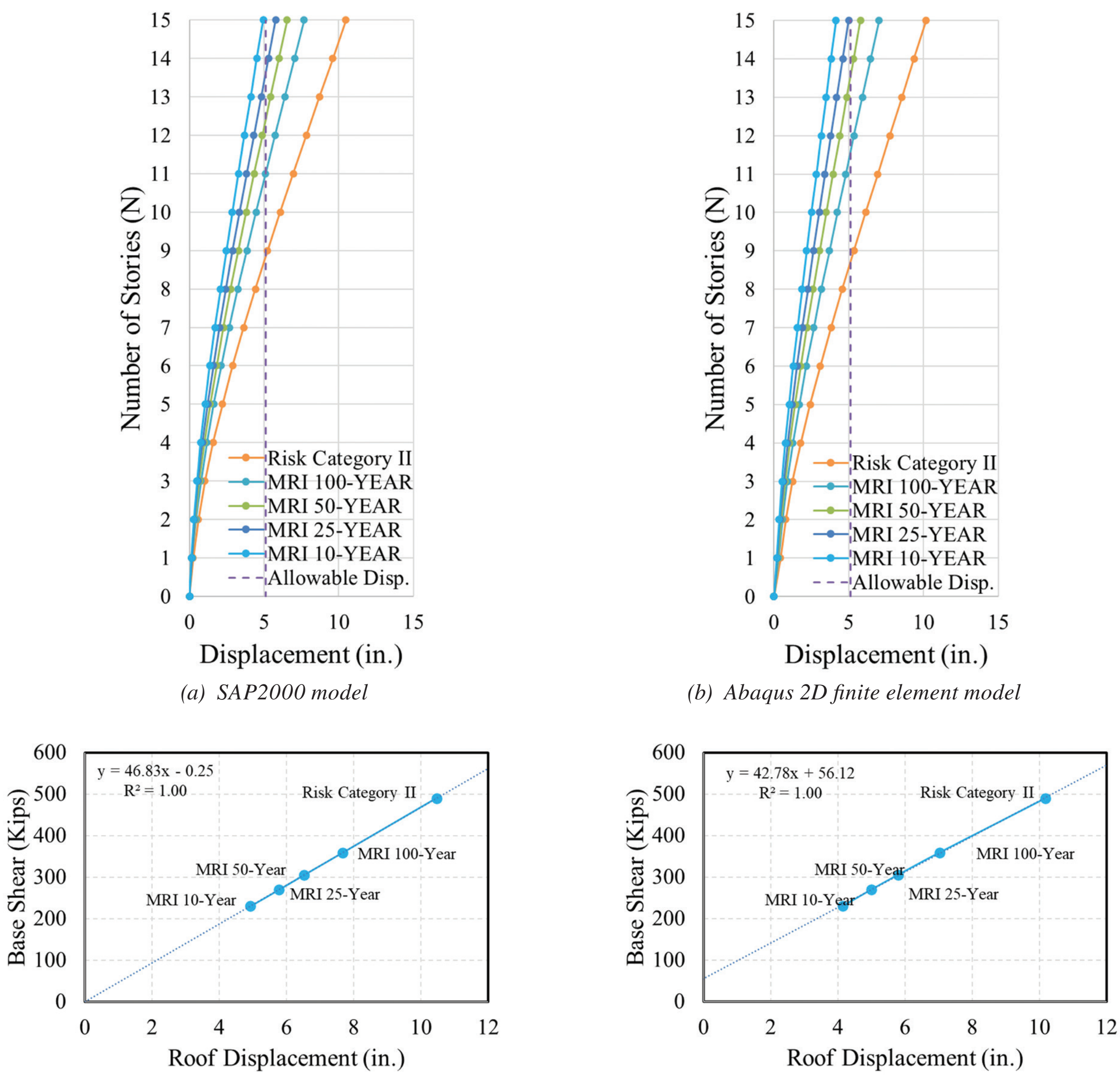
For the building considered in this design example, two coupled C-PSW/CF are used to resist the wind loads in the North–South direction. This section presents the wind design of coupled C-PSW/CF.

