Minimum Requirements and Section Detailing Provisions for Steel-Plate Composite (SC) Walls in Safety-Related Nuclear Facilities

SAAHASTARANSHU R. BHARDWAJ, AMIT H. VARMA and SANJEEV R. MALUSHTE

ABSTRACT

Steel-plate composite (SC) walls are comprised of concrete walls sandwiched between steel faceplates located on the exterior surfaces. These faceplates are anchored to the concrete infill using steel anchors and connected to each other using ties. The faceplates serve as stay-in-place formwork for concrete casting and act as the primary reinforcement after the concrete sets. Steel-plate composite (SC) walls have been used extensively in the third generation of nuclear power plants and are also being considered for small modular reactors (SMRs) of the future. The American Institute of Steel Construction (AISC) has published Supplement No. 1 to AISC N690, which includes Appendix N9 for the design of SC walls in safety-related nuclear facilities. This paper presents the minimum requirements for SC walls and the section detailing provisions from Appendix N9 along with their bases. The minimum requirements include requirements for minimum and maximum SC wall thickness, faceplate thickness, and steel and concrete material strengths. The provisions of Appendix N9 are applicable to SC walls that satisfy these minimum requirements. The section detailing provisions include requirements for the size and spacing of steel anchors and ties to (1) provide composite action between the faceplates and concrete infill, (2) prevent local buckling of the faceplates, (3) provide interfacial shear (slip) resistance, (4) provide structural integrity by preventing section delamination through the concrete thickness, and (5) provide out-of-plane shear strength. The design provisions account for the effects of interaction between out-of-plane shear demands (in both *x*- and *y*-directions) and the corresponding interfacial shear demands while accounting for the differences in behavior between yielding and nonyielding steel anchors and ties as classified by AISC N690 Supplement No. 1.

Keywords: AISC N690s1, modular construction, steel plate, SC wall, steel-plate composite, nuclear, safety-related.

INTRODUCTION

Steel-plate composite (SC) walls have been used extensively in the third generation of nuclear power plants and are also being considered for small modular reactors (SMRs) of the future. For example, the AP1000® nuclear power plants being constructed in China (Sanmen and Haiyang) and in the United States (VC Summer, South Carolina, and Vogtle, Georgia) utilize SC walls for most of the containment internal structures (CIS). Additionally, the AP1000 (DCD, 2011) plants being built in the United States utilize SC wall design for the enhanced shield building to provide seismic resistance and beyond design basis aircraft impact resistance. Similarly, the US-APWR® (DCD, 2013) power plants being considered for licensing in the United States utilize SC walls for the entire CIS. Future nuclear power plants, including advanced light water reactors (ALWRs) and small modular reactors (SMRs), are considering SC walls for achieving modularity and expediting the construction schedule while improving structural strength, safety, and resilience for seismic load combinations and accident thermal load combinations (Varma et al., 2015).

As shown in Figure 1, steel-plate composite (SC) walls consist of concrete walls sandwiched between two steel plates (also referred as faceplates) located at the exterior surfaces. The faceplates are anchored to the concrete infill using steel anchors and are connected to each other using tie bars. These faceplates serve as stay-in-place formwork for casting concrete and act as the primary reinforcing steel after the concrete sets. Thus, the SC wall system eliminates the need for conventional formwork, which can expedite the construction schedule. The SC wall system also eliminates the need for conventional steel reinforcement (rebars), which reduces congestion-related issues and can further expedite construction. Additional advantages of SC construction include modularity and construction schedule (Varma et al., 2015), structural strength and safety for seismic and accident thermal loading combinations (Sener et al., 2015a; Booth et al., 2015a), and resilience to impactive (Bruhl et al., 2015) and impulsive (Bruhl and Varma, 2015) loading. These are all discussed in Varma et al. (2015) and not repeated here.

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The behavior of SC walls under axial compression (Zhang et al., 2014), out-of-plane flexure (Sener et al., 2015b), and out-of-plane shear (Sener and Varma, 2014) is similar to that of reinforced concrete (RC) walls. However, behavior of SC walls under in-plane shear (Seo et al., 2016; Varma et al., 2011a; Ozaki et al., 2004), combined in-plane forces and out-of-plane shear forces (Varma et al., 2014), can be significantly different from that of RC walls. Additionally, specific limit states such as faceplate local buckling (Zhang et al., 2014), interfacial shear failure (Sener and Varma, 2014) between the faceplates and concrete infill, and section delamination (through the concrete infill) need to be adequately considered in the design of SC walls. These limit states are discussed in the following sections along with section detailing provisions to prevent them from limiting the design.

The use of SC construction in the United States has been hindered by the absence of a U.S.-based design code for SC walls. In 2006, AISC formed a subcommittee on modular composite construction under Task Committee 12 for nuclear structures. Over the next nine years, from 2006 to 2015, specification for the design of SC walls in safety-related nuclear facilities was developed and finalized as an appendix (Appendix N9) in AISC N690s1 (AISC, 2015), which is Supplement No. 1 to AISC N690-12 (AISC, 2012). An outline of the modular composite specification (Appendix N9) and a brief discussion of how the provisions of the appendix may be used are provided in Bhardwaj et al. (2015).

The modular composite specification (Appendix N9) starts with minimum detailing requirements that SC walls should satisfy so that the rest of the provisions of the specification may be used for their design. These include requirements for steel faceplate thickness, SC wall thickness, steel yield strength, concrete strength, and others. The details of these minimum requirements and the corresponding rationale are presented in the following section.

MINIMUM REQUIREMENTS FOR SC WALLS

The majority of SC wall tests have been performed on walls with two faceplates, where composite action is provided using either steel anchors or tie bars or a combination of both. The provisions of the modular composite specification have been developed based on this experimental database and the associated mechanics-based behavioral models. As a result, the modular composite specification is limited to walls with two faceplates anchored to the concrete infill by means of steel anchors, tie bars, or a combination of both.

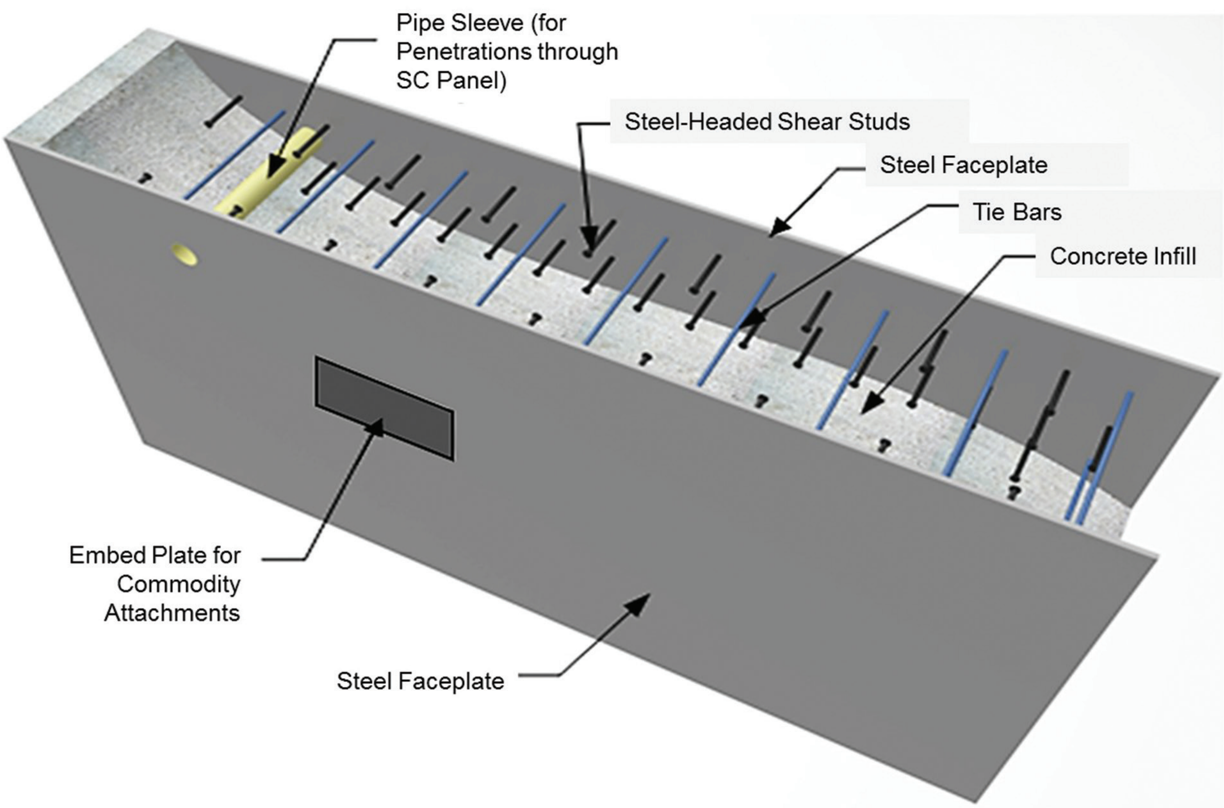


Fig. 1. Typical SC wall configuration (AISC, 2015).

Table 1. Minimum Requirements for SC Walls		
Parameter	**Minimum Value**	**Maximum Value**
Reinforcement ratio, ρ	0.015	0.050
Faceplate thickness, t_p	0.25 in. (6 mm)	1.50 in. (38 mm)
SC section thickness, t_{sc}—interior walls	12 in. (300 mm)	60 in. (1500 mm)
SC section thickness, t_{sc}—exterior walls	18 in. (450 mm)	60 in. (1500 mm)
Steel faceplate yield stress, F_y	50 ksi (350 MPa)	65 ksi (450 MPa)
Concrete compressive strength, f_c'	4 ksi (28 MPa)	8 ksi (55 MPa)

The provisions are not applicable to SC walls reinforced with more than two steel plates, which may be used for the design of the primary shield structure supporting and shielding the reactor vessel. The design of such structures composed of extremely thick SC walls with three or more steel plates is discussed in Booth et al. (2015b).

The modular composite specification is limited to the design of SC walls with boundary elements or flanges, which are typically the purview of safety-related nuclear facilities consisting of labyrinthine SC walls connected to each other and to the concrete floor or basemat. The modular composite specification does not include provisions for the design of SC wall piers (with no flanges or large boundary columns) that are typically used in commercial building structures. The seismic behavior and design of SC wall piers is discussed in Kurt et al. (2016) and Epackachi et al. (2015) and the upcoming AISC 341 (AISC, 2016) seismic design provisions.

