CONTAINMENT INTERNAL STRUCTURE: DESIGN CRITERIA FOR SC WALLS

MUAP-11019

Non-Proprietary Version

September 2011

REVISION HISTORY

Revision	Page	Description
0	All	Initial Issue

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TABLE OF CONTENTS

LIST OF ACRONYMS iii
LIST OF FIGURES iv
LIST OF TABLES v
ABSTRACT vi
DESIGN PHILOSOPHY AND EXECUTIVE SUMMARY vii

1.0 SC-TYPE WALLS IN US-APWR CIS 1-1
1.1 SC-Type Wall Dimensions 1-1
1.2 SC Wall Interior and Connection Regions 1-3
1.3 Report Outline 1-4

2.0 SC SPECIFIC DESIGN ISSUES AND SECTION DETAILING 2-1
2.1 Section Details 2-1
2.2 Local Buckling of Steel Faceplates 2-1
2.3 Shear Connector Strength and Spacing 2-5
2.4 Steel Faceplate Development Length 2-6
2.5 Interfacial Shear Strength 2-7
2.6 Tie Bar Spacing and Size 2-8
2.7 Tie Bar Detailing 2-9
2.8 SC Faceplate Penetration Detailing 2-10

3.0 AXIAL TENSION STRENGTH 3-1
3.1 ACI 349-06 Code Recommendation 3-1
3.2 Applicability to SC Design 3-1

4.0 AXIAL COMPRESSIVE STRENGTH 4-1
4.1 ACI 349-06 Code Recommendation 4-1
4.2 Applicability to SC Design 4-1
4.3 Additional Considerations for SC Compressive Strength 4-2

5.0 OUT-OF-PLANE FLEXURAL STRENGTH 5-1
5.1 ACI 349-06 Code Recommendations 5-1
5.2 Applicability to SC Design 5-2
5.3 Definition of SC Wall Uniaxial Moment Capacity 5-3
5.4 Experimental Verification of SC Flexural Capacity 5-5

6.0 OUT-OF-PLANE SHEAR STRENGTH 6-1
6.1 ACI 349-06 Code Recommendations 6-1
6.2 Recommendation for SC Walls 6-2
6.3 Verification Using Experimental Data 6-3
6.4 Shear Strength Contribution (V_s) of Tie Bars 6-4

7.0 IN-PLANE SHEAR STRENGTH 7-1
7.1 ACI 349-06 Code Recommendations 7-1
7.2 Experimental Data for In-Plane Shear Strength of SC Walls 7-2
7.3 Conservative Equation for In-plane Shear Strength 7-4

8.0 DESIGN FOR COMBINED FORCES 8-1
8.1 Design for Out-of-Plane Shear Demands 8-2
8.2 Design for Combined Axial Tension, Flexure, and In-Plane Shear 8-2
8.3 Design for Combined Axial Compression, Flexure, and In-Plane Shear 8-4

9.0 ACCIDENT THERMAL CONSIDERATIONS 9-1
9.1 Effects on Design Force Demands 9-1
9.2 Effects on Design Capacities 9-2
9.3 SC Specific Design Issue – Local Buckling 9-3

10.0 REFERENCES 10-1

APPENDICES

Appendix 1 - ICC Evaluation Service, Inc. Report ER-5217 Nelson Deformed Bar Anchor Studs

Appendix 2 - Sample Calculations of Reinforcement Requirements

Appendix 3 - Confirmatory Test Matrix for SC Wall Design Criteria

Appendix 4 - Design Criteria for Primary Shield Structure

Appendix 5 - Reference 8, “Out-of-Plane Shear Behavior of SC Composite Structures”

Appendix 6 - Reference 19, “In-Plane Shear Behavior of SC Composite Walls: Theory vs. Experiment”

Appendix 7 - Reference 18, “Steel-Plate Composite (SC) Walls: Analysis and Design Including Thermal Effects”

LIST OF ACRONYMS

The following list defines the acronyms used in this document.

