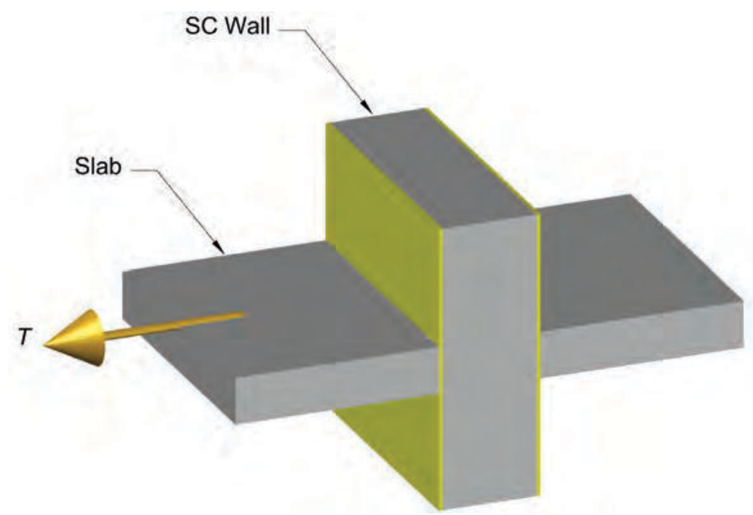


Fig. 11-25. Axial tension strength demand for RC slab-to-SC wall joint.

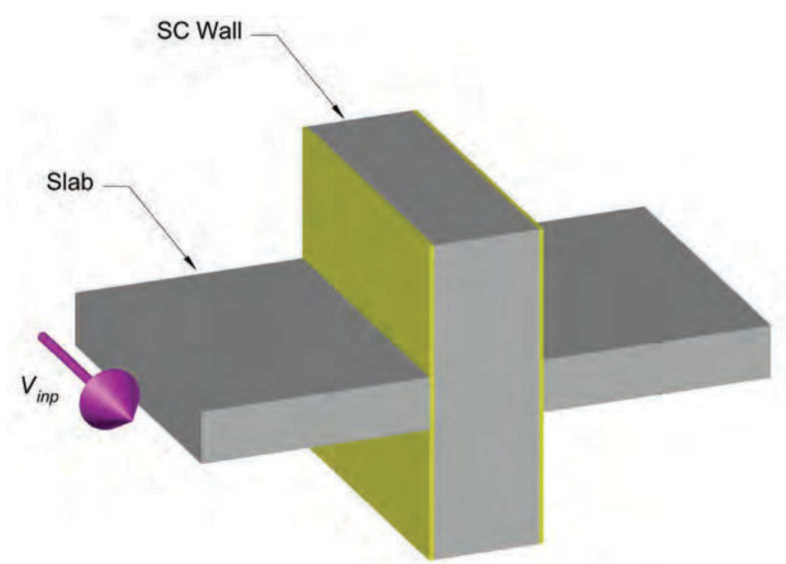


Fig. 11-26. In-plane shear strength demand for RC slab-to-SC wall joint.

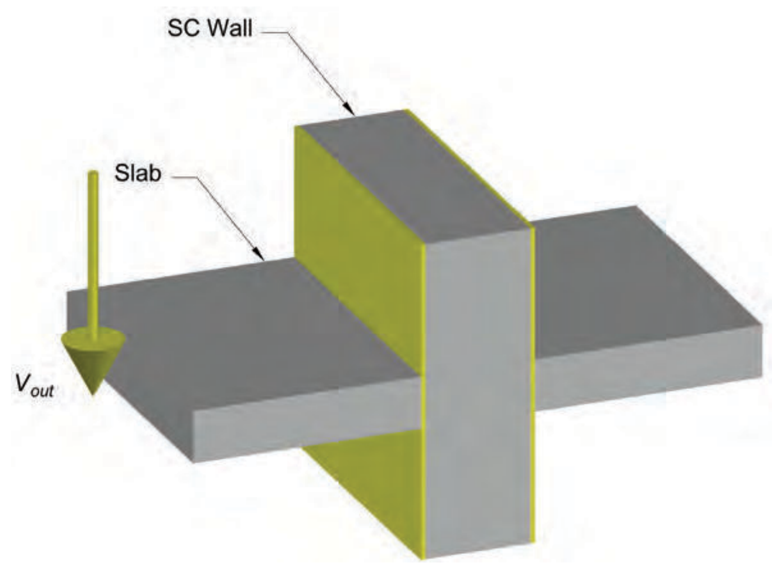


Fig. 11-27. Out-of-plane shear strength demand for RC slab-to-SC wall joint.