Table 1 summarizes some of the minimum requirements for SC walls that can be designed using the provisions of the modular composite specification. These minimum requirements were selected based on the range of parameters in the experimental database of SC walls and some other criteria as described here. The minimum thickness, t_{sc}, for exterior walls is based on Table 1 of the Standard Review Plan (SRP), Section 3.5.3, Revision 3 (NRC, 2007). The maximum limit for t_{sc} is based on the experimental database of out-of-plane shear tests conducted on SC walls in Japan (Ozaki et al., 2001), South Korea (Hong et al., 2009), and the United States (Sener and Varma, 2014). The reinforcement ratio, ρ, is calculated using Equation 1:

$$\rho = \frac{2t_p}{t_{sc}} \quad \text{(AISC N690 Eq. A-N9-1)} \qquad (1)$$

where

t_p = faceplate thickness

t_{sc} = section thickness for SC wall

The limits for ρ, shown in Table 1, were established because the use of reinforcement ratios lower than 0.015 can lead to potential concerns regarding (1) handling strength and stiffness of empty modules and (2) higher residual stresses due to fabrication activities and concrete casting. The use of reinforcement ratios higher than 0.05 can potentially result in higher concrete stresses and change the governing in-plane shear strength limit state from steel faceplate yielding to concrete compression strut failure, which can potentially reduce the strength and ductility of SC walls (Seo et al., 2016).

The limits for faceplate thickness (shown in Table 1) were established because 0.25-in.-thick faceplate is needed for adequate stiffness and strength during concrete placement and rigging and handling operations. Additionally, faceplates thinner than 0.25 in. (6 mm) can have the material properties and imperfections (waviness, etc.) associated with sheet metal (instead of structural plates) (Bruhl et al., 2015). The maximum faceplate thickness of 1.5 in. (38 mm) corresponds to a reinforcement ratio of 0.050 for a 60-in. (1500-mm) -thick SC wall. The minimum thickness for interior walls is based on the maximum reinforcement ratio (ρ = 0.050) and minimum faceplate thickness, t_p, equal to 0.25 in. (6 mm). The specified minimum thickness values for interior and exterior walls are conservatively larger than absolute minimum values.

As shown in Table 1, a minimum yield stress of 50 ksi (350 MPa) is specified to prevent premature yielding of the steel faceplates due to (1) residual (locked-in) stresses from concrete casting and (2) thermally induced stresses from accident thermal scenarios because such premature yielding could limit the strength and ductility of SC walls (Varma et al., 2013). High-strength steels with yield stress greater than 65 ksi (450 MPa) are typically less ductile and hence not desirable for beyond-safe shutdown earthquake (SSE) shaking (Varma, 2000).

The use of concrete with compressive strength less than 4 ksi (28 MPa) is rare in safety-related nuclear facilities, with the possible exception of the concrete basemat. A minimum concrete strength of 4 ksi (28 MPa) is also specified so that under in-plane shear loading, the minimum principal (compressive) stress in concrete remains in the elastic range while faceplate yielding governs. Provisions of the modular composite specification are based on the test results of specimens with specified concrete strength of 8 ksi (55 MPa) or less. Figure 2 shows the range of concrete strengths covered in the experimental database of out-of-plane shear tests conducted internationally on SC walls, discussed in Sener and Varma (2014). In Figure 2, the ordinate is the ratio of the experimental out-of-plane shear strength, V_{exp}, with respect to the nominal out-of-plane shear strength calculated using ACI 349 (ACI, 2006) code equations, $V_{n.ACI}$, as discussed in Sener and Varma (2014). The entire database of SC wall tests includes specimens with concrete strengths in the range of 4–8 ksi (28–55 MPa).

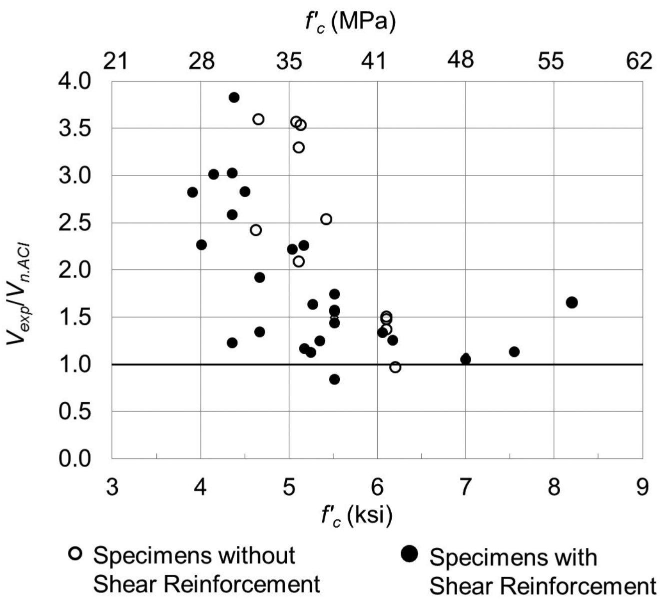


Fig. 2. Range of concrete compressive strength from experimental database (Sener and Varma, 2014).

SECTION DETAILING REQUIREMENTS

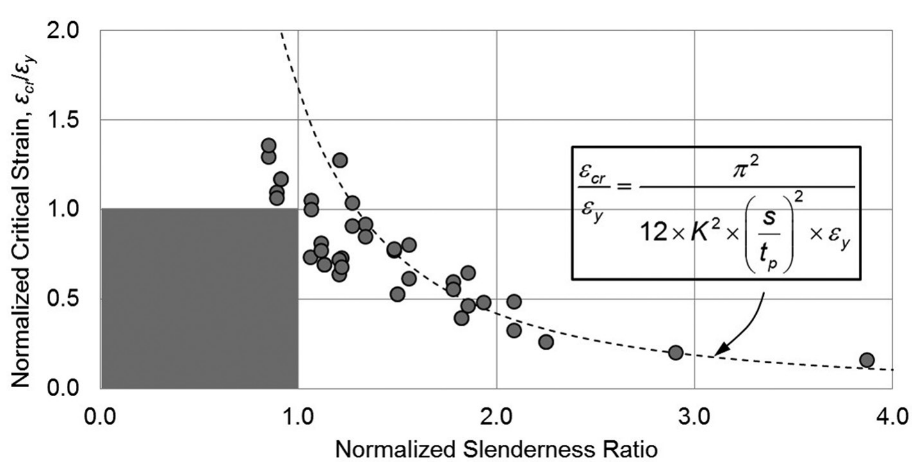
Faceplate Slenderness Requirement

Local buckling of steel faceplates is an important limit state to be considered in the design of SC walls. When subjected to compressive stresses, the steel faceplates of SC walls can undergo local buckling between the steel anchors. This local buckling behavior of steel faceplates has been investigated experimentally by Akiyama et al. (1991), Usami et al. (1995), Kanchi et al. (1996), Choi and Han (2009), and Zhang (2014). These experimental studies have evaluated the effects of plate slenderness ratio, s/t_p (defined as the steel anchor spacing, s, divided by the faceplate thickness, t_p, and yield stress, F_y, on local buckling of faceplates).

Zhang et al. (2014) have summarized these experimental studies and conducted additional numerical analyses to confirm and expand the experimental database. Figure 3 shows the relationship between the normalized critical buckling strain (buckling strain/steel yield strain, $\varepsilon_{cr}/\varepsilon_y$) and the normalized faceplate slenderness ratio ($s/t_p \times F_y/E$). For large slenderness ratios, ε_{cr} is reasonably consistent with Euler's elastic column buckling curve with partially fixed (K = 0.7) end conditions. As the slenderness ratio decreases, ε_{cr} becomes more conservative with respect to Euler's column buckling curve due to material inelasticity. No data points are located in the shaded area, which implies that yielding in compression occurs before local buckling for faceplate slenderness ratio (s/t_p) less than 1.0.

Based on the studies conducted by Zhang et al. (2014), the modular composite specification requires the steel faceplates to be nonslender—that is, undergo yielding in compression before local buckling—as follows:

> Faceplates shall be anchored to concrete using steel anchors, ties, or a combination thereof. The



$$\frac{\varepsilon_{cr}}{\varepsilon_y} = \frac{\pi^2}{12 \times K^2 \times \left(\frac{s}{t_p}\right)^2 \times \varepsilon_y}$$

Fig. 3. Normalized critical buckling strain vs. slenderness ratio with K = 0.7 (from Zhang et al., 2014).

width-to-thickness ratio of the faceplates, b/t_p, shall be limited by Equation 2:

$$\frac{b}{t_p} \leq 1.0\sqrt{\frac{E_s}{F_y}} \quad \text{(AISC N690 Eq. A-N9-2)} \qquad (2)$$

where

E_s = modulus of elasticity of steel, ksi (MPa)

F_y = specified minimum yield stress of faceplate, ksi (MPa)

b = largest unsupported length of faceplate between rows of steel anchors or ties, in. (mm)

t_p = thickness of faceplate, in. (mm)

Because ties may also act as steel anchors, Equation 2 considers the largest unsupported length between rows of steel anchors or ties, b. For steel faceplates with a specified yield stress greater than or equal to 50 ksi (350 MPa), the slenderness limit of Equation 2 implicitly addresses the influence of residual stresses or stresses due to concrete casting (Bhardwaj and Varma, 2016). The use of faceplates with a specified yield stress less than 50 ksi (350 MPa) is not permitted because the slenderness limit of Equation 2 cannot assure yielding in compression before local buckling due to the influence of residual stresses and concrete casting stresses (Zhang, 2014).

Requirements for Composite Action

The steel faceplates are anchored to the concrete infill using steel anchors (and/or tie bars), which develop composite action by resisting the relative slip between the steel faceplates and the concrete infill. These steel anchors can develop the yield strength of the steel faceplate over a certain length depending on their spacing. Steel anchors used in SC construction may consist of steel-headed studs, embedded steel shapes, tie bars (smooth or deformed), or a combination thereof, which can be attached to the faceplates by welding or bolting.

Classification of Steel Anchors

Steel anchors that have a ductile shear force-slip displacement behavior can redistribute the interfacial shear equally over several connectors. Such connectors are referred to as yielding type (e.g., steel-headed stud anchors). Steel anchors that have a nonductile, shear, force-slip behavior cannot redistribute interfacial shear force over several connectors and are referred to as nonyielding type. The modular composite specification provides requirements to classify steel anchors as yielding or nonyielding type:

> Connectors with interfacial slip of at least 0.20 in. (5 mm), while maintaining a resistance greater than 90% of the peak shear strength, shall be classified as yielding steel anchors. Steel anchors not meeting this requirement shall be classified as nonyielding steel anchors. Steel-headed stud anchors shall be classified as yielding steel anchors and the available shear strength, Q_{cv}, shall be obtained using AISC 360 (AISC, 2010). Classification and available strength, Q_{cv}, for all other types of steel anchors shall be established through testing.