Step 1: General Information of the Considered Building

In the North–South direction, coupled C-PSW/CF with overall lengths of 35 ft are used to resist the wind loads, as depicted in Figure 8. The coupled C-PSW/CF consist of two 12.5-ft (150-in.)-long planar walls and composite coupling beams with a length of 10 ft (120 in.). The lengths in this case were chosen to fit the geometric requirements of the building, and similarly, in practice, architectural consideration would like to dictate the layout and lengths of these components. Each coupled C-PSW/CF resists gravity loads of a tributary area of 2633 ft^2, as shown in Figure 8, which results in 158 kips of axial compression force at the individual wall at each story. The total axial compression force on the individual C-PSW/CF is 2,370 kips at the first floor.

Step 2: Calculation of the Wind Loads

The wind speeds for Risk Category II and the 10-year MRI are used to calculate the wind loads for the design of members (strength check) and controlling drift limits



(a) SAP2000 model

(b) Abaqus 2D finite element model

(c) Base shear-roof displacement using SAP2000

(d) Base shear-roof displacement using Abaqus

Fig. 12. Lateral displacement of uncoupled C-PSW/CF subjected to different wind loads.

Table 5. Details of C-PSW/CF in the North–South Direction				
Uncoupled C-PSW/CF	t_{pf}, in.	t_{pw}, in.	Length, in.	Wall Thickness, in.
	½	½	150	18

Table 6. Details of Coupling Beams in the North–South Direction				
Coupling beams	$t_{f.CB}$, in.	$t_{w.CB}$, in.	Depth, h_{CB}, in.	Width, b_{CB}, in.
	¾	½	24	18

Table 7. Required Moments and Forces in Walls and Coupling Beams					
C-PSW/CF				Composite Coupling Beams	
$M_{r,wall}$ kip-in.	$V_{r,wall}$ kips	$P_{r,wall}$ kips	$T_{r,wall}$ kips	$M_{r,CB}$ kip-in.	$V_{r,CB}$ kips
1.47×10^5	458	6,110	1,310	2.28×10^4	381

and occupant comfort of coupled C-PSW/CF in the North–South direction. Since two coupled C-PSW/CF are used in the North–South direction and the building is torsionally regular, one-half of the wind loads is resisted by each coupled system. The wind speeds for Risk Category II and 10-year MRI are 107 mph and 74 mph for Chicago, respectively, as given in Table 1. The resulting wind pressures are shown in Table 2.

Step 3: Select Preliminary Sizes for C-PSW/CF and Coupling Beams

The force and moment distribution along the height in a coupled C-PSW/CF depends directly on the wall and coupling beam sections. Therefore, the preliminary sizes of members need to be selected for calculating the required forces and moments of the members. In other words, member stiffnesses have a direct impact on the coupling ratio. Figure 13, Table 5, and Table 6 summarize the initial sizing of the wall panels and coupling beams for this design example. As shown, the two planar C-PSW/CF in this example have a length, L_{wall}, of 12.5 ft (150 in.) and overall wall thickness, t_{sc}, of 18 in. Also, the chosen wall thickness is assumed suitable for architectural requirements of the building. Steel web and flange plate thicknesses, t_p, are initially sized at ½ in., which results in a 17-in.-thick concrete infill. The coupling beam width, b_{CB}, and depth, h_{CB}, are initially sized at 18 in. and 24 in., while coupling beam thicknesses are ¾ in. for the steel flange plates, $t_{f.CB}$, and ½ in. web plates, $t_{w.CB}$. The ½ in. wall plate thickness was chosen, like the case in the planar wall example, based on construction considerations including fabrication efficiency and handling the panels during erection.