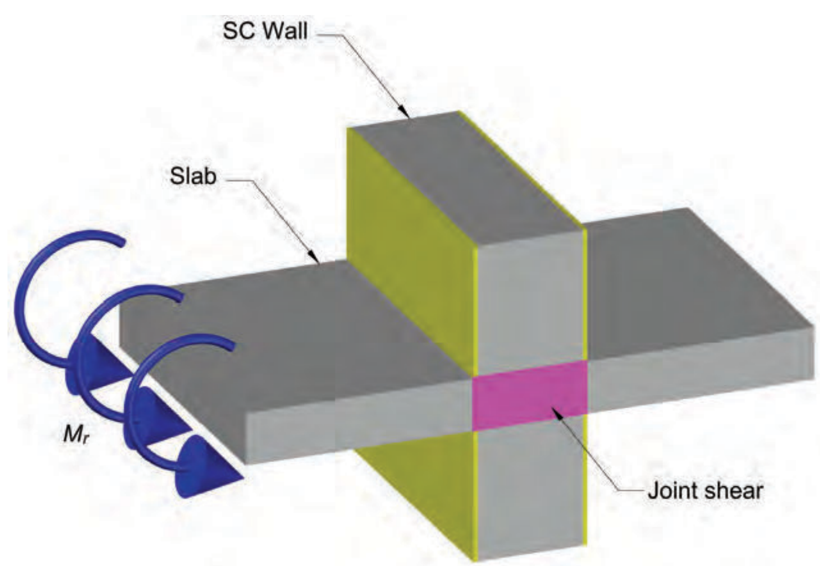


Fig. 11-28. Out-of-plane flexural demand and joint shear demand for RC slab-to-SC wall joint.

Chapter 12
Impactive and Impulsive Loads

The design of SC walls for safety-related nuclear facilities may need to be checked for impactive loads, such as tornado-borne missiles, whipping pipes, aircraft missiles, or other internal and external missiles, and for impulsive loads, such as jet impingement loads, blast pressure, compartment pressurization, or jet shield reactions. The effects for impactive and impulsive loads are considered in extreme environmental and abnormal load combinations concurrent with other loads. These effects are permitted to be determined using inelastic analysis with limits on the ductility ratio demand, μ_{dd}, defined as the ratio of maximum displacement from analysis to the effective yield displacement in ANSI/AISC N690, Equation A-N9-7, as given in Table 12-1. Yield displacement is established using the cross-sectional effective flexural stiffness for analysis, EI_{eff}, according to ANSI/AISC N690, Equation A-N9-8.

The ductility of the member at failure is more dependent on the failure mode than on the type of loading. This is observed in the values of ductility ratios in Table 12-1. A ductility ratio greater than 1.0 is permitted for brittle failure modes because even brittle structures have been observed to display some inelastic deformation capabilities.

The available strength of SC walls for impulsive and impactive loads may be governed by flexural yielding or out-of-plane shear failure. ANSI/AISC N690 Appendix N9 classifies SC walls as flexure-controlled if the available strength for the limit state of flexural yielding is less than the available strength for the limit state of out-of-plane shear failure by at least 25%. Otherwise, SC walls are classified as shear-controlled. This requirement is based on the fact that the increase in strength under rapid strain exhibited by steel is better established than that for the shear strength of concrete. Careful consideration should be given to special cases where the flexural behavior goes significantly past yield into the strain-hardening range. In such cases, the margin for available strength in shear over the available strength in flexure should be higher.

ANSI/AISC N690 Appendix N9 permits dynamic increase factors (DIFs) based on the strain rates involved to be applied to static material strengths of steel and concrete for purposes of determining section strength. However, the DIF values are limited by ANSI/AISC N690, Table N9.1.1. The DIF is limited to 1.0 for all materials where the dynamic load factor associated with the impactive or impulsive loading is less than 1.2 (NRC, 2001). Plastic hinge rotation capacity need not be considered if the deformation limit is kept under 10 for flexure controlled sections (Varma et al., 2011c). For the axial ductility ratio, the effective yield displacement is calculated using the cross-sectional effective axial stiffness. This axial stiffness is calculated using the material elastic modulus and the model section thickness calibrated in accordance with ANSI/AISC N690, Section N9.2.3.

At the rates of strain that are characteristic of certain impactive and impulsive loads, both the concrete and the structural steel exhibit elevated yield strengths, while the strain at the onset of strain hardening and the tensile strength increase slightly. The modulus of elasticity remains nearly constant. The DIF values given in ANSI/AISC N690 represent the ratio of dynamic to static yield strengths or ultimate strengths, and are direct functions of the strain rates involved. The values have been taken from NEI 07-13 (NRC, 2011).