As shown in Figure 4, interfacial slip displacement capability of at least 0.20 in. (5 mm) before reduction in shear strength to 90% of the available shear strength is required to qualify a yielding type connector. Steel-headed stud anchors are typically capable of sustaining at least 0.20 in. (5 mm) of interfacial slip displacement in a ductile manner (Ollgaard et al., 1971). All other types of steel anchors need to be tested to determine their available shear strength and slip displacement capacity. An adequate number of tests must be performed to ascertain the available strength of nonyielding steel anchors. The safety factors applicable for nonyielding steel anchors can be obtained from the experimental studies by following the reliability analysis procedures used by Ravindra and Galambos (1978).

For cases where a combination of yielding and nonyielding steel anchors are used, the system is classified conservatively as nonyielding type because (1) the peak strengths of yielding and nonyielding steel anchors may not occur at similar slip displacement levels; (2) the post-peak behavior of yielding and nonyielding steel anchors may be significantly different; and (3) as a result, the interfacial shear force cannot be distributed equally over several connectors. The system is classified as nonyielding type, and the strength of yielding steel anchors has to be limited to the

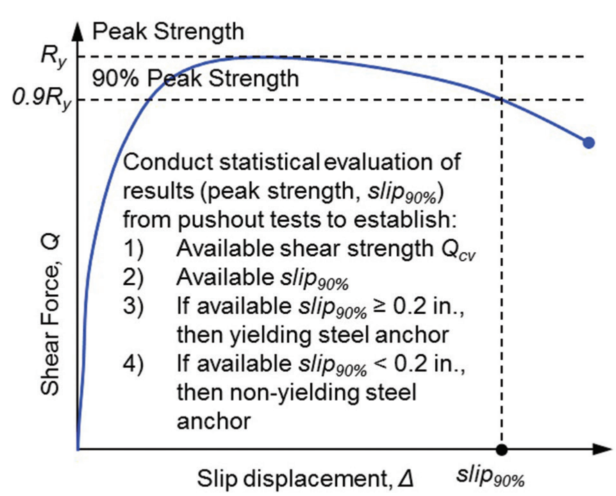


Fig. 4. Typical steel anchor force-slip behavior from a pushout test.

strength corresponding to the slip displacement at which the nonyielding steel anchors reach their ultimate strength. This is illustrated in Figure 5. This requirement is mentioned in the modular composite specification as follows:

> Where a combination of yielding steel anchors and nonyielding steel anchors is used, the resulting steel anchor system shall be classified as nonyielding. In these cases, the strength of yielding steel anchors shall be taken as the strength corresponding to the interfacial slip at which the nonyielding steel anchors reach their ultimate strength.

Spacing of Steel Anchors: Development Length

The development length, L_d, is the distance over which the steel faceplate can develop its yield strength due to the shear strength and number of steel anchors over L_d. Thus, any target development length can be achieved by designing the size and spacing of steel anchors. The target development length has a direct influence on the degree of composite action in terms of the strain compatibility achieved between the steel faceplate and concrete infill. This partial composite action (strain compatibility, or interfacial slip) has a direct influence on the flexural stiffness (*EI*) of the composite section.

Zhang et al. (2014) investigated the relationship between the target development length and the degree of composite action (strain compatibility) between steel and concrete. They concluded that the target development length should not exceed three times the section (or wall) thickness, t_{sc}, and that 75 to 90% partial composite action (in terms of strain compatibility) can be achieved for target development lengths less than or equal to $3t_{sc}$. They also investigated the relationship between partial composite action (strain compatibility) and flexural section stiffness, *EI*, of the composite section. They concluded that 75 to 90% partial composite action has less than a 10% influence on the cracked-transformed flexural stiffness, *EI*, of the composite section.

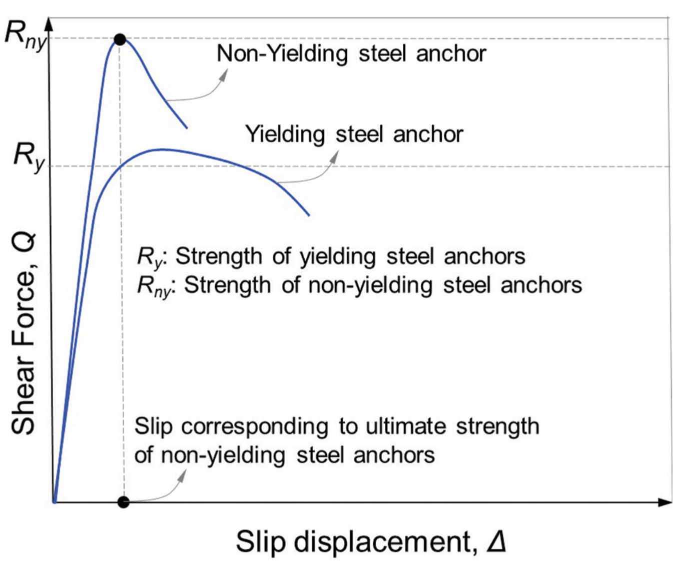


Fig. 5. Strength of yielding steel anchors that form part of a nonyielding steel anchor system.

Based on Zhang et al. (2014), the modular composite specification requires the development length to be less than or equal to $3t_{sc}$. For the range of geometric parameters—wall thickness t_{sc}, plate thickness t_p, and stud anchor diameter and spacing—used in nuclear construction, this requirement ($L_d \le 3t_{sc}$) will result in faceplate development lengths that are comparable to ACI 349 (ACI, 2006) based development lengths calculated for No. 11, 14 or 18 rebar used typically in nuclear concrete construction.

Figure 6 shows the free-body diagram associated with the development length, L_d, of the steel faceplate. In the diagram, the width of the faceplate is equal to the transverse spacing, s_T, of the stud anchors, and it develops yield stress, F_y, over the development length. For designs with yielding stud anchors, the interfacial shear force is assumed to redistribute uniformly over the development length, and the value is governed by the available shear strength, Q_{cv}, of the yielding anchor. Zhang et al. (2014) developed Equation 3 using the free-body diagram shown in Figure 6 to relate the development length, L_d, to the available shear strength, Q_{cv}, and spacing of yielding stud anchors, where s_L is the spacing in the direction of the development length and s_T is the spacing transverse to it:

$$Q_{cv}\frac{L_d}{s_L} \ge s_T t_p F_y \tag{3}$$

Equation 4 was developed by the authors for nonyielding stud anchors. The interfacial shear force is assumed to distribute linearly over the development length, and the maximum value is governed by the available shear strength, Q_{cv}, of the nonyielding anchor. Both Equations 3 and 4 are based on the consideration that the total shear strength of the anchors over the development length should be greater than or equal to the yield strength of the faceplate:

$$\frac{1}{2}Q_{cv}\frac{L_d}{s_L} \ge s_T t_p F_y \tag{4}$$

The stud anchor spacing, *s*, is typically equal in the longitudinal (s_L) and transverse (s_T) directions, and Equations 3 and 4 can be simplified to Equation 5, which is provided in the modular composite specification. The engineer selects (or designs) the development length, L_d, for the SC wall and calculates the stud anchor spacing required to achieve it. The development length cannot exceed three times the wall thickness. According to the modular composite specification, the spacing required to develop the yield strength of the faceplates over the development length, L_d, is given by Equation 5:

$$s \le c_1 \sqrt{\frac{Q_{cv} L_d}{T_p}} \qquad \text{(AISC N690 Eq. A-N9-3)} \qquad (5)$$

where

L_d = development length, in. (mm) $\le 3t_{sc}$

Q_{cv} = available shear strength of steel anchor, kips (N)

T_p = $F_y t_p$ for LRFD, kip/in. (N/mm)

c_1 = 1.0 for yielding steel anchors

= 0.7 for nonyielding steel anchors

The constant c_1 takes into consideration the difference in the resistance distributions of yielding and nonyielding steel anchors.

Spacing of Steel Anchors: Interfacial Shear

When subjected to out-of-plane shear force, V, there are three potential failure modes: (1) out-of-plane flexural yielding, (2) out-of-plane shear failure through the concrete infill and tie bars, or (3) interfacial shear failure at the steel–concrete interface through the shear connectors. The out-of-plane flexural yielding limit state is discussed in detail by Sener et al. (2015b), and the out-of-plane shear failure mode is discussed in detail by Sener and Varma (2014). They have also provided design strength equations and resistance (ϕ) factors for SC walls. The modular composite specification also includes equations for calculating the (1) flexural strength (M_n) based on Sener et al. (2015b) and (2) the out-of-plane shear strength (V_c) of SC walls based on Sener and Varma (2014). Therefore, this subsection focuses on the third failure mode—that is, interfacial shear failure—which is somewhat similar to bond shear failure in reinforced concrete beams. The design philosophy is to prevent interfacial shear failure from occurring before out-of-plane shear failure—that is, interfacial shear failure should not be the governing failure mode of the three potential modes.

Figure 7a shows the free-body diagram of an SC wall subjected to out-of-plane shear forces, V. The out-of-plane shear forces, V, change the out-of-plane bending moment, M, by ΔM along the length of the shear span, L_v. As a result, the tension force in the steel faceplate changes by $\Delta M / jt_{sc}$ over the shear span length, where jt_{sc} is the arm length associated with the bending moment over the cross-section and can be estimated conservatively as $0.9t_{sc}$ (Sener et al., 2015b). This change (in tension force) is in equilibrium with the interfacial shear flow between the steel faceplate and concrete infill, which is resisted by the steel anchors as shown in Figure 7b. The interfacial shear strength of the anchors must be greater than or equal to the shear flow demand to prevent failure.