Step 4: Calculation of Required Forces

This step involves determining the required design forces for the walls and coupling beams. The forces calculated

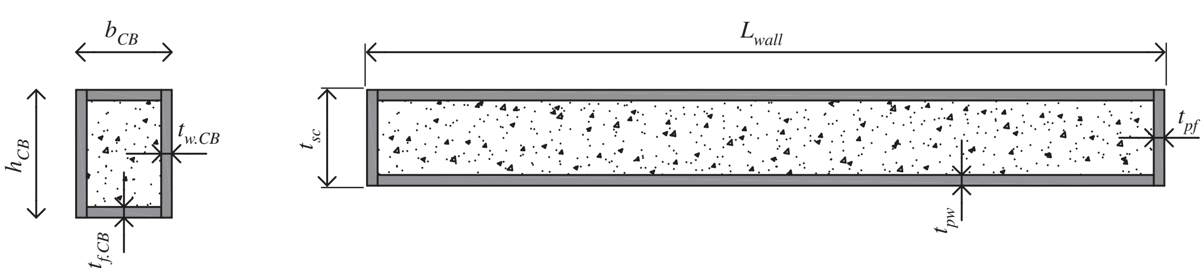


Fig. 13. Cross section of coupling beams and C-PSW/CF.

Table 8. Strength of Composite Coupling Beams and Comparison with the Demand						
Coupling beam	$\phi M_{n.CB}$ kip-in.	$\phi V_{n.CB}$ kips	$M_{r.CB}$ kip-in.	$V_{r.CB}$ kips	$\dfrac{M_{r.CB}}{\phi M_{n.CB}}$	$\dfrac{V_{r.CB}}{\phi V_{n.CB}}$
	2.25×10^4	699	2.28×10^4	381	1.02	0.55

Table 9. Strength of C-PSW/CF and Comparison with Demand								
C-PSW/CF	$\phi M_{n,wall.T}$ kip-in.	$\phi M_{n,wall.C}$ kip-in	$\phi V_{n,wall}$ kips	$M_{r,wall}$ kip-in.	$V_{r,wall}$ kips	$\dfrac{M_{r,wall}}{\phi M_{n,wall.T}}$	$\dfrac{M_{r,wall}}{\phi M_{n,wall.C}}$	$\dfrac{V_{r,wall}}{\phi V_{n,wall}}$
	3.81×10^5	5.27×10^5	6,120	1.47×10^5	458	0.39	0.28	0.07

include the base shear and moments acting on the walls, and coupling beam shear and moment demands. A structural analysis is conducted using the wind loads for Risk Category II to determine the demands. The effective stiffnesses for planar walls and coupling beams are considered to estimate the moments and forces distribution in the members. Table 7 summarizes the required forces and moments in walls and coupling beams, which were calculated considering the wind and dead load combination (wind + 0.90 dead load). Based on the structural analysis, the third-floor coupling beam has the highest required moment and shear force, and in this example, all the coupling beams are designed for these forces. Due to coupling action, tension and compression walls are subjected to axial loads of 1,313 and 6,113 kips, respectively. The overturning moment is 1.28×10^6 kip-in., and the calculated coupling ratio of the system, using Equation 1, is 77%.