Response of SC walls subjected to impulsive loads can be determined by one of the following methods:

a. The dynamic effects of impulsive loads are considered based on approximation of the wall panel as a single degree of freedom (SDOF) elastic, perfectly plastic system, where the resistance function and limiting ductility are defined as in ANSI/AISC N690, Section N9.1.6b. System response is determined by either a nonlinear time history analysis or, for well-defined impulse functions, rectangular and triangular pulses, selected from established response charts such as those in Biggs (1964).

b. The dynamic effects of impulsive loads are considered based on the approximation of the wall panel as a SDOF system with bilinear stiffness. System response is determined by a nonlinear time history analysis. Either the ductility is limited as defined in ANSI/AISC N690, Section N9.1.6b, or the plate principal strain may be limited to 0.05.

c. The dynamic effects of impulsive loads are considered by performing a nonlinear FE analysis. The plate principal strain is limited to 0.05.

In cases of impulsive and impactive loads that are expected to deform the structure beyond its elastic limits, the usefulness of load combinations given in ANSI/AISC N690, Section NB, is rather limited. These combinations do not provide any means of accounting for the additional work done by the static loads, which may be present as the structure deforms beyond its effective yield point.

If the energy balance method is used, only the energy available to resist the impactive and impulsive loads should be used. Alternatively, if an elastoplastic analysis is performed, the effective ductility ratio, μ', to be used in the analysis for impactive and impulsive loading is given by Equation 12-1:

Table 12-1. Ductility Ratio Demand	
Description of Element	**Ductility Ratio Demand, μ_{dd}**
Flexure controlled SC walls	$\mu_{dd} \le 10$
Shear controlled SC walls (yielding shear reinforcement spaced at section thickness divided by two or smaller)	$\mu_{dd} \le 1.6$
Shear controlled SC walls (other configurations of yielding or nonyielding shear reinforcement)	$\mu_{dd} \le 1.3$
For axial compressive loads	$\mu_{dd} \le 1.3$

$$\mu' = \frac{\mu_{dd} D_y - D_s}{D_y - D_s} \tag{12-1}$$

where

D_s = displacement due to static loads, in. (mm)

D_y = displacement at yield, in. (mm)

μ_{dd} = ductility factor

The effective ductility ratio is to be used in conjunction with the effective available resistance, which is equal to the available resistance less the force due to static loads. Instead of a more rigorous analysis, seismic forces can be conservatively treated as equivalent static loads in the analysis for determining the adequacy of the structure for impactive and impulsive loading.

Design of SC walls for impactive loads needs to satisfy the criteria for both local effects and overall structural response. Local impact effects include perforation of the SC wall. For a structural system to act as a missile barrier, the member needs to be sufficiently thick to prevent perforation. Bruhl et al. (2015a) have presented a three-step approach to design an individual SC wall for a specific missile. The evaluation procedure is explained in Figure 12-1. The front surface faceplate is conservatively neglected in this analysis. Thus, impact of a projectile on the concrete dislodges a conical concrete plug, which in turn impacts the rear faceplate.

Step 1. The design method involves first selecting a concrete wall thickness, t_c. An existing wall thickness can be used to verify the protection afforded by a given wall. For new designs, the concrete thickness can be obtained from governing design requirements or 70% of the thickness for an RC wall determined using DOE-STD-3014 (DOE, 2006) or NEI 07-13 (NRC, 2011).

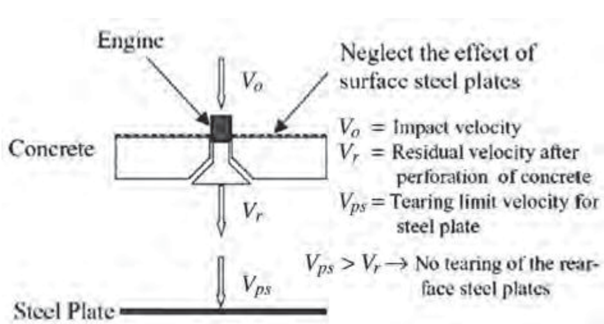


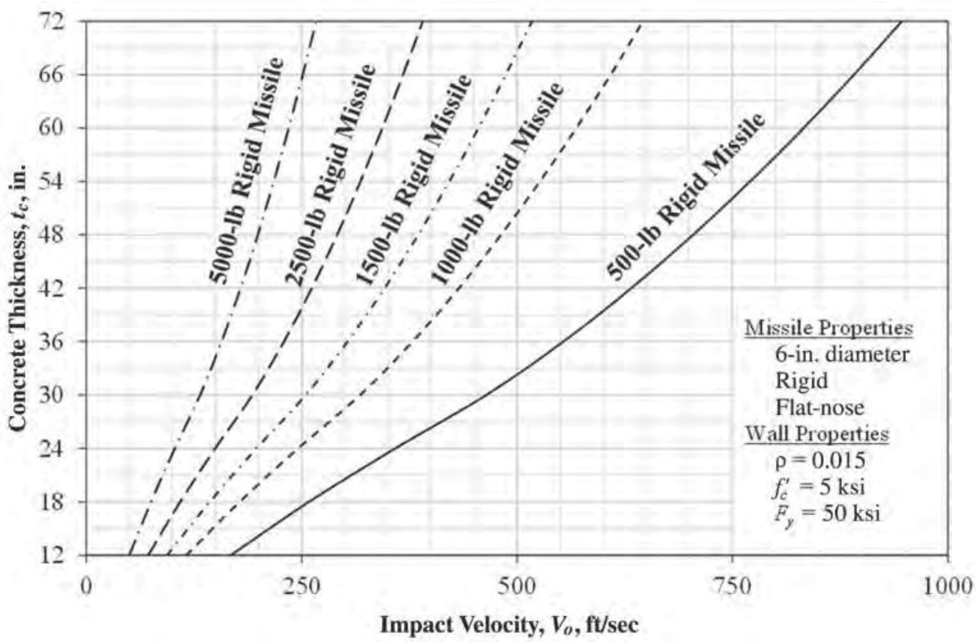
Fig. 12-1. Evaluation procedure for tearing of SC panels against impact (Mizuno et al., 2005).