1D	One-Dimensional
ACI	American Concrete Institute
ASTM	American Standard and Testing Materials
AWS	American Welding Society
CIS	Containment Internal Structure
DCD	Design Control Document
ICBO	International Conference of Building Officials
LEFE	Linear Elastic Finite Element
OBE	Operating Basis Earthquake
PWR	Pressurized Water Reactor
RC	Reinforced Concrete
RWSP	Refueling Water Storage Pit
SG	Steam Generator
SC	Steel-Concrete
SSI	Soil-Structure Interaction
SRSS	Square-Root-Sum-of-the-Squares
UBC	Uniform Building Code
US NRC	United States Nuclear Regulatory Commission

LIST OF FIGURES

Figure 1.1-1 Steel Plate Thickness for Primary Shield Walls....1-2
Figure 1.2-1 Interior and Connection Regions for SC Walls....1-3
Figure 2.1-1 Stud and Tie Bar Grid Pattern on Steel Faceplates....2-1
Figure 2.2-1 Local Buckling of Steel Faceplates: (a) Buckling Mode, (b) Test Observation, (c) Close-up of Test....2-2
Figure 2.2-2 ACI 318-05 Code and Commentary Excerpt for Axial Compressive Strength....2-3
Figure 2.2-3 Experimental Behavior of Specimens in Table 2.2-1....2-4
Figure 2.3-1 Strength Reduction Factors For Anchors in Concrete from ACI 349-06....2-5
Figure 2.3-2 Nominal Shear Strength of Studs....2-5
Figure 2.4-1 Free Body Diagram for Calculating Development Length....2-6
Figure 2.6-1 Maximum Spacing Requirement for Tie Bars....2-8
Figure 2.6-2 Minimum Shear Reinforcement Requirement....2-9
Figure 2.7-1 Fracture Failure of Tie Bar with Coupler....2-10
Figure 2.8-1 Typical Faceplate Penetration Detail....2-11
Figure 5.3-1 Strain diagram and stress resultants for computing moment capacity with compression reinforcement.....5-4
Figure 5.4-1: Comparison of uniaxial flexural capacity of SC sections to experimentally observed capacities....5-5
Figure 6.1-1 Concrete and Shear Reinforcement Shear Strength Equations....6-1
Figure 6.2-1 Ratio of Tests to ACI 349-06 Shear Strength Equation (Reference 12)....6-2
Figure 6.3-1 Comparisons of Shear Strength....6-4
Figure 7.1-1 In-Plane Shear Strength....7-1
Figure 7.1-2 Upper Bound Limits for In-Plane Shear Strength of RC Walls....7-1
Figure 7.2-1 Experimental results from Ozaki tests, and comparison with Equation 7.3-1....7-2
Figure 7.2-2 Experimental results from Sasaki tests, and comparison with Equation 7.3-1....7-3
Figure 8.1-1 Force and Moment Demands for Design of SC Walls....8-1
Figure 8.2-1 Section Equilibrium Used to Compute Steel Areas when N_x, N_y in Tension....8-3
Figure 8.3-1 Section Equilibrium Used to Compute Steel Areas when N_x, N_y in Compression 8-5
Figure 9.1-1 Calculated Temperature Profile in 4ft. thick SC Walls....9-1
Figure 9.1-2 Orthogonal Through-Thickness Cracking Pattern (from Reference 17)....9-2

LIST OF TABLES

Table 1.1-1 SC Walls Thickness (Including Category 1 and 2) for CIS 1-1
Table 2.2-1 Representative Test Matrix of Steel Local Buckling Tests (Reference 3)...............2-2
Table 4.3-1 Maximum SC wall unbraced lengths for ignoring slenderness. 4-3
Table 4.3-2 Unbraced lengths of SC walls in the US-APWR CIS. 4-3
Table 5.2-1 Depth to span ratios of SC walls in the US-APWR CIS. 5-2
Table 5.3-1 Comparison of SC wall moment capacities with and without compression reinforcement. 5-5
Table 8.2-1 Design for Combined Forces with N_x or N_y in Tension 8-2
Table 8.3-1 Design for Combined Forces with N_x or N_y in Compression 8-4

ABSTRACT

The purpose of this technical report is to present criteria for design of the composite steel-concrete (SC) walls in the US-APWR Containment Internal Structure (CIS). This report comprises Task 2-A in the comprehensive design and validation methodology outlined in technical report MUAP-11013 Revision 1 (Reference 10). As stated in that report, the SC wall design criteria developed herein are based primarily on the provisions of the American Concrete Institute (ACI) 349-06 code (Reference 1), because the fundamental behaviors of SC walls are similar to those of reinforced concrete (RC) walls in several aspects.