Step 5: Strength Check of C-PSW/CF and Coupling Beams

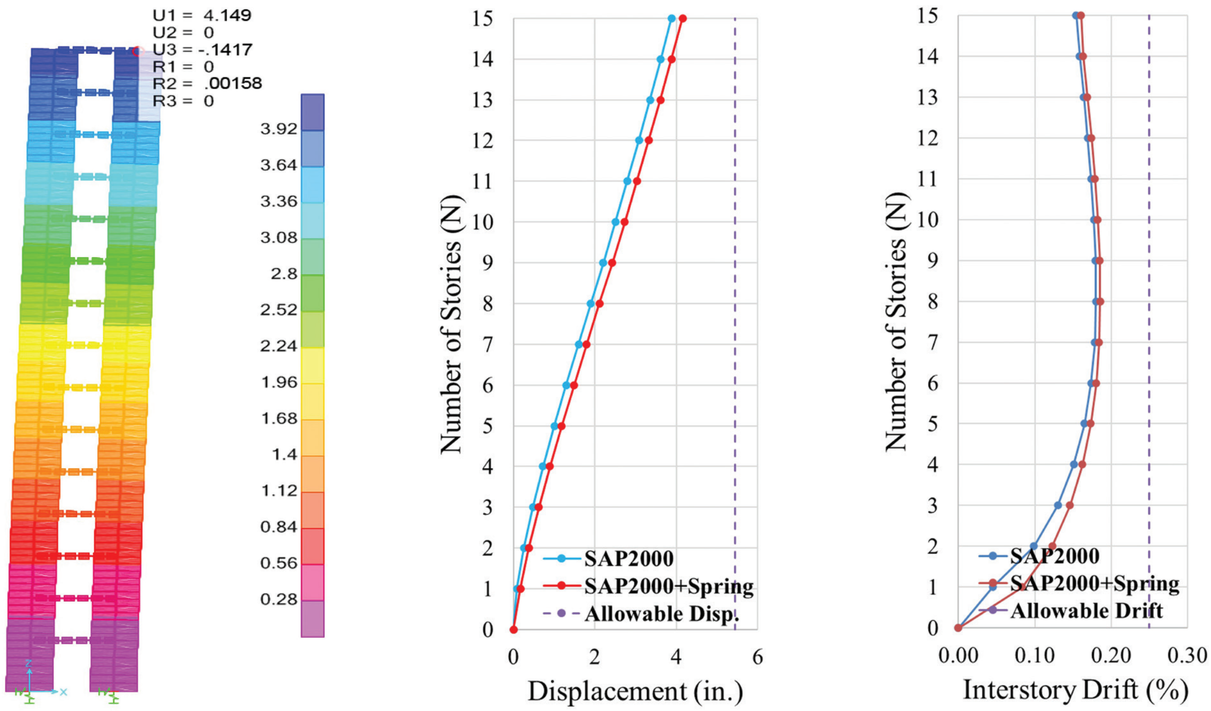
The design flexural and shear strengths, $\phi M_{n,CB}$ and $\phi V_{n,CB}$, of composite concrete-filled coupling beams; the design flexural strengths of the walls in tension and compression, $\phi M_{n,wall.T}$ and $\phi M_{n,wall.C}$, and the design shear strengths of the walls, $\phi V_{n,wall}$, are calculated in this step. The flexural strength for the coupling beams and planar walls in this example is calculated for the cross section, assuming the plastic stress distribution method where all steel plates reach F_y and concrete in compression reach $0.85f_c'$, while concrete in tension is assumed not to contribute to the strength. The shear strength of the coupling beams and planar walls are calculated using Equations 15 and 8, respectively. Tables 8 and 9 present the design strength of coupling beams and C-PSW/CF and comparisons with the demands. As given in Table 8, the flexural strength of the coupling beam governs the design, and the flexural capacities of C-PSW/CF are considerably higher than the required moments.

Step 6: Serviceability (Drift and Occupant Comfort) Check of Coupled C-PSW/CF

Similar to the uncoupled walls, wind loads calculated using the wind speed at the 10-year MRI are used to check drift requirements. Two-dimensional finite element models, developed with the SAP2000 program (using beam elements), of the coupled walls for the considered structure use both a completely fixed-base connection and a rotational spring with a stiffness given from Equation 16. Figure 14 shows the model output in terms of (1) the deformation shape, (2) story displacements over the height for both a fixed boundary condition assumption and rotational spring boundary condition, and (3) the interstory drift for both a fixed boundary condition assumption and rotational spring boundary condition. As shown in the figure, using a completely fixed boundary condition at the base of the wall results in a 3.9 in. roof displacement, while assuming a rotational spring at the base with rotational stiffness calculated by Equation 16 gives a roof displacement of 4.2 in.