Step 2. Next, the residual velocity of the missile after passing through the concrete is estimated using the formula in NEI 07-13, which is valid for rigid non-deformable missiles with initial velocity less than the perforation velocity. The ejected concrete plug is assumed to travel at the same residual velocity as the missile as the two, together, impact the rear faceplate.

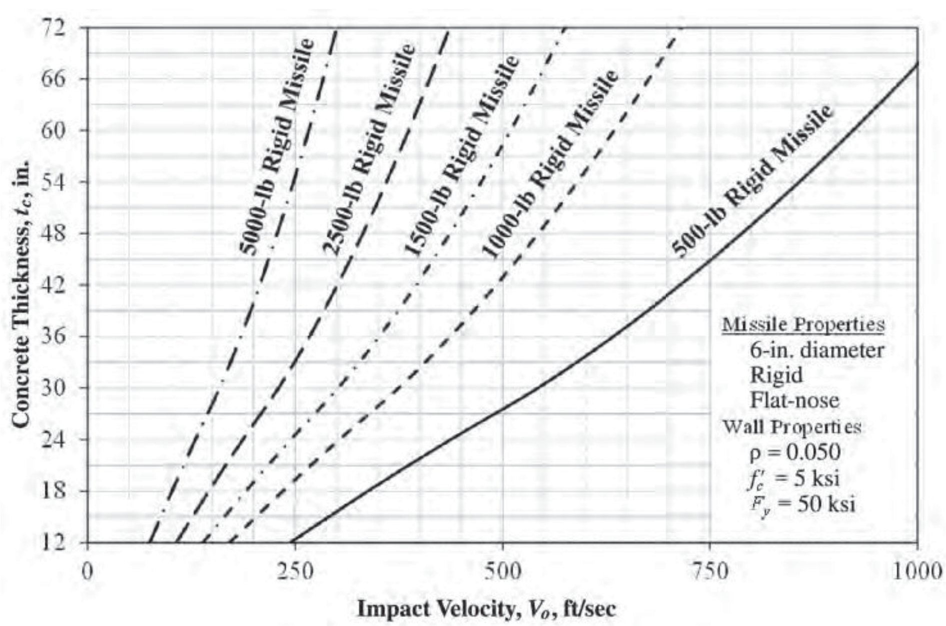
Step 3. The required faceplate thickness, t_p, can then be calculated using the formula presented by Børvik et al. (2009). The corresponding equations for this method are found in Bruhl et al. (2015a).

Using the three-step method, graphs can be generated for various missile types or specific wall configurations. Using the procedure outlined in Bruhl et al. (2015a), Figure 12-2 has been generated for a flat-nosed, 6-in.-diameter, rigid missile impacting walls of any thickness. Similarly, Figure 12-3 has been generated for the minimum practical SC wall—an interior wall of 12-in.-section thickness, t_{sc}, with 0.25-in.-thick faceplates impacted by missiles of various diameters.

For SC walls with 0.015 and 0.050 reinforcement ratios, respectively, Figures 12-2(a) and (b) provide the required concrete wall thickness for an initial missile velocity for a variety of missile weights. Figure 12-3 is used to determine the capacity of a 12-in.-thick SC wall (minimum permissible section thickness) for different missile types. If the specified missile to design against—diameter, weight and initial velocity—falls below the applicable line, the wall will prevent perforation. An increase of 25% in the faceplate thickness over the value calculated by empirical methods is necessitated by the scatter in the experimental data. This scatter, which is essentially independent of empirical equations, is accounted for by a 25% increase in faceplate thickness based on the ASCE *Structural Analysis and Design of Nuclear Plant Facilities Manual* (ASCE, 1980).