Some of the unique aspects of SC wall behavior, i.e., SC specific limit states, are identified and prevented through design/detailing. This is done so that the fundamental behaviors of US-APWR SC walls are similar to that of reinforced concrete, and the ACI 349-06 design provisions can be used conservatively for design. This includes design/detailing requirements for the headed studs that anchor the steel faceplates to the concrete and allow the plates to have development lengths similar to standard reinforcing bars. It also includes design/detailing requirements for the transverse tie bars, which provide structural integrity and function as out-of-plane shear reinforcement. The design/detailing requirements for these important SC wall components are established from pertinent SC test results and also correlated to the applicable ACI 349-06 code provisions.

This Report then identifies the ACI 349-06 code strength requirements for each individual demand type, including tension, compression, flexure, out-of-plane shear, and in-plane shear. For each of these demands, the original basis for the reinforced concrete code provisions is discussed, and the applicability of the provisions to SC behavior is presented. In cases where the RC criteria are directly applicable, such as for tension and compression, the ACI equations are directly implemented. In other cases, such as out-of-plane shear and in-plane shear, conservative adjustments to the provisions are presented and justified in terms of experimentally observed SC behavior. Finally, SC design for combined forces applies the same design principles used in reinforced concrete design to compute the total area of steel required for the combined effects of applied moments and forces.

The effects of accident thermal loading on SC wall behavior are discussed in this Report, and experimental results are used to demonstrate that accident thermal loading reduces the stiffness of the structure due to concrete cracking but does not have a significant influence on the strength (design capacity). Design for load combinations involving accident thermal loading is to be similar to design for load combinations involving operating thermal conditions.

A simple but conservative design approach based on ACI 349-06 code provisions is proposed in Appendix 4 ("Design Criteria for Primary Shield Structure") for the primary shield structure, which has a unique, multi-cellular geometry created by multiple transverse and longitudinal web plates.

In summary, the SC design criteria presented in this Report prevents SC specific limit states for US-APWR CIS walls through design/detailing, and utilizes ACI 349-06 code strength equations and reinforced concrete design principles conservatively for design. The experimental database presented in MUAP-11005 (Reference 3) supports this conservative design approach, and relevant portions of the database have been discussed in applicable sections. Appendix 3 ("Confirmatory Test Matrix for SC Wall Design Criteria") proposes several confirmatory tests to showcase the US-APWR SC design, behavior and ductility.

DESIGN PHILOSOPHY AND EXECUTIVE SUMMARY

The US-APWR containment internal structure (CIS) utilizes composite steel-concrete (SC) construction instead of conventional reinforced concrete (RC), which is motivated by issues of construction speed, economy, and structural efficiency. SC construction typically involves the use of steel faceplates acting compositely with concrete infill, while conventional RC construction involves the use of deformed steel reinforcing bars that are embedded in the concrete with adequate clear cover. The design of RC structures for safety-related nuclear facilities is governed by the ACI 349-06 code provisions. There is currently no such design code for SC structures in the US.

Experimental and analytical research of the fundamental behavior of SC walls subjected to different loading conditions (axial compression, flexure, out-of-plane shear, in-plane shear etc.) has been extensively conducted in Japan over the past 25 years. The report MUAP-11005 (Reference 3) presents some of the major research findings and accomplishments. It includes experimental and analytical results from: (i) $1/10^{th}$ scale test of a related PWR CIS subjected to cyclic lateral loading, (ii) $1/6^{th}$ scale test of the US-APWR CIS primary shield structure subjected to cyclic lateral loading, (iii) in-plane shear tests of SC walls with and without flanges, and with and without axial compression, (iv) out-of-plane shear tests of SC beams with and without tie bars, (v) axial compression tests of SC squat columns, and (vi) accident thermal load tests of SC walls panels.

The experimental investigations in MUAP-11005 (Reference 3) cover a wide range of parameters, such as steel reinforcement ratios of 1.5 to 5% ($2 \cdot t_p/T$, where t_p = plate thickness and T = section thickness), shear connector spacing to plate thickness ratios of 20 to 50, with or without tie bars, and section thickness up to 24 in.

These experimental results identify some SC specific failure modes, such as (i) local buckling of the steel faceplates, (ii) interfacial shear failure of the connectors used to anchor the steel faceplates to the concrete infill, and (iii) splitting or delamination failure of the composite section through the concrete infill. Additionally, these experimental results show that if the SC specific failure modes are prevented, then the fundamental behavior of SC walls is similar to that of RC walls with comparable reinforcement ratio. For example, under flexural loading plane sections remain plane and perpendicular to the neutral axis, and concrete cracking has little influence on moment capacity. Out-of-plane behavior is governed by shear cracking of the concrete, and the yield strength of the tie bars through which the crack passes. The in-plane behavior is governed by the yield strength of the steel plates and the orthogonal cracking of the concrete infill. These fundamental behaviors have been studied and demonstrated by researchers in the US. For example several research papers are included in Appendices 5-7.