In the coupled wall case, considering the rotational stiffness of foundation connection increases the roof displacement by 7%; which is less than the impact for the planar wall-only case. The main reason that foundation stiffness is less important in the coupled wall case is that the coupled systems resist lateral loads by developing coupling action. As a result, considering the rotational stiffness of foundation increases slightly force and moment demands in the members. Lateral drift ($H/500$) and interstory drift ($h/400$) requirements are 5.1 in. and 0.25%. Hence, the structure meets drift requirements based on the criteria checked. Also, as has been shown in this analysis, the rotational stiffness assumed for the foundation connection has a direct influence on lateral displacements, while not as major as the planar wall case.

A sensitivity study to further investigate the effect of flexural stiffness of coupled C-PSW/CF on the lateral load response was conducted. A 2D nonlinear finite element



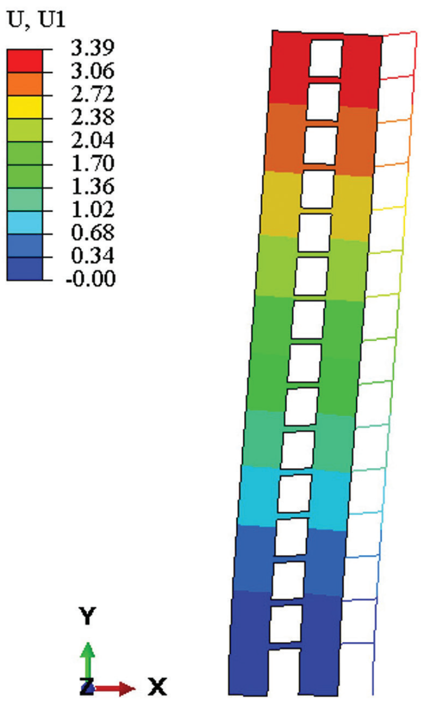
(a) Lateral deformation shape *(b) Story displacement* *(c) Interstory drift*

Fig. 14 Coupled C-PSW/CF subjected to wind loads at a 10-year MRI.

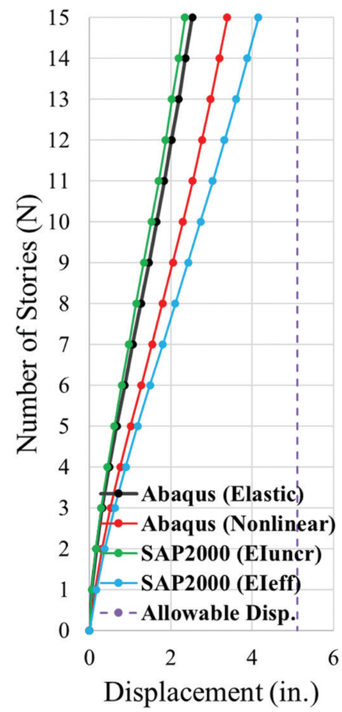
model of the uncoupled C-PSW/CF in the given structure was developed using the finite element software Abaqus and compared with the SAP2000 model. Layered composite shell elements (S4R) were used to model composite walls and three-dimensional truss elements (T3D2) were used for steel flange plates in the Abaqus model. Nonlinear material properties were considered for both steel and concrete. The detailed description of the development of C-PSW/CF model was presented in previous works written by Shafaei et al. (2019 and Bruneau et al. (2019).

Figure 15 shows the lateral deformation of 2D nonlinear finite element models (developed through the Abaqus program) of uncoupled C-PSW/CF subjected to wind loads at the 10-year MRI. Figure 15(b) shows the calculated lateral displacements of the coupled wall considering four different models using SAP2000 and Abaqus. There is a small difference between the elastic Abaqus and SAP2000 (using uncracked section stiffness) models because the SAP2000 model has beam elements (considering only the flexural behavior), but the elastic Abaqus model has composite shell elements (2D finite element model), which consider the shear deformation of the coupling beams and planar C-PSW/CF as well. Also shown in Figure 15(b), there is a difference between the Abaqus 2D nonlinear finite element model and SAP2000 (using beam elements with the effective stiffness). The reason is that the axial compression forces (gravity loads) result in higher flexural stiffness of walls; in other words, the gravity loads prevent concrete cracking, as shown in Figure 16(a). All of the composite coupling beams undergo cracking as well as the bottom of C-PSW/CF, as shown in Figure 16. Due to this cracking, the effective flexural stiffness of concrete-filled composite coupling beams should be considered in SAP2000 model to estimate conservatively the lateral displacements. Note that for completeness, other wind speeds were checked in the models and are shown in Figure 17.