(a) 6-in.-diameter, flat-nose, rigid missile, 0.015 reinforcement ratio



(b) 6-in.-diameter, flat-nose, rigid missile, 0.050 reinforcement ratio

Fig. 12-2. Required SC wall thickness to prevent perforation.

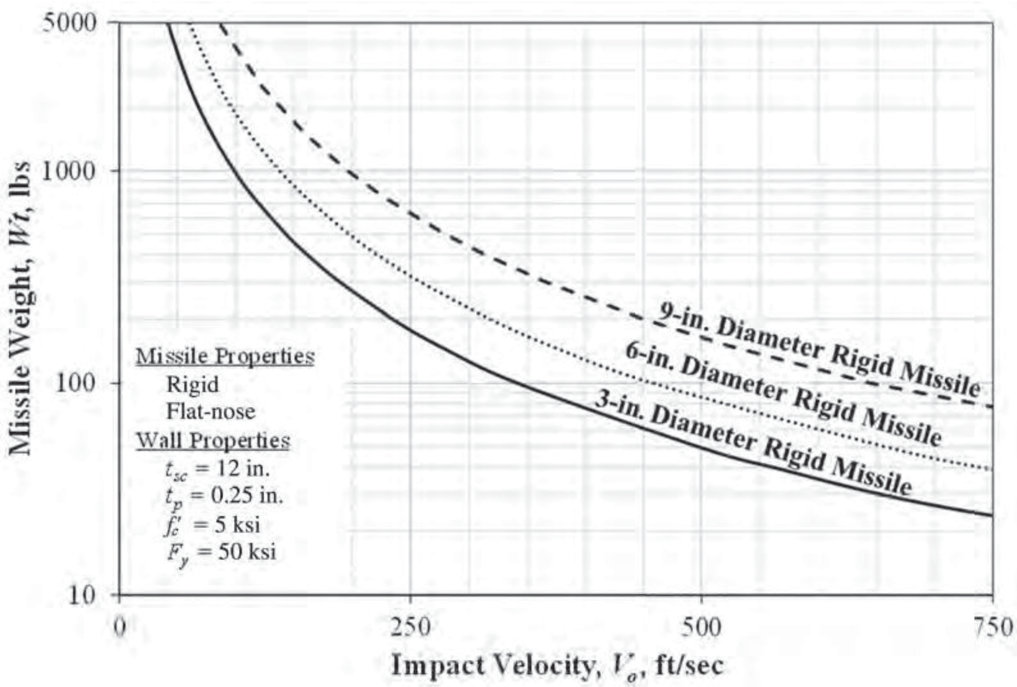


Fig. 12-3. Non-deformable (rigid) missile resistance of minimum SC wall.

Chapter 13
Fabrication, Erection and Construction Requirements

13.1 DIMENSIONAL TOLERANCES FOR FABRICATION

The dimensional tolerances discussed in ANSI/AISC N690, Chapter NM, need to be satisfied during the fabrication, erection and construction of SC panels, sub-modules and modules. Modular SC construction consists of different phases. Dimensional tolerances are applicable to:

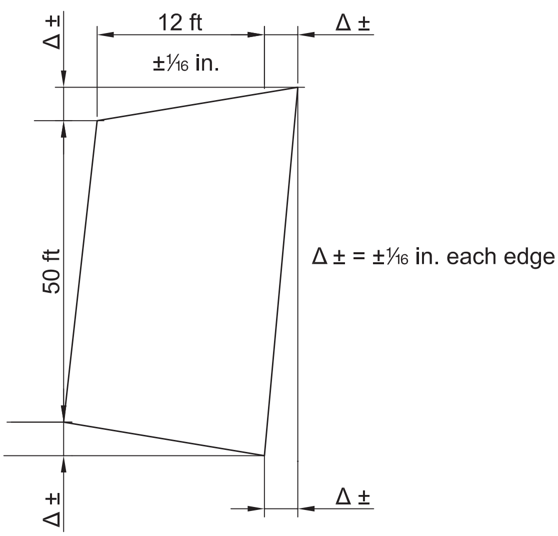


Fig. 13-1. Phase I: Fabrication of individual panels with applicable tolerances.

(a) SC wall panels and sub-modules fabricated in the shop and inspected before release.

(b) Adjacent SC walls panels, sub-modules, and modules just before connecting them.

(c) Erected SC wall modules before concrete casting.

(d) Constructed SC structures after concrete casting.

SC wall panels are typically fabricated in the shop and then shipped to the field. The overall dimensions of the fabricated SC wall panels are limited by the applicable shipping restrictions. SC wall panels that are shipped by road are limited to 8 to 12 ft (2.5 to 3.7 m) in width and 40 to 50 ft (12 to 15 m) in maximum length, as shown in Figure 13-1. Additionally, SC wall sub-modules that may consist of corner, joint or splicing modules may also be fabricated in the shop and then shipped to the field. They are subjected to the same size restrictions as the wall panels. There may be additional height restrictions based on the mode of transportation.