The CIS consists of several SC walls with wall thickness ranging from 36 in. to 67 in. and reinforcement ratios ranging from 1.5 to 4.2%. More than 80% of the SC walls are 48 in. thick, with 0.5 in. thick A572 Gr.50 steel plates with reinforcement ratio of 2.1%.

The design philosophy for these SC walls is to: (i) prevent SC specific failure modes and limit states by designing and detailing the section adequately, (ii) demonstrate the conservativeness of ACI 349-06 code equations for the strength of equivalent SC walls using experimental results, and (iii) to design them using more conservative forms of ACI 349-06 code equations and combined force design approaches for RC structures.

For example, experimental results consistently indicate that steel faceplates with 50 ksi nominal yield strength and slenderness (defined by s/t_p where s = shear connector spacing and t_p = plate thickness) less than or equal to 20 do not undergo local buckling. They develop their full yield strength in compression. Additionally, even during accident thermal conditions and full axial restraint, these steel plates (with slenderness less than 20) do not undergo any local buckling for temperature increases up to 300^oC. Thus, the SC specific limit state of steel faceplate local buckling is prevented by designing or detailing plate slenderness to be less than or equal to 20 everywhere in the CIS.

The steel faceplates are anchored to the concrete using steel headed stud anchors (also referred to as shear studs) made from A108 steel with tensile strength equal to 65 ksi. The shear strength of these studs can be computed using ACI 349-06 (Appendix D) recommendations for headed studs cast-in-place concrete. The spacing and strength of these shear studs is designed to achieve a reasonable development length for the steel faceplates that is comparable to the development lengths of #11, #14, or #18 deformed rebars typically used in RC structures. This is done primarily to achieve congruence with conventional RC design and detailing practices. It further ensures that the behavior of the US-APWR SC walls will be similar to those of equivalent RC walls.

The interfacial shear strength of the SC wall steel faceplates is governed by the strength and spacing of the shear studs. Therefore, the stud spacing and strength are further designed to prevent interfacial shear failure as an SC specific limit state. This is done by designing the interfacial shear strength (of the shear studs) to be always greater than the corresponding out-of-plane shear strength of the SC section. This makes out-of-plane shear the governing failure mode for moment gradient demands, and prevents interfacial shear failure as a limit state.

Steel tie bars are provided to connect the two exterior steel faceplates through the concrete infill. These steel tie bars provide structural integrity to the SC section and prevent SC specific delamination or splitting failure mode from occurring. For the US-APWR SC walls these tie bars are made from ASTM A496 *deformed wire* (nominal yield stress = 70ksi), which is endorsed by ACI 349-06 for use as steel reinforcement in concrete structures. The area and spacing of the tie bars are designed to meet the minimum shear reinforcement requirements of ACI 349-06 Section 11.5.6.3 and tie bar reinforcement requirements of ACI 349-06 Section 7.10.5.2. These requirements result in 0.75 in. diameter deformed wire tie bars that are spaced at 24 in. in both orthogonal directions.

The steel tie bars are further detailed to develop ductile yielding in axial tension before eventual fracture failure as follows. Each full length tie bar through the SC section consists of two half-length pieces that are stud welded to the opposite steel faceplates according to AWS D1.1 requirements (Reference 5). These tie bars are manufactured by the Nelson® stud company with stud welding beads at one end, and available as off-the-shelf products. The two half-length pieces are then spliced in the center using a LENTON® Lock B-Series mechanical coupler, which is also available as an off-the-shelf product and capable of developing the full tensile strength of the tie bar.

Thus, the US-APWR SC walls are detailed to prevent SC specific failure modes such as steel plate local buckling, interfacial shear failure, and delamination or splitting failure from governing the design. The SC walls are further detailed to have development lengths that are comparable to those of equivalent RC walls, and detailed with ductile tie bar systems that are welded to the steel faceplates using stud welding, which is a reliable and well-known technology. The tie bar systems are detailed to achieve the ACI 349-06 minimum shear reinforcement and tie bar *spacing requirements. The tie bars are not included in the interfacial shear strength or development length calculations, which are conservatively based on the shear studs alone.*

The SC walls detailed as described above have fundamental behavior similar to that of equivalent RC walls, and can be designed according to applicable ACI 349-06 code provisions after demonstrating their conservativeness using SC experimental results.