The results of these analyses show a fundamentally different structural behavior of the uncoupled and coupled wall systems subjected to lateral loading in mid-rise applications. While uncoupled walls primarily resist lateral loading through flexure, the coupled walls develop coupling action to resist applied loading. The practical implications of this are that uncoupled walls are more sensitive to assumptions of the foundation stiffness, and using the effective stiffness yields fairly conservative estimates of drift when applied at service level wind loading. For coupled systems, using the effective stiffness also conservatively captures drift estimations. Because concrete cracking in the coupling beams, even at service level winds, more closely matches the effective stiffnesses calculated from equations presented previously, effective flexural stiffness of coupling beams should be used for the serviceability check of coupled system. Also,

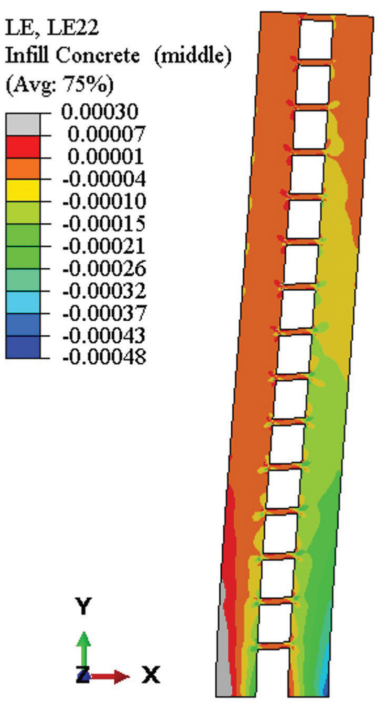


(a) *Lateral deformation shape of 2D finite element model*

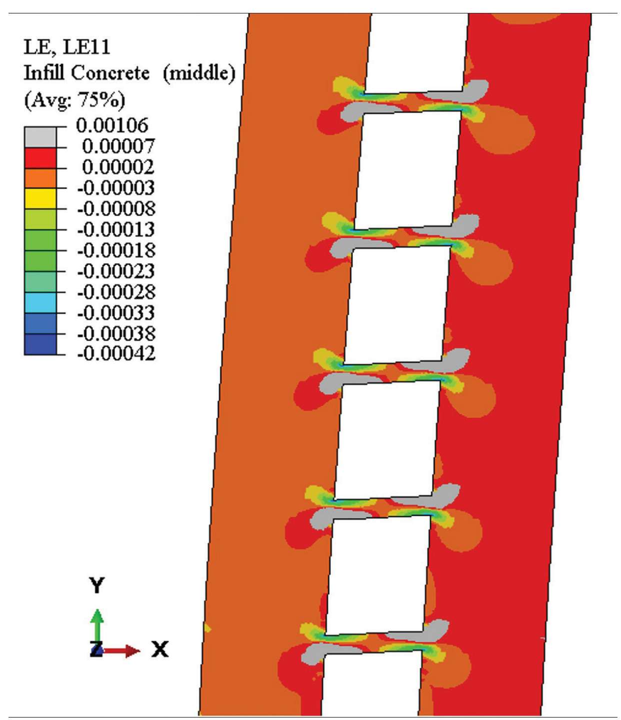


(b) *Lateral displacements of models*

Fig. 15. Coupled C-PSW/CF with different lateral stiffness.