SC wall panels and sub-modules are connected at the site by welding or bolting to make larger modules, as shown in Figure 13-2. The size and shape of a module is driven by rigging, handling, and field erection/connection considerations. These modules are erected and connected to other modules by welding or bolting to make SC structures, as shown in Figure 13-3. The tolerances ensure that the faceplates of empty SC modules are sufficiently aligned and plumb prior to concrete placement. Concrete is then poured into assembled and erected SC modules and structures.

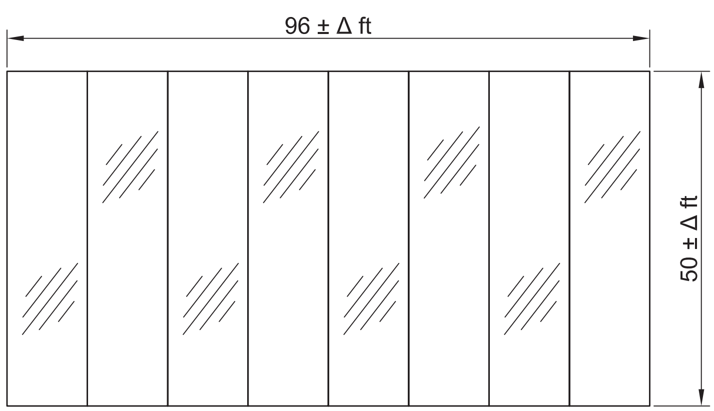


Fig. 13-2. Phase II: Combinations of panels to form a sub-module.

If the tolerances mentioned in ANSI/AISC N690, Chapter NM, are met, no additional considerations in analysis need to be made. Deviations in excess of specified tolerances are not acceptable, and need to be given due consideration by performing reconciliatory analysis or by fixing the modules to meet the tolerances. The dimensional tolerances for SC wall panels and sub-modules fabricated in the shop have to be inspected before release for shipping to the site. The dimensional tolerances are primarily for the fabricated panel thickness, t_{sc}, where the tolerance at tie locations is equal to $t_{sc}/200$ rounded up to the nearest $\frac{1}{16}$ in.(2 mm), and the tolerance between tie locations is equal to $t_{sc}/100$ rounded up to the nearest $\frac{1}{16}$ in. (2 mm).

Due to restricted access within the expanse of the fabricated panels, inspection is required only along the free edges. Because the fit-up tolerances ensure that panels or sub-modules can be combined together, measuring these tolerances at the free edges is considered sufficient. Additionally, it is understood that the maximum deviation of SC wall panels from permissible fit-up tolerances will be at the free edges. Shipping restrictions limit the maximum width to 10 ft (3 m). Project-specific inspection plans can be developed by the fabricators as needed. The dimensional tolerance on tie locations is based on the tolerance for steel headed stud anchor locations in AWS D1.1/D1.1M (AWS, 2010) or AWS D1.6/D1.6M (AWS, 2007), as applicable. This dimensional tolerance also constrains the tolerances for tie spacing and the tie angle with respect to the attached faceplates.

The fabricated panels and sub-modules are shipped to the site and then connected by welding or bolting to make larger modules. The dimensional tolerances for faceplates of adjoining panels, sub-modules or modules that are connected by welding are governed by the applicable weld tolerances from the AWS code (AWS D1.1/D1.1M for carbon steel and AWS D1.6/D1.6M for stainless steel). For welds that are qualified using project-specific qualification criteria in AWS, the dimensional tolerances should be based on that specified in the qualified weld procedure for the project. No additional squareness or skewed alignment tolerances are needed except those specified for the faceplates of adjoining panels, sub-modules or modules.

The dimensional tolerances for the erected SC modules before concrete placement are based on those for steel structures in the AISC *Code of Standard Practice* (AISC, 2016a). The dimensional tolerances for the constructed SC modules and structures after concrete placement are based on those for concrete construction in ACI 349-06 (ACI, 2006) and ACI 117 (ACI, 2010). The faceplate waviness needs to be checked following concrete placement to limit excessive faceplate displacement due to concrete placement. ANSI/AISC N690 Equation NM2.1 provides the waviness requirement. Figure 13-4 illustrates how faceplate waviness is measured. The faceplate waviness discussed refers to the total out-of-straightness of the faceplates and is not the net difference between waviness before and after concrete hardening. Corrective measures or reconciliatory analysis need to be performed in case the faceplate waviness requirement is not met.