The axial tension design strength (T_n), axial compression design strength (P_n), and the flexure design strength (M_n) of SC walls can be calculated using applicable ACI 349-06 code recommendations. The conservativeness of these ACI 349-06 code equations are demonstrated using available experimental results for axial compression and flexure capacity.

SC wall sections subjected to flexure are doubly reinforced with steel plates on both the compression and tension sides. The contribution of the compression steel reinforcement to the flexural design strength is not included as part of the design philosophy and for added conservatism.

The out-of-plane shear strength of SC wall sections can be calculated using applicable ACI 349-06 code equations (V_n=V_c+V_s). However, these equations do not appropriately reflect the effects of section depth (size) on the shear strength contribution of the concrete infill (V_c). Therefore, the ACI 349-06 code equation for V_c is further reduced based on experimental results to be more conservative. The shear strength contribution of the steel tie bars (V_s) can be calculated using ACI code equations because the tie bars are specifically detailed to develop their yield strength and ductility before fracture in axial tension. The conservativeness of the out-of-plane shear strength equations are demonstrated using available experimental results. Additional confirmatory tests are proposed in Appendix 3 to further demonstrate the conservatism of the out-of-plane shear strength equations.

The effects of axial tension or compression on the out-of-plane shear strength can be calculated using the corresponding ACI 349-06 code equations. Additional confirmatory tests are proposed in Appendix 3 to demonstrate the conservatism of these equations for the combination of axial tension and out-of-plane shear.

The in-plane shear strength of SC walls can be calculated using applicable ACI 349-06 code equations ($V_n = A_{cv}[\alpha_c\, f'_c{}^{0.5}+\rho_t f_y]$). As an added conservatism, the contribution of the concrete infill to the in-plane shear strength ($A_{cv}\alpha_c f'_c{}^{0.5}$) is ignored, and the in-plane shear strength is limited to the contribution of the steel plates only (V_n= $A_{cv}\ \rho_t\ f_y$ = $A_s\ f_y$). This in-plane shear strength equation compares conservatively with available experimental data for SC wall panels with reinforcement ratios from 2 – 4.5%, which is comparable to the range of reinforcement ratios for SC walls in the CIS.

The experimental results for in-plane shear strength of SC walls do not demonstrate the upper bound limit ($V_n = 10\ A_{cv}\ f'_c{}^{0.5}$) that is typically enforced for RC walls due to their overall failure caused by increasing crack widths under cyclic loading and overall crushing failure of the concrete compression strut in the wall or sliding shear failure at the base. In spite of the experimental data, this <u>upper bound limit is conservatively enforced</u> for checking the overall in-plane shear strength of SC wall lengths.

Thus, the SC walls design strengths of various demand types (axial tension, compression, flexure, out-of-plane shear, and in-plane shear) are based on the ACI 349-06 code recommendations with added conservatism where needed/appropriate. The conservatism of the

ACI code equations are demonstrated using available experimental data. Confirmatory tests are proposed to further showcase the conservatism of the out-of-plane shear strength equations with or without concurrent axial tension.

Since the US-APWR SC walls are detailed to prevent SC specific limit states, and the behavior for individual force demands and the corresponding design strengths are similar to those of RC walls, the design of SC walls for combined forces is done according to the same design philosophy and approach implemented conventionally for RC walls. As an added conservatism, the contribution of the steel reinforcement in compression is never included in the design calculations. Additionally, the contribution of the concrete infill to the in-plane shear strength is also not included in the design calculations.

The combined force demands N_x, N_y, N_{xy}, M_x, M_y, and M_{xy} from the finite element analyses of the CIS (See Figure 8.1-1 for convention) are used to compute the total area of steel required in the x and y directions (A_x^{req} and A_y^{req}) as follows:

(i) M_{xy} is directly added to M_x and M_y to increase their magnitude for design. M_x^{total} is the sum of M_x and M_{xy}, and M_y^{total} is the sum of M_y and M_{xy}.

(ii) N_{xy} is used to compute area of steel required in both the x and y directions, and they are equal. The shear reinforcement is equally distributed on both faces. The contribution of the concrete infill to the in-plane shear strength is not included.