Figure 7c shows the free-body diagram with yielding anchors resisting the interfacial shear flow. The interfacial shear strength is equal to the number of anchors, calculated as the shear span length divided by the longitudinal spacing, L_v / s_L, multiplied by the available shear strength, Q_{cv}, of the yielding anchor. As expressed by Equation 6, the interfacial shear strength should be greater than or equal to the demand shear flow. If the longitudinal and transverse spacing of anchors is equal (i.e., $s = s_L = s_T$), then Equation 6 can be simplified to Equation 7. In Equation 7, $\Delta M / L_v$ is equal to the out-of-plane shear force, V, and is limited to the out-of-plane shear strength, V_c, of the SC wall section. Thus, Equation 7 can be simplified to Equation 8, which specifies the maximum spacing, s, of anchors to prevent interfacial shear failure from occurring before out-of-plane shear failure:

$$Q_{cv} \frac{L_v}{s_L} \ge \frac{M_x}{0.9t_{sc}} s_T \qquad (6)$$

$$s \le \sqrt{\frac{Q_{cv}(0.9t_{sc})}{V_c}} \qquad (7)$$

Similarly, Figure 7d shows the free-body diagram with nonyielding anchors resisting the interfacial shear flow. For this case, the interfacial shear strength is equal to one-half

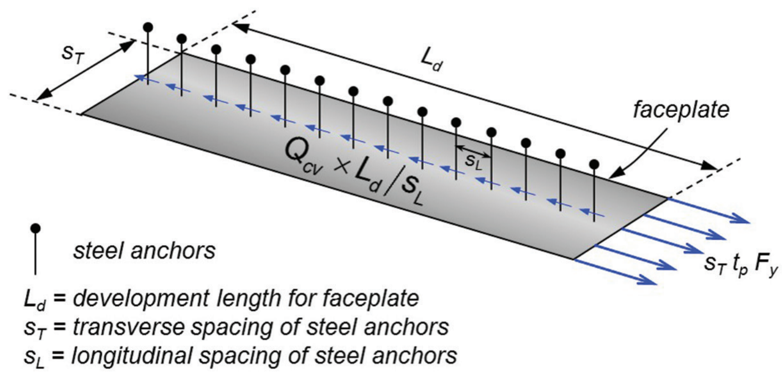
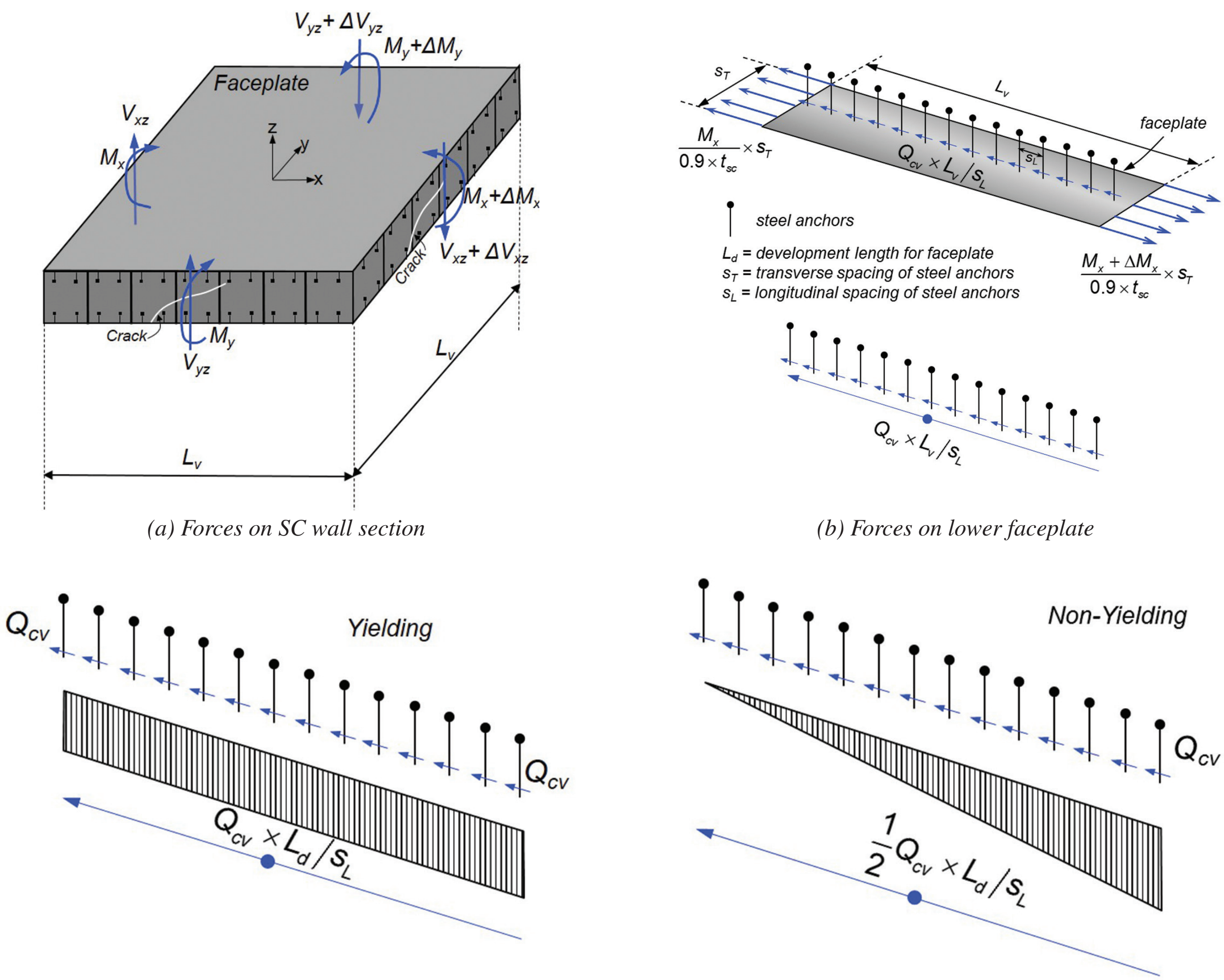


Fig. 6. Yielding steel anchor spacing requirement (Zhang et al., 2014).

of the number of anchors, calculated as L_v/s_L, multiplied by the available shear strength, Q_{cv}, of the nonyielding anchor because the most stressed nonyielding anchor will fail before redistributing the shear flow over several anchors. Similar to the preceding discussion, Equation 8 specifies the maximum spacing of anchors to prevent interfacial shear failure from occurring before out-of-plane shear failure. In this Equation 8, the factor c_1 distinguishes between the design of yielding and nonyielding anchors. The modular composite specification presents this requirement as follows:

> The spacing required to prevent interfacial shear failure before out-of-plane shear failure of the SC section is given by Equation 8:
>
> $$s \le c_1\sqrt{\frac{Q_{cv}l}{V_c/0.9t_{sc}}} \qquad \text{(AISC N690 Eq. A-N9-4)} \quad (8)$$
>
> where
>
> V_c = available out-of-plane shear strength per unit width of SC panel section, kip/ft. (N/mm)
>
> Q_{cv} = available shear strength of steel anchor, kip (N)
>
> l = unit width, 12 in./ft. (1000 mm/m)
>
> t_{sc} = SC section thickness, in. (mm)

The constant c_1 takes into consideration the difference in the resistance distribution of yielding and nonyielding steel anchors (Figure 7). The modular composite specification requires that the spacing of steel anchors be less than the spacing calculated using Equations 5 and 8. Steel anchor spacing is typically governed by Equation 5—that is, the requirement for the development length to be no greater than $3t_{sc}$. However, for portions of the SC structure subjected



(a) Forces on SC wall section

(b) Forces on lower faceplate

(c) Shear resistance of yielding steel anchors

(d) Shear resistance of nonyielding steel anchors

Fig. 7. Steel anchor spacing requirement for preventing interfacial shear failure before out-of-plane shear failure.

to extremely large out-of-plane moment gradient, Equation 8—that is, the requirement for interfacial shear strength to be greater than the available out-of-plane shear strength—may control the steel anchor spacing.

Tie Requirements

Ties are required to connect the steel faceplates of the SC wall through the concrete infill. A tie may be a single structural element (e.g., tie rod) or an assembly of several structural elements (e.g., tie bar with gusset plate at one or both ends). They provide direct connectivity between the steel faceplates and, along with the stud anchors, enable the SC wall section to behave as an integral unit. SC walls for nuclear applications can be extremely thick (up to 60 in. as permitted by AISC N690s1) with relatively thin (0.5- to 1.0-in.-thick) steel faceplates on the surfaces. If the steel faceplates are not tied together, then there is a potential failure mode, which consists of splitting or delamination of the wall section (along a plane parallel to the faceplates) through the concrete thickness. Such a failure mode has only been observed in the force transfer region of an axially loaded eccentric lap-splice connection (Seo and Varma, 2016), but not in member tests.

Ties serve multiple purposes in SC walls. They provide structural integrity in terms of resistance to delamination or splitting failure of the wall section through the concrete thickness. They provide out-of-plane shear reinforcement and contribute to the out-of-plane shear strength, depending on their classification and spacing. Ties act in tandem with steel anchors to contribute to the interfacial shear strength of SC walls. Ties can also participate in the force transfer mechanisms associated with SC wall connections if they are engaged appropriately. Per the modular composite specification:

> The opposite faceplates of SC walls shall be connected to each other using ties consisting of individual components such as structural shapes, frames or bars.

Classification and Spacing of Ties

The design tensile strength of ties considers the limit states of (1) gross yielding, (2) net section rupture, and (3) failure of tie-to-faceplate connections. If the limit state of gross yielding governs, then the ties are considered as yielding; otherwise, the ties are considered as nonyielding. Due to the differences between nominal and actual (measured) material properties, there may be cases where components that appear to be governed nominally by yielding may, in reality, be controlled by nonyielding limit states. Therefore, a minimum margin was specified between the nominal strength calculated for yielding and nonyielding limit states. The modular composite specification addresses this requirement as mentioned here:

> Ties shall be classified as yielding shear reinforcement when
>
> $$F_{ny} \leq 0.8F_{nr} \qquad \text{(AISC N690 Eq. A-N9-5)} \qquad (9)$$
>
> where
>
> F_{nr} = nominal rupture strength of the tie, or the nominal strength of the associated connection, whichever is smaller, kips (N)
>
> F_{ny} = nominal yield strength of the tie, kips (N)
>
> Otherwise, ties shall be classified as nonyielding shear reinforcement.

These requirements ensure that for ties to be classified as yielding, their nominal rupture strength (or the nominal strength of associated connections) should be at least 1.25 (1/0.8) times the nominal yield strength. The nominal strength of the associated connection is calculated as the governing nominal strength of the welded or bolted connection of the tie to the faceplate. The classification of ties as yielding or nonyielding also governs their contribution to the out-of-plane shear strength.

The maximum spacing requirement for ties is influenced by the tie spacing requirement for compression members in ACI 349, Section 7.10.5.2 (ACI, 2006), which specifies the maximum tie spacing for reinforced concrete compression members to be limited to 48 times the tie bar diameter or the least dimension of the compression member. Due to the fundamental differences between behavior of reinforced concrete columns and SC walls, the modular composite specification specifies the following maximum spacing requirement for ties:

> Ties shall have spacing no greater than the section thickness, t_{sc}.