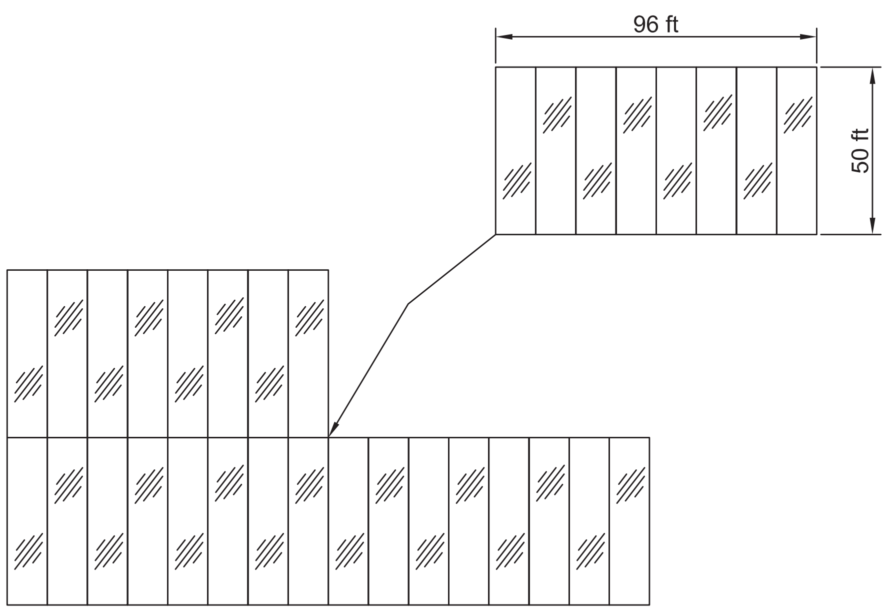


Fig. 13-3. Phase III: Erection of a module at the site prior to concreting.

Benchmarked finite element models (Zhang et al., 2014) were used to study the effect of faceplate waviness on the compressive strength of SC walls with nonslender and slender faceplates. Finite element models of nonslender SC walls with faceplate waviness up to $0.65t_p$ were analyzed. The faceplates developed more than 95% of their yield strength, $0.95A_sF_y$, at the axial compressive strength. Figure 13-5 was developed using the results of the finite element analyses. It illustrates the compression force, F_{steel}, carried by the faceplates normalized with respect to its yield strength, A_sF_y, versus the average strain over the length. For nonslender faceplates, $s/t_p = 24$, the reduction in the normalized compressive strength of the faceplates is less than 5% for an increase in imperfection from $0.1t_p$ to $0.6t_p$. However, for slender faceplates, $s/t_p = 36$, that are not permitted by ANSI/AISC N690 Appendix N9, this reduction in the normalized compressive strength is more substantial, and the post-peak behavior is degrading. Bhardwaj and Varma (2016) observed that SC walls meeting the faceplate waviness requirement and the detailing requirements of Appendix N9 that shear reinforcement is spaced at $t_{sc}/2$, do not experience significant loss in available compressive strength due to initial imperfections and concrete casting pressure when considering a typical pour height of 10 ft. However, the effects of imperfections need to be considered when the concrete pour height is larger or the ties are spaced at the section thickness.