(a) *Concrete cracking strain at the bottom of C-PSW/CF*



(b) *Concrete cracking strain in coupling beams*

Fig. 16. Strain distribution over the coupled C-PSW/CF model.

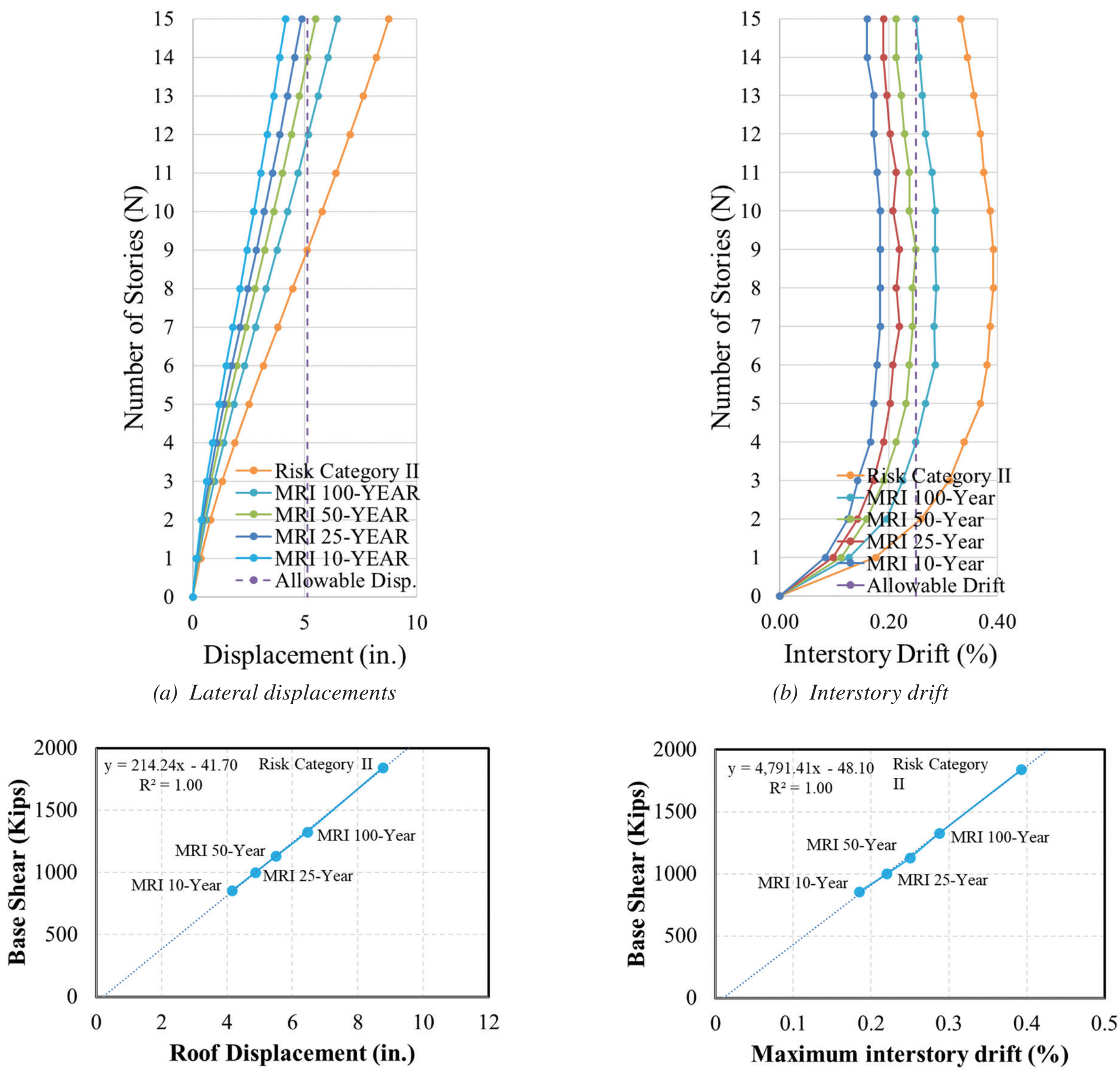
the coupled systems have less sensitivity to the assumed foundation stiffness, but considering the rotational stiffness of wall-to-foundation connections has a direct effect on the force and moment distributions along the height of the systems. From a design standpoint, in mid-rise structures, using the effective stiffness for checking drift limits in both uncoupled and coupled C-PSW/CF systems appears to be reasonable and conservative.