(iii) N_x and M_x^{total} are used to compute the area of steel required in the x direction. If N_x is tensile, then the area of steel required for N_x is distributed equally on both faces. The area of steel required for M_x^{total} is calculated assuming no contribution from the steel reinforcement in compression, but the calculated area is added to both faces (tensile and compressive).

(iv) N_y and M_y^{total} are used to compute the area of steel required in the y direction. If N_y is tensile, then the area of steel required for N_y is distributed equally on both faces. The area of steel required for M_y^{total} is calculated assuming no contribution from the steel reinforcement in compression, but the calculated area is added to both faces (tensile and compressive).

The total area of steel required is computed on both faces, and in both directions (A_x^{req} and A_y^{req}), and compared with the area of steel available on both faces in both directions (A_x^{avail} and A_y^{avail}). Thus, the design for combined forces is based on a conservative interpretation of the conventional design of RC walls for combined forces.

The design for loading combinations involving accident thermal loading is similar to the design for loading combinations involving operating thermal loading conditions only. Experimental results indicate that accident thermal loading produces nonlinear (parabolic) thermal gradients and extensive cracking through the concrete cross-section. This through section cracking reduces the section stiffness, but does not have a significant influence on the out-of-plane shear strength, flexure capacity, or in-plane shear strength of the SC walls. The effects of stiffness reduction on the design force demands (N_x, N_y, etc.) are included directly in the finite element models as described in MUAP-11018 (Reference 6).

Since accident thermal loads do not have a significant influence on the SC wall design strengths, the same conservative design approach described above can be used for the design of loading combinations involving accident thermal loading conditions.

1.0 SC-TYPE WALLS IN US-APWR CIS

The US-APWR uses composite steel-concrete (SC) walls for the primary and secondary shielding walls that comprise the Containment Internal Structure (CIS). These SC walls have been categorized into three major categories: (1) SC walls less than 56 in. thick, (2) SC-type walls greater than 56 in. thick, and (3) SC-type walls that are used for the primary shield structure. These categories and their location and distribution within the US-APWR CIS are discussed in detail in MUAP-11018 (Reference 6).

1.1 SC-Type Wall Dimensions

Table 1.1-1 summarizes the wall thicknesses used for SC walls in the CIS. As shown, the secondary shield walls (section ID 104) form the majority of the CIS, and they consist of 48 in. thick SC-type walls with 0.5 in. thick A572 Gr. 50 steel plates with reinforcement ratio of 2.08%. The concrete infill for all SC walls has nominal compressive strength of 4000 psi. The south secondary shield walls (section ID 109) are also 48 in. thick, but they utilize 0.625 in. thick steel plates resulting in a reinforcement ratio of 2.60%.

The mid-height and lower pressurizer walls are 48 in. thick, but they utilize thicker steel plates of 0.75 in. and 1.0 in., respectively, resulting in much higher reinforcement ratios of 3.13% and 4.17%. The upper pressurizer wall is 36 in. thick but with 0.5 in. thick steel plates resulting in reinforcement ratio of 2.78%. The outer wall of the refueling water storage pit (RWSP) has total wall thickness 39 in. and reinforcement ratio of 2.56%.

The reactor cavity walls have different thickness (45 in., 56 in., and 67 in.) with low reinforcement ratios of 1.5%, 1.8%, and 2.2%.

Table 1.1-1 SC Walls Thickness (Including Category 1 and 2) for CIS

The wall thicknesses for the primary shield structure are summarized in Figure 1.1-1. The total section thickness varies from approximately 10 ft. to 15 ft., and includes three steel plates (two on the surface and one in the middle) with plate thickness from 0.5 in. to 2.0 in. In Figure 1.1-1, the purple portions have 0.5 in. thick steel plates, the red portions have 0.625 in. thick steel plates, the light blue portions have 0.75 in. thick steel plates, and the green portions have 1.25 in. thick steel plates. The top plate on the 10 ft. to 15 ft. thick section is 2 in. thick (shown in gold color), as are the plates that surround the main steam line penetrations.

Figure 1.1-1 Steel Plate Thickness for Primary Shield Walls

1.2 SC Wall Interior and Connection Regions

All SC walls are differentiated into *interior regions* and *connection regions* as shown in Figure 1.2-1. The *connections regions* for each SC wall are located close to edges, supports, or reaction points as shown in the Figure. Forces are transferred to and from the SC walls, and shared between the steel faceplates and the concrete infill over these *connection regions* while developing composite action.