Transfer Length (LTR)

The transfer length, L_{TR}, is defined as the length required to develop strain compatibility between the steel and concrete portions of the composite section if only one of the portions (e.g., concrete) is loaded at the end. The concept of transfer length is similar to load introduction length (length over which steel anchors transfer longitudinal shear in composite sections) discussed in Section I6 of AISC 360-10 (AISC, 2010). Zhang et al. (2014) have analytically investigated transfer lengths for composite SC walls subjected to axial loading on the concrete only at the ends. As shown in Figure 8, strain compatibility (steel strain/concrete strain), or the percentage of composite action, increases with distance from the concrete-only loaded ends. The transfer lengths are typically greater than or equal to $3t_{sc}$ for SC walls with reinforcement ratios of 0.015 to 0.050.

Zhang et al. (2014) have shown that SC walls designed with steel anchor spacing, s, satisfying the faceplate slenderness requirement (Eq. 3) and achieving development lengths, L_d, less than or equal to $3t_{sc}$, have transfer lengths, L_{TR}, greater than or equal to $3t_{sc}$. It is important to note that the development length, L_d, is associated with the shear strength of steel anchors and their ability to develop the yield strength of the faceplate. The transfer length, L_{TR}, is associated with the relative stiffness (force-slip behavior) of the steel anchors and their ability to develop strain compatibility between the faceplates and concrete infill. The transfer lengths are longer than the development lengths for typical SC wall designs (faceplates and steel anchor size and spacing).

The effects of having transfer lengths longer than the development lengths are inconsequential. The design capacities or available strengths of SC walls depend on developing the yield strength of the faceplates, not strain compatibility. The effective stiffness of the composite section depends on strain compatibility but is dominated by the effects of concrete cracking. The effects of having longer transfer lengths (and 75 to 90% composite action) on effective stiffness are marginal compared to the reduction due to concrete cracking (Zhang et al., 2014).

The transfer length, L_{TR}, used in the tie strength and spacing requirements discussed next is limited to $3t_{sc}$. Smaller values are improbable, and larger values will reduce the required tension force, F_{req}, that the ties have to be designed for. Thus, using L_{TR} of $3t_{sc}$ is conservative for the calculation of the required tensile strength described next.

Required Tensile Strength: Delamination Failure

The required tensile strength for ties is based on a postulated failure mode of section delamination or splitting through the concrete thickness of the SC wall. As mentioned earlier, this failure mode has not been observed in any SC wall member or component tests. However, it is possible in the connection regions of SC walls where only one of the two components (faceplates or concrete infill) are directly loaded—for example, in eccentric lap-splice anchorage of SC walls to the concrete basemat (Seo and Varma, 2016). The failure mode is improbable but catastrophic and can be prevented by appropriately designed tie bars. This subsection develops the required (tensile) strength of ties to prevent the occurrence of a postulated splitting or delamination failure mode in the connection and load transfer region of an SC wall.

There are two loading cases, where forces are applied to only one of the two components (faceplates or concrete infill), which can introduce an eccentric moment, M_o, in the SC walls. This eccentric moment needs to be resisted by tie bars. The required tensile strength of the tie bars to resist the eccentric moment can be determined as follows. Case 1 is when the load is applied to concrete only, and the moment is resisted by the composite section. If the compressive forces are applied only to the concrete, they will transfer into the composite section over the transfer length, L_{TR}. Figures 9 and 10 illustrate the forces in the composite section. Over this transfer length, there will be an eccentric moment, M_o, that will have to be resisted by the cross-section without splitting. The resisting moment, M_R, is depicted in Figure 11.

Figure 9 considers a lateral section of the wall length along the transfer length, L_{TR}. The compressive force applied only to the concrete (on the left) spreads to the composite section over the transfer length (on the right). In Figure 10, K_s and K_c are the stiffness of steel and concrete, respectively. Figure 10 establishes that there is an eccentric moment, M_o, resulting from the significant thickness, t_{sc}, of the wall, as well as the fact that the force applied on the left-hand side and the resultant on the right-hand side are not collinear.

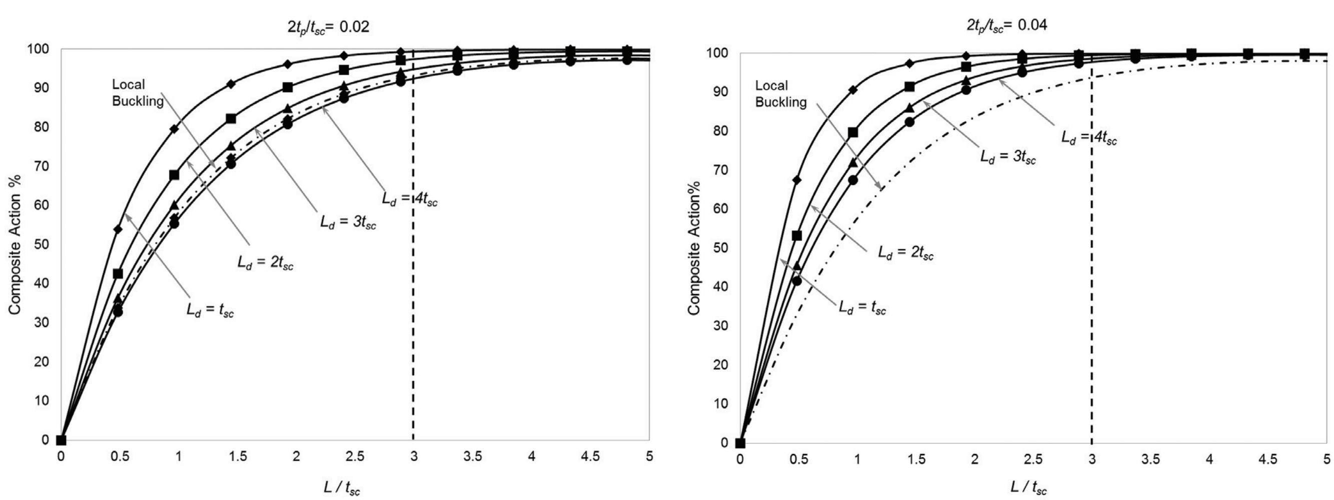


Fig. 8. Development of strain compatibility with distance from member end (Zhang et al., 2014).

The moment M_o is as shown in Equation 10:

$$M_O = \frac{P}{2}\left(\frac{K_s}{K_s + K_c}\right)\left(\frac{t_{sc}}{4}\right) = steel\ plate\ force \times \frac{t_{sc}}{4} \quad (10)$$

Figure 11 shows how the eccentric moment, M_o, is resisted by the tie bars (with area equal to A_{tie}) acting along with the concrete in compression. As shown, the strain diagram is assumed to be linear, but the contribution of the concrete to resist tensile stresses is conservatively neglected. The size of the concrete compression block is also assumed to be very small to simplify calculations, and the contribution of the concrete compression block to the resisting moment, M_R, is also conservatively ignored. As shown by the plan view in Figure 11, a region of the wall (dimensions L_{TR} and s_{tt}) with contributing ties is considered. The resisting moment, M_R, is calculated as shown in Equation 11 by including the contributions of all the ties in the wall region, where σ^i_{req} is the stress in the tie bar and n is the number of tie bars in the transfer length region. The tie bar force required to resist the overturning moment, F^n_{req}, is equal to $A_{tie}\sigma^n_{req}$, and other terms have been defined previously.

$$\begin{aligned} M_R &= \sum_{i=1}^{n-1} 2\left[0.5A_{tie}\left(\frac{i}{n}\sigma^n_{req}\right)(i)(s_{tl})\right] + 2\left[0.25A_{tie}(\sigma^n_{req})(n)(s_{tl})\right] \\ &= \left[\frac{1}{3}\left(\frac{L_{TR}}{s_{tl}}\right)^2 + \frac{1}{6}\right]s_{tl}F^n_{req} \end{aligned} \quad (11)$$

The required tie strength, F_{req}, is estimated by setting M_R equal to M_o. Modular composite specification presents the required tensile strength for tie bars, F_{req}, as follows:

The required tensile strength, F_{req}, for each individual tie shall be determined by Equation 12:

$$F_{req} = \left(\frac{t_p F_y t_{sc}}{4}\right)\left(\frac{s_{tt}}{s_{tl}}\right)\left[\frac{6}{18\left(\frac{t_{sc}}{s_{tl}}\right)^2 + 1}\right]$$

(AISC N690 Eq. A-N9-6) (Eq. 12)

where

F_y = specified minimum yield stress of the faceplate, ksi (MPa)

s_{tl}, s_{tt} = spacing of shear reinforcement in orthogonal directions, in. (mm)

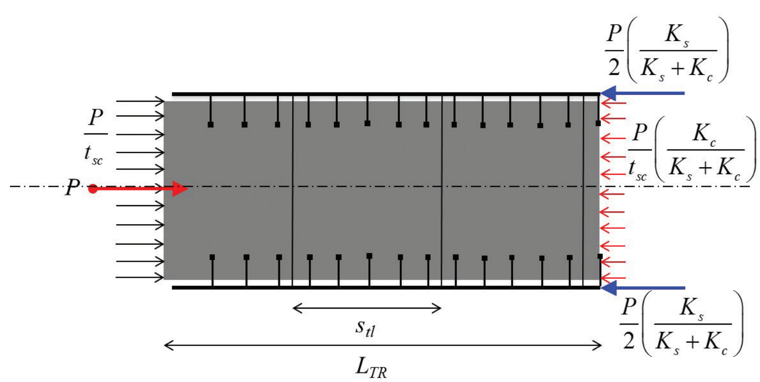


Fig. 9. Load applied to concrete only, resisted by composite section.

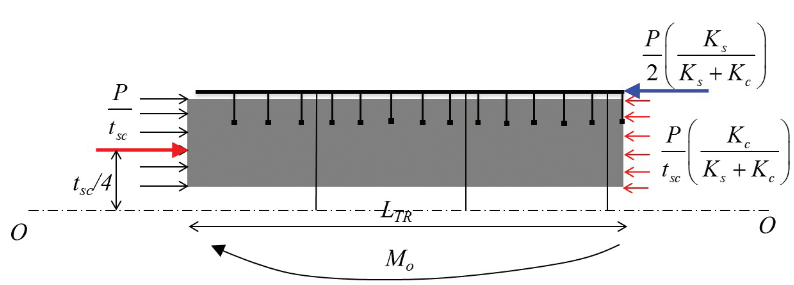


Fig. 10. Eccentric moment, M_o, acting to split section.

t_p = thickness of the faceplate, in. (mm)
t_{sc} = SC section thickness, in. (mm)

Based on the study by Zhang et al. (2014) discussed earlier, a transfer length value of $3t_{sc}$ has been used conservatively in the formulation of Equation 12.