Step 7: Detail Design of Coupling Beams

In this step, the slenderness of composite coupling beams is checked for compactness requirements and also minimum steel requirements if composite coupling beams are used. For the given case of a composite coupling beam, the compactness requirements are met, as are the minimum steel requirements. Also, the width of coupling beams (h_{CB}) and wall thickness (t_{sc}) are both 18 in., allowing for a constructible beam-to-wall connection.

Step 8: Detail Design of C-PSW/CF

This step determines the tie bar diameter and spacing requirements based on the stability of the empty steel modules. Tie bars with ⅝ in. diameter and 12 in. spacing are selected based on a ½ in. plate thickness. The process for determining the tie bar diameter and spacing is the same as for the planar wall-only case shown previously.



(a) Lateral displacements

(b) Interstory drift

(c) Base shear-roof displacement

(d) Base shear-maximum interstory drift

Fig. 17. Coupled C-PSW/CF subjected to different wind loads using SAP2000.

Step 9: Connection Design

Finally, the coupling beam-to-wall, wall-to-foundation, and wall-to-wall connections are designed based on the required forces. The connection design is not included in this paper, as research on this topic is still ongoing by authors.

SUMMARY AND CONCLUSIONS

In this paper, wind (nonseismic) design requirements and procedures for uncoupled and coupled C-PSW/CF, also known as the SpeedCore system, were presented. A wind design example for a 15-story building located in Chicago using both planar (uncoupled) and coupled C-PSW/CF is provided. Conclusions and recommendations from this study are as follows:

1. Construction considerations generally govern the steel plate thickness for C-PSW/CF systems in mid-rise structures. Construction considerations include (a) the stability of empty modules during construction, (b) the adequacy to resist fresh concrete hydrostatic pressure, and (c) the weldability of the steel plates while avoiding excessive localized distortions in the assembled pieces.
2. It is recommended to use the provided effective stiffness for walls and coupling beams for the wind design of C-PSW/CF systems when a detailed nonlinear finite element model is not available.
3. In the wind (nonseismic) design, the strength of uncoupled C-PSW/CF systems is considerably higher than the demand in mid- to high-rise structures as drift limits tend to govern design.
4. The rotational stiffness of the wall-to-foundation connection has a direct influence on the lateral displacements of uncoupled C-PSW/CF systems because the behavior is analogous to a cantilever beam and lateral forces are resisted primarily through flexure. Hence, in the wind design of uncoupled C-PSW/CF, the rotational stiffness of the foundation connection should be considered to accurately estimate the lateral displacements.
5. The rotational stiffness of the wall-to-foundation connection has a marginal influence on the lateral displacement of coupled C-PSW/CF systems compared to uncoupled systems, as the coupled system resists lateral by developing coupling action. Additionally, considering rotational stiffness of wall-to-foundation connections has a direct influence on the force and moment distributions along the height of coupled systems.
6. In most cases, the flexural strength of coupling beams governs the wind design of coupled C-PSW/CF systems in mid-rise structures, and the flexural capacities of the walls are considerably higher than the demand.
7. From a design standpoint, in mid-rise structures, using the effective stiffness for checking drift limits in both uncoupled and coupled C-PSW/CF systems appears to be reasonable and conservative.
8. In wind design of C-PSW/CF systems, the coupling beam-to-wall, wall-to-foundation, wall-to-wall connections (plate splices) are designed based on the required forces, rather than doing a capacity-based design, which would be done for a structure governed by seismic loading.

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