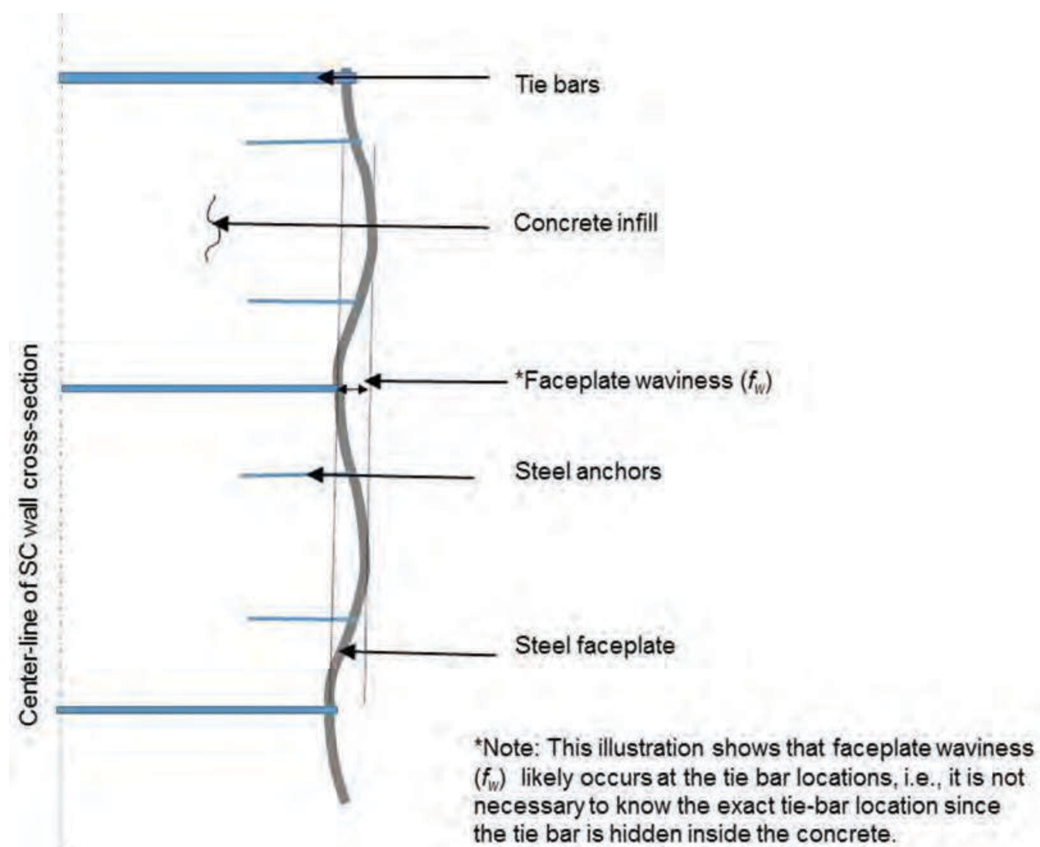


Fig. 13-4. Faceplate waviness—the faceplate waviness and the variation in tie dimensions have been exaggerated for illustration purposes.

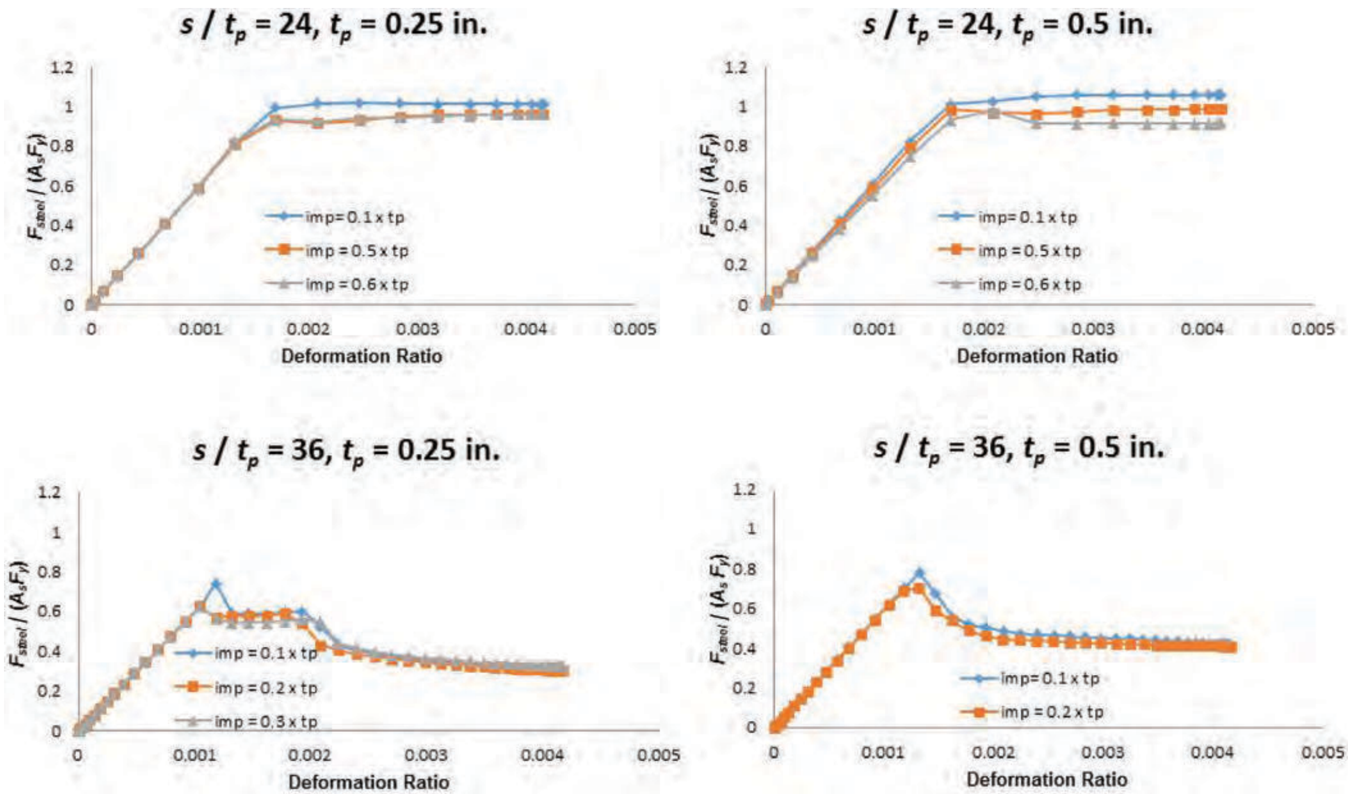


Fig. 13-5. Normalized force carried by faceplates versus average strain.