The second case that can give rise to eccentric moments is when the tensile forces are applied only to the faceplates. In this case, the forces will transfer to the composite section until concrete cracking occurs after the transfer length, L_{TR}. Over this transfer length, there will be an eccentric moment, M_o, that will have to be resisted by the cross-section without splitting. Additionally, there may be a case where there is an imbalance in the forces in the thick SC cross-section due to different actual areas and yield strengths of the faceplates. For example, under in-plane shear loading, the composite section typically develops its yield strength, which could correspond to slightly different yield forces in the faceplates due to differences in their actual areas or yield stresses (the modular composite specification requires the faceplates to have same nominal thickness and yield stress). The required force calculated using Equation 12 is applicable for these cases, too. It is important to note that the required force, F_{req}, is a hypothetical demand that has been posited to ensure structural integrity of the SC wall by avoiding the splitting failure of the section. It should not be deducted from the available capacity of the ties.

Contribution of Ties to Out-of-Plane Shear Strength of the SC Wall

The out-of-plane shear behavior of SC walls is similar to that of RC walls, with some differences associated with concrete crack spacing and width due to the discrete nature of the bond between the faceplates and concrete infill achieved using discretely spaced steel anchors and/or ties. Researchers in Japan (Ozaki et al., 2001), South Korea (Hong et al., 2009), and the United States (Varma et al., 2011b; Sener et al., 2016) have performed experiments to study the out-of-plane behavior of SC sections. Sener and Varma (2014) have compared the shear strengths obtained from this experimental database with ACI 349 (ACI, 2006) and other (South Korean, Japanese, and Eurocode) shear strength equations. The comparisons demonstrated that out-of-plane shear failure is a nonductile failure mode, and the concrete contribution to out-of-plane shear strength reduces with increasing wall thickness due to size effects. Based on these observations, the modular composite specification has the following provisions for determining the nominal out-of-plane shear strength of the SC walls:

> The nominal out-of-plane shear strength per unit width shall be established by one of the following:
>
> 1. Conducting project-specific large-scale out-of-plane shear tests.
> 2. Using applicable test results.

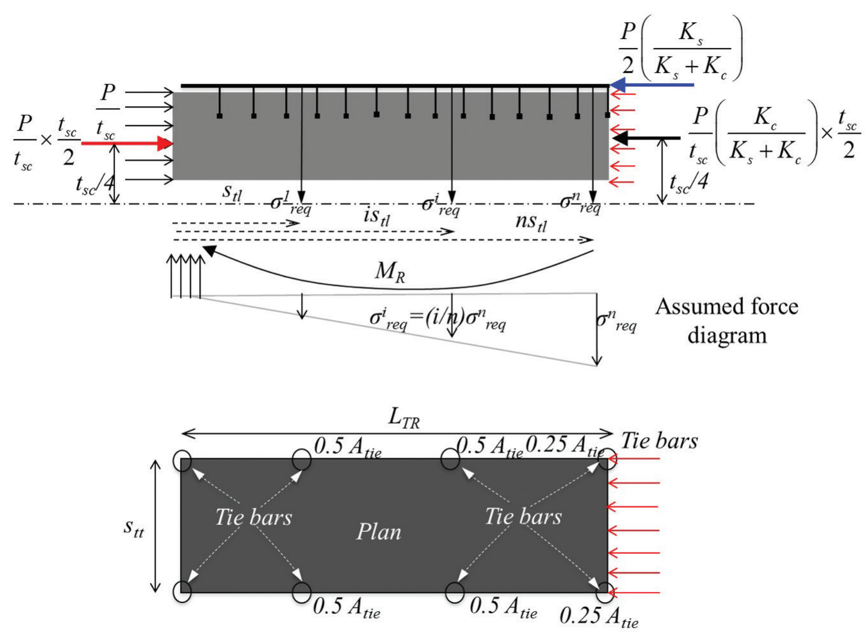


Fig. 11. Resisting moment, M_R.

3. Using the provisions of this section (the provisions mentioned in the corresponding section of the modular composite specification).

The modular composite specification addresses the nonductile nature of the failure mode by defining suitable values for resistance factor ($\phi_{vo} = 0.75$) based on the reliability analysis presented in Sener and Varma (2014). The nominal shear strength of the SC walls depends on the spacing of shear reinforcement and the classification of shear reinforcement as yielding or nonyielding. If the shear reinforcement spacing is less than $t_{sc}/2$, the nominal out-of-plane shear strength will include out-of-plane shear contributions from concrete as well as steel, when ties act as shear reinforcement.

The modular composite specification addresses the size effect by limiting out-of-plane shear contribution of concrete in SC walls to $1.5\sqrt{f_c'}$ (in psi units) $\left[0.05\sqrt{f_c'} \text{ (in ksi units)}\right]$. The shear reinforcement contribution is based on the well-known mechanism of a shear or flexure-shear crack passing through yielding-type shear reinforcement ties and engaging them in axial tension. The classification of the shear reinforcement (or ties) as yielding and the determination of its available axial tensile strength are important for this calculation. The modular composite specification limits the maximum possible contribution of the shear reinforcement to the out-of-plane shear strength to $8\sqrt{f_c'}A_c$ (in psi units) $\left[0.25\sqrt{f_c'}A_c \text{ (in ksi units)}\right]$, where A_c is the area of concrete per unit width. This upper limit is influence by the similar limit in ACI 349 (ACI, 2006).

According to the modular composite specification, the out-of-plane shear strength for an SC wall with shear reinforcement spaced not greater than $t_{sc}/2$ is determined as follows:

> The nominal out-of-plane shear strength per unit width for SC panel sections with shear reinforcement spacing no greater than half of the section thickness shall be calculated as per Equation 13:
>
> $$V_{no} = V_{conc} + V_s \quad \text{(AISC N690 Eq. A-N9-20)} \quad (13)$$
>
> where
>
> $$V_{conc} = 0.05\sqrt{f_c'}t_c l \quad \text{(AISC N690 Eq. A-N9-21)} \quad (14)$$
>
> $$V_{conc} = 0.13\sqrt{f_c'}t_c l \quad \text{(AISC N690 Eq. A-N9-21M)} \quad (14M)$$
>
> $$V_s = \xi p_s F_t \left(\frac{1}{s_{tt}}\right) \le 0.25\sqrt{f_c'}t_c l \quad \text{(AISC N690 Eq. A-N9-22)} \quad (15)$$
>
> $$V_s = \xi p_s F_t \left(\frac{1}{s_{tt}}\right) \le 0.67\sqrt{f_c'}t_c l \quad \text{(AISC N690 Eq. A-N9-22M)} \quad (15M)$$
>
> F_t = nominal tensile strength of ties, kips (N)
>
> f_c' = concrete compressive strength, ksi (MPa)
>
> l = unit width, 12 in./ft.(1000 mm/m)
>
> p_s = t_c/s_{tl}
>
> s_{tl} = spacing of shear reinforcement along the direction of one-way shear, in. (mm)
>
> s_{tt} = spacing of shear reinforcement transverse to the direction of shear, in. (mm)
>
> t_c = concrete infill thickness
> = $t_{sc} - 2t_p$, in. (mm)
>
> ξ = 1.0 for yielding shear reinforcement
> = 0.5 for nonyielding shear reinforcement

The concrete contribution, V_{conc}, is determined per Equation 14, and steel contribution, V_s, is determined per Equation 15. For nonyielding shear reinforcement with spacing less than or equal to $t_{sc}/2$, it is possible that the concrete shear or flexure shear crack will engage all the individual shear reinforcements that it will pass through. However, it is unclear whether these individual shear reinforcements will be able to develop their individual design strengths before one of them—the one with the largest axial force—fails in a nonductile manner. Therefore, the shear reinforcement contribution in Equation 15 has been reduced to one-half.

If the spacing of the yielding shear reinforcement is greater than $t_{sc}/2$, the maximum out-of-plane shear strength is limited to the greater of (1) the concrete shear strength contribution or (2) the shear reinforcement contribution alone. This is based on the ability of the SC section to develop an internal truss mechanism for equilibrium. The strength of this truss mechanism is limited to that of the tie shear reinforcement. The concrete and steel contributions cannot be added for shear reinforcement spacing greater than $t_{sc}/2$ because the shear or flexural-shear crack may not pass through more than one tie.

Per the modular composite specification, the out-of-plane shear strength for an SC wall with shear reinforcement spaced greater than $t_{sc}/2$ is determined as follows:

> The nominal out-of-plane shear strength per unit width for SC panels with shear reinforcement spaced greater than half the section thickness shall be the greater of V_{conc} and V_s. V_{conc} shall be calculated using Equation 14, and V_s shall be calculated using Equation 15 taking both ξ and p_s as 1.0.

The behavior of nonyielding shear reinforcement with spacing greater than half the wall thickness will be same as that of yielding shear reinforcement spaced at more than half the wall thickness.

Interaction of Out-of-Plane Shear Forces

The out-of-plane shear demands in both x and y directions, V_{rx} *and* V_{ry}, rely on using the same tie shear reinforcement for the steel contribution, V_s, to the corresponding available out-of-plane shear strengths, V_{cx} and V_{cy}. Both out-of-pane shear demands, V_{rx} and V_{ry}, subject the ties to axial tension demand after concrete cracks and its contribution, $V_{c\ conc}$, in the respective directions is exceeded. Additionally, shear reinforcement and steel anchors are subject to interfacial shear demands in both the x and y directions. The modular composite specification provides Equation 16 to check the interaction of out-of-plane shear and interfacial shear demands on an SC wall as follows:

> If the required out-of-plane shear strength per unit width for both the x and y axes, V_{rx} and V_{ry}, is greater than the available out-of-plane shear strength contributed by the concrete per unit width of the SC panel section, $V_{c\ conc}$, and the out-of-plane shear reinforcement is spaced no greater than half the section thickness, the interaction of out-of-plane shear forces is limited by Equation 16:
>
> $$\left[\left(\frac{V_r - V_{c\ \mathrm{conc}}}{V_c - V_{c\ \mathrm{conc}}}\right)_x + \left(\frac{V_r - V_{c\ \mathrm{conc}}}{V_c - V_{c\ \mathrm{conc}}}\right)_y\right]^{5/3} + \left[\frac{\sqrt{V_{rx}^2 + V_{ry}^2}\Big/(0.9t_{sc})}{\Psi\left(lQ_{cv}^{\mathrm{avg}}/s^2\right)}\right]^{5/3} \le 1.0 \qquad \text{(AISC N690 Eq. A-N9-23)} \quad (16)$$
>
> where
>
> V_c = available out-of-plane shear strengths per unit width of SC panel section in local x (V_{cx}) and y (V_{cy}) directions, kip/ft. (N/mm)
>
> $V_{c\ conc}$ = available out-of-plane shear strength contributed by concrete per unit width of SC panel section, kip/ft. (N/mm)
>
> V_r = required out-of-plane shear strength per unit width of SC panel section in local x (V_{rx}) and y (V_{ry}) directions using LRFD or ASD load combinations, kip/ft. (N/mm)
>
> Q_{cv}^{avg} = weighted average of the available interfacial shear strengths of ties and steel anchors while accounting for their respective tributary areas and numbers (Eq. 17), kips (N)
>
> l = unit width, 12 in./ft. (1000 mm/m)
>
> s = spacing of steel anchors, in. (mm)
>
> t_{sc} = SC section thickness, in. (mm)
>
> x = subscript relating symbol to the local x-axis, as defined earlier
>
> y = subscript relating symbol to the local y-axis, as defined earlier
>
> ψ = 1.0 for panel sections with yielding shear reinforcement and yielding steel anchors
>
> = 0.5 for panel sections with either nonyielding shear reinforcement or nonyielding steel anchors for design in accordance with LRFD
>
> V_c = $\phi_{vo}V_{no}$, kip/ft (N/m), where V_{no} is nominal out-of-plane shear strength per unit width of SC panel section and $\phi_{vo} = 0.75$
>
> $V_{c\ conc}$ = $\phi_{vo}V_{conc}$, kip/ft (N/m), where V_{conc} is nominal out-of-plane shear strength contributed by concrete per unit width and $\phi_{vo} = 0.75$

The interaction equation, Equation 16, is based on the shear-tension interaction equation in ACI 349 (ACI, 2006) Appendix D, Commentary RD.7, which is applicable to concrete anchors, or connectors, with ductile and nonductile limit states. In the first part of the interaction equation, the numerators are the tensile force demands in the tie bars, which are calculated as the portions of the out-of-plane shear demands greater than the corresponding concrete contribution, $V_{c\ conc}$. The denominators are the available strength contributions of the ties, V_s. The second term in the interaction equation accounts for the shear demand in the ties and steel anchors due to their participation in resisting interfacial shear demands, which are also the result of out-of-plane shear demands as discussed previously. The numerator is the vector sum of the out-of-plane shear demands, V_{rx} and V_{ry}, obtained by algebraic manipulation of Equation 8.

The denominator is the weighted average of the shear strength contributions of ties and steel anchors, Q_{cv}^{avg}, and can be calculated using Equation 17:

$$Q_{cv}^{avg} = \frac{n_{et}Q_{cv}^{tie} + n_{es}Q_{cv}}{n_{et} + n_{es}} \qquad (17)$$

where

Q_{cv}^{tie} = available interfacial shear strength of tie bars, kip (N)

n_{et} = effective number of ties contributing to a unit cell

n_{es} = effective number of shear connectors contributing to a unit cell

and where the unit cell is the quadrilateral region defined by a grid of four adjacent ties.

For example, Figure 12 illustrates the unit cell for an SC wall of thickness 36 in. (900 mm), with ties spaced at 36 in. (900 mm) and steel anchors spaced at 9 in. (225 mm). As shown in the figure, the tie bars at the corners participate in four adjoining unit cells, and the steel anchors at the boundaries participate in two adjacent unit cells. The steel anchors within the boundaries of the unit cells contribute fully. For the example shown in Figure 12, the effective number of tie bars contributing to the unit cell, n_{et}, is equal to 1, and the effective number of steel anchors, n_{es}, is equal to 15 [namely, (1)(9) + (0.5)(12) = 15].

When the spacing of the shear reinforcement is greater than $t_{sc}/2$, the nominal out-of-plane shear strength is governed by the greater of the steel and concrete contributions as discussed previously. When the steel contribution is greater than the concrete contribution, Equation 16 will not include the concrete contribution. The modular composite specification discusses this requirement as follows:

> If the available strength, V_c, is governed by the steel contribution alone and the out-of-plane shear reinforcement is spaced greater than half the section thickness, $V_{c\,conc}$ shall be taken as zero in Equation 16.

When one of the out-of-plane shear demands, V_{cx} or V_{cy}, is less than the concrete contribution, there will be no interaction of out-of-plane shear demands. For shear reinforcement spaced greater than $t_{sc}/2$, if the concrete contribution is more than the shear reinforcement contribution, the concrete infill will be subject to two-way shear (punching shear), which will be resisted by perimeter of the unit cell for the SC panel section.

SUMMARY AND CONCLUSIONS

This paper discussed the minimum requirements and section detailing provisions for steel-plate composite (SC) walls in safety-related nuclear facilities as discussed in Appendix N9 to AISC N690s1 (AISC, 2015). The minimum requirements—including the minimum and maximum section thickness, t_{sc}; faceplate thickness, t_p; reinforcement ratio, ρ; concrete strength, f_c'; steel yield stress, F_y—were based primarily on the experimental database of SC walls tested under different loading conditions and practical concerns related to fabrication and handling requirements. The section detailing provisions include requirements for size and spacing of stud anchors and ties, which provide composite action, structural integrity, interfacial shear resistance, and out-of-plane shear strength to the SC wall design. These steel anchors and ties are classified as yielding or nonyielding type based on behavior and failure mode.

The stud anchor detailing provisions are based on requirements to prevent local buckling before yielding of

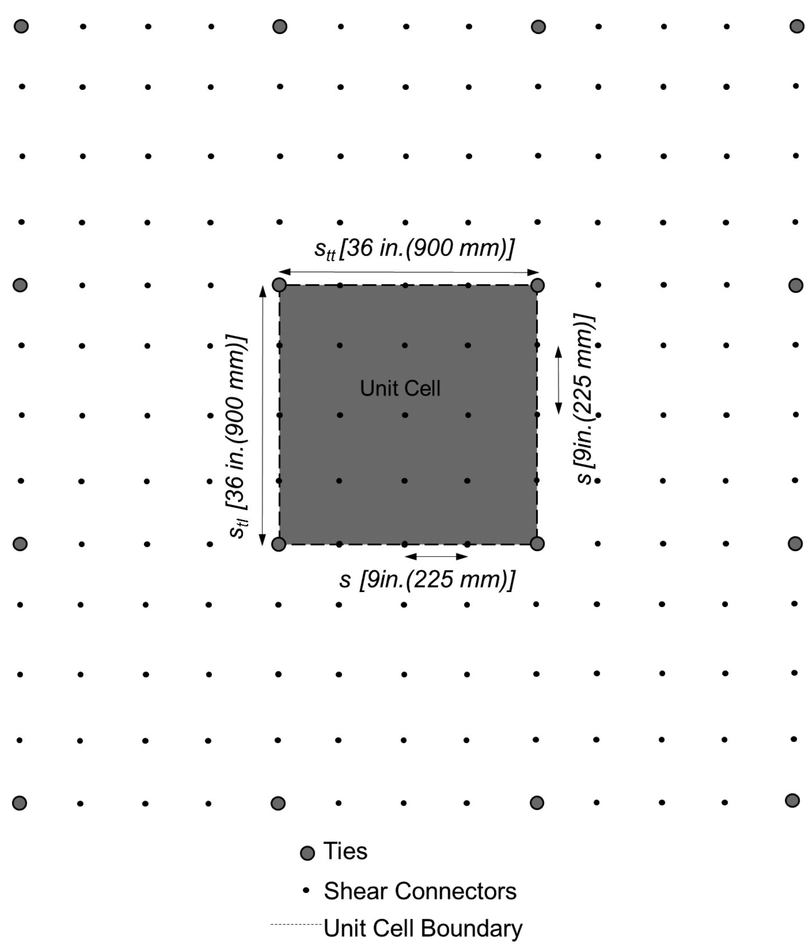


Fig. 12. Unit cell for calculating Q_{cv}^{avg}.

Table 2. Parameters for Design Example	
Parameter	**Value**
Faceplate thickness, t_p	0.5 in.
SC section thickness, t_{sc}	56 in.
Steel minimum specified yield stress, F_y	50 ksi (A 572 Gr. 50)
Concrete minimum specified compressive strength, f'_c	5 ksi
Steel anchor (¾-in.-diameter steel-headed stud anchors) spacing, s_L (s_T)	6 in.
Tie (0.5-in. × 6-in. flat bars) spacing, s_{tt} (s_{tl})	24 in.

the faceplates in compression, having development lengths less than three times the wall section thickness, and preventing interfacial shear failure from occurring before out-of-plane shear failure. The tie detailing provisions are based on the requirements to prevent section delamination through the plain concrete in between the steel faceplates and out-of-plane shear strength. All these provisions distinguish between yielding and nonyielding types of steel anchors and ties because of the differences in resistance distribution along the length.

The paper also discussed AISC N690s1 provisions related to the out-of-plane shear strength of SC walls and the contribution of ties depending on their classification—yielding or nonyielding—and spacing along the length. The interaction of different out-of-plane shear demands in the *x*- and *y*-directions on the design of steel anchors and ties was also discussed. All these minimum requirements and section detailing provisions have to be checked for SC wall sections used in safety-related nuclear facilities before the remaining provisions of AISC N690s1 (AISC, 2015) can be applied for their design.

DESIGN EXAMPLE

This section presents sample calculations for a typical SC wall used in a safety-related nuclear facility. The wall dimensions are established in the plant layout process and serve as the initial iteration for the design. Table 2 summarizes the geometric and material details of the sample SC wall that satisfies all the minimum requirements of Table 1. Table 3 summarizes the calculations associated with the section detailing requirements. The steel anchor and tie bar spacing meets the faceplate slenderness requirement of Equation 2. Steel anchors additionally meet the spacing required to develop the yield strength of the steel faceplate over the development length (Equation 5) and the spacing required to prevent interfacial shear failure before out-of-plane shear failure (Equation 8). The tie bars have tensile strength greater than the tensile demand, F_{req} (Equation 12). Steel-headed stud anchors used are classified as yielding shear connectors. Tie bars are of the yielding type (Equation 9). The tie bar spacing meets the spacing requirement.

REFERENCES

ACI (2006), *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*, ACI 349-06, American Concrete Institute, Farmington Hills, MI.

AISC (2010), *Specification for Structural Steel Buildings*, ANSI/AISC 360-10, American Institute of Steel Construction, Chicago, IL.

AISC (2012), *Specification for Safety-Related Steel Structures for Nuclear Facilities*, ANSI/AISC N690, American Institute of Steel Construction, Chicago, IL.

AISC (2015), *Specification for Safety-Related Steel Structures for Nuclear Facilities Including Supplement No. 1*, AISC N690s1, American Institute of Steel Construction, Chicago, IL.

AISC (2016), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341, Public Review Draft, February 2016, American Institute of Steel Construction, Chicago, IL.

Akiyama, H., Sekimoto, H., Masaki F., Kazuo N. and Kiyoshi, H. (1991), "A Compression and Shear Loading Tests of Concrete Filled Steel Bearing Wall," *Transactions of the 11th Structural Mechanics in Reactor Technology (SMiRT-11) Conference*, pp. 323–328.

Bhardwaj, S.R., Varma, A.H. and Al-Shawaf, T. (2015), "Outline of Specification for Composite SC Walls in Nuclear Facilities," *Proceedings of the ASCE/SEI Structures Congress*, ASCE, Portland, OR, pp. 1021–1031.

Bhardwaj, S.R. and Varma, A.H. (2016), "Effect of Imperfections on the Compression Behavior of SC Walls," *Proceedings of the Annual Stability Conference*, Structural Stability Research Council, Orlando, FL, April 12–15, AISC, Chicago, IL.

Table 3. SC Example Minimum Requirements and Detailing Provisions Check

Requirement	Check
Minimum requirements (Table 1)	t_p, t_{sc}, F_y and f_c' meet the minimum requirements. Reinforcement ratio, ρ, within the limits.
Faceplate slenderness requirement (Eq. 2)	$\dfrac{b}{t_p} \le 1.0\sqrt{\dfrac{E_s}{F_y}}$ $\quad 12 \le 24$ Steel anchor and tie spacing meet the requirement.
Steel anchor classification	Steel-headed stud anchors are yielding steel anchors.
Steel anchor spacing: development length (Eq. 5)	$L_d = 3t_{sc}; Q_{cv} = 18.7$ kips (83.2 kN) [AISC 360, Section I8.3] $s \le c_1\sqrt{\dfrac{Q_{cv}L_d}{T_p}}$ $\quad$ 6 in. ≤ 11.2 in. (150 mm ≤ 285 mm) Steel anchor spacing meets the development length requirement.
Tie bar classification and spacing (Eq. 9)	$s_{tt} \le t_{sc} \Rightarrow$ 24 in. < 56 in. (610mm < 1420 mm) Ties meet the spacing requirement. $F_{ny} \le 0.8F_{nr} \Rightarrow$ 150 kip < 156 kip < (667 kN 694 kN) Tie bars (yielding type) connected to faceplates using complete-joint-penetration welds.
Required tension strength: delamination failure (Eq. 12)	$F_{req} = 21.2$ kips (94.3 kN) Available tensile strength of ties is greater than F_{req}.
Contribution of ties to out-of-plane shear strength of SC walls, V_s (Eqs. 13, 14, 15)	$V_s = 172$ kips (765 kN); $V_{conc} = 74$ kips (329 kN) $\Rightarrow V_c = 184$ kips (818 kN) (AISC, 2015)
Steel anchor spacing: interfacial shear (Eq. 8)	$s \le c_1\sqrt{\dfrac{Q_{cv}I}{V_c/0.9t_{sc}}}$ $\quad$ 6 in. < 7.8 in. (152 mm < 198 mm) Spacing of steel anchors meets the requirement for preventing interfacial shear failure.

Booth, P.N., Varma, A.H., Sener, K. and Malushte, S. (2015a), "Flexural Behavior and Design of Steel-Plate Composite (SC) Walls for Accident Thermal Loading," *Nuclear Engineering and Design*, Vol. 295, pp. 817–828.

Booth, P.N., Varma, A.H., Sener, K. and Mori., K. (2015b), "Seismic Behavior and Design of a Primary Shield Structure Consisting of Steel-Plate Composite (SC) Walls," *Nuclear Engineering and Design*, Vol. 295, pp. 829–842.

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, pp. 75–87.

Bruhl, J. and Varma, A.H. (2015), "Summary of Blast Tests on Steel-Plate Reinforced Concrete Walls," *Proceedings of the ASCE/SEI Structures Congress*, ASCE, Portland, OR, April, pp. 151–159.

Choi, B.J. and Han, H.S. (2009), "An Experiment on Compressive Profile of the Unstiffened Steel Plate-Concrete Structures under Compression Loading," *Steel and Composite Structures*, Vol. 9, No. 6, pp. 519–534.

DCD (2011), Design Control Document for the AP1000, *U.S. Nuclear Regulatory Commission*, Washington, DC.

DCD (2013). US-APWR Design Control Document and Environmental Report, *U.S. Nuclear Regulatory Commission*, Washington, DC.

Epackachi, S., Nguyen, N., Kurt, E., Whittaker, A. and Varma, A. (2015), "In-Plane Seismic Behavior of Rectangular Steel-Plate Composite Wall Piers," *Journal of Structural Engineering*, ASCE, Vol. 141, No. 7.

Hong, S., Kim, W., Lee, K., Hong, N.K. and Lee, D. (2009), "Out-of-Plane Shear Strength of Steel Plate Concrete Walls Dependent on Bond Behavior," *Transactions of the 20th International Conference on Structural Mechanics in Reactor Technology* (SMiRT-20), Div-6, Paper 1855, Espoo, Finland, pp. 1–10.

Kanchi, M., et al. (1996), "Experimental Study on a Concrete Filled Steel Structure: Part 2. Compressive Tests (1)," [in Japanese], *Summaries of Technical Papers of Annual Meeting*, Architectural Institute of Japan, B-2, Structures II, Structural Dynamics Nuclear Power Plants, pp. 1,071–1,072.

Kurt, E.G., Varma, A.H., Booth, P.N. and Whittaker, A., (2016), "In-Plane Behavior and Design of Rectangular SC Wall Piers without Boundary Elements," *Journal of Structural Engineering*, ASCE, Vol. 142, No. 6.

NRC (2007), "Barrier Design Procedures," *Standard Review Plan 3.5.3*, Report NUREG-0800, Revision 3, U.S. Nuclear Regulatory Commission, Washington, DC.

Ollgaard, J.G., Slutter, R.G. and Fisher, J.W. (1971), "Shear Strength of Stud Shear Connections in Lightweight and Normal Weight Concrete," *Engineering Journal*, AISC, Vol. 8, No. 2, pp. 55–64.

Ozaki, M., Akita, S., Niwa, N., Matsuo, I. and Usami, S. (2001), "Study on Steel Plate Reinforced Concrete Bearing Wall for Nuclear Power Plants: Part 1, Shear and Bending Loading Tests of SC Walls," *Transactions of the 16th International Conference on Structural Mechanics in Reactor Technology* (SMiRT-16), Washington, DC, Paper ID 1554, pp. 1–10.

Ozaki, M., Akita, S., Osuga, H., Nakayama, T. and Adachi, N. (2004), "Study on steel Plate Reinforced Concrete Panels Subjected to Cyclic In-Plane Shear," *Nuclear Engineering and Design*, Selected and Updated Papers from the 16th International Conference on Structural Mechanics in Reactor Technology, Washington DC, Vol. 228, Nos. 1–3, pp. 225–244.

Ravindra, M.K. and Galambos, T.V. (1978), "Load and Resistance Factor Design for Steel," *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, pp. 1,337–1,353.

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

Sener, K., Varma, A.H., Booth, P.N. and Fujimoto, R. (2015a), "Seismic Behavior of a Containment Internal Structure Consisting of Composite SC Walls," *Nuclear Engineering and Design*, Vol. 295, pp. 804–816.

Sener, K.C., Varma, A.H. and Deniz, A. (2015b), "Steel-Plate Composite SC Walls: Out-of-Plane Flexural Behavior, Database and Design," *Journal of Constructional Steel Research*, Vol. 108, pp. 46–59.

Sener, K., Varma, A.H. and Seo, J. (2016), "Experimental and Numerical Investigations of the Shear Behavior of Steel-Plate Composite (SC) Beams without Shear Reinforcement," *Engineering Structures*, Vol. 127, pp. 495–509.

Seo, J., Varma, A.H., Sener, K. and Ayhan, D. (2016), "Steel-Plate Composite (SC) Walls: In-Plane Shear Behavior, Database, and Design," *Journal of Constructional Steel Research*, Vol. 119, pp. 202–215.

Seo, J. and Varma, A.H. (2017), "Experimental Behavior and Design of Steel Plate Composite-to-Reinforced Concrete Lap Splice Connections," *Journal of Structural Engineering*, ASCE, published online Jan. 30.

Usami, S., Akiyama, H., Narikawa, M., Hara, K., Takeuchi, M. and Sasaki, N. (1995), "Study on a Concrete Filled Steel Structure for Nuclear Plants (Part 2). Compressive Loading Tests on Wall Members," *Transactions of the 13th International Conference on Structural Mechanics in Reactor Technology* (SMiRT-13), pp. 21–26.

Varma, A.H. (2000), "Seismic Behavior, Analysis and Design of High Strength Square Concrete Filled Tube (CFT) Columns," Ph.D. Dissertation, Lehigh University, Bethlehem, PA.

Varma, A.H., Malushte, S.R. and Lai, Z. (2015), "Modularity & Innovation Using Steel-Plate Composite (SC) Walls for Nuclear and Commercial Construction," *Proceedings of the 11th International Conference on Advances in Steel-Concrete Composite Structures (ASCCS)*, Beijing, China.

Varma, A.H., Malushte, S.R., Sener, K.C. and Lai, Z. (2014), "Steel-Plate Composite (SC) Walls for Safety Related Nuclear Facilities: Design for In-Plane Forces and Out-of-Plane Moments," *Nuclear Engineering and Design*, Vol. 269, pp. 240–249.

Varma, A.H., Zhang, K., Chi, H., Booth, P.N., and Baker, T. (2011a), "In-Plane Shear Behavior of SC Composite Walls: Theory vs. Experiment." *Transactions of the 21st International Conference on Structural Mechanics in Reactor Technology (SMiRT 21)*, New Delhi, India, IASMIRT, Paper ID 764, pp. 1–10

Varma, A.H., Sener, K.C., Zhang, K., Coogler, K. and Malushte, S.R. (2011b), "Out-of-Plane Shear Behavior of SC Composite Structures," *Transactions of the 21st International Conference on Structural Mechanics in Reactor Technology Conference* (*SMiRT 21*), New Delhi, India, IASMIRT, Paper ID 763, pp. 1–10.

Varma, A.H., Zhang, K. and Malushte, S. (2013), "Local Buckling of SC Composite Walls at Ambient and Elevated Temperatures," *Transactions of the 22nd International Conference on Structural Mechanics in Reactor Technology (SMiRT 22),* IASMIRT, pp. 1–10.

Zhang, K. (2014), "Axial Compression Behavior and Partial Composite Action of SC Walls in Safety-Related Nuclear Facilities," Ph.D. Dissertation, Purdue University, West Lafayette, IN.

Zhang, K., Varma, A.H., Malushte, S.R. and Gallocher, S. (2014), "Effect of Shear Connectors on Local Buckling and Composite Action in Steel Concrete Composite Walls," *Nuclear Engineering and Design*, Vol. 269, pp. 231–239.