These *connection regions* will be designed according to MUAP-11020 (Reference 16) to be no more than approximately 2.0 times the section thickness (T) in length from the ends.

The *interior regions* will have fully composite SC section behavior because the steel faceplates will develop adequate composite action with the concrete infill over the *connection regions*. These regions with fully composite SC section behavior will be designed according to this Report.

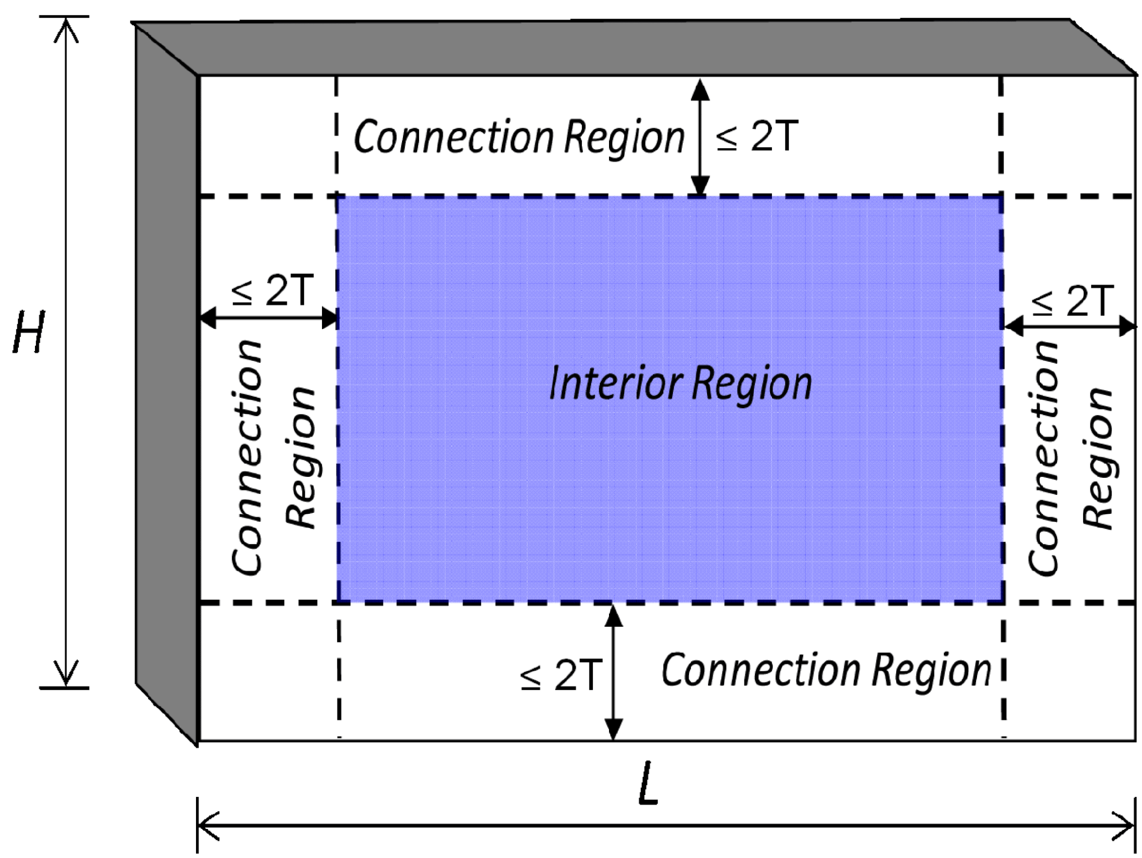


Figure 1.2-1 Interior and Connection Regions for SC Walls

1.3 Report Outline

The main body of this Report is meant for the design of SC walls (including both Category 1 and 2 SC-type walls identified in MUAP-11013 (Reference 10). Appendix 4 ("Design Criteria for Primary Shield Structure") of this Report addresses the design of Category 3 SC-type walls that form the primary shield structure. This Appendix follows the design criteria presented in this report (Sections 2 – 8) with some additional conservative assumptions.

Section 2 of this Report address SC specific design issues, for example, local buckling of the steel plates, composite action between steel plates and concrete infill, and structural integrity of the composite section.

Sections 3, 4, and 5 of this Report include design strength equations for axial tension, axial compression, and flexural moment demands. These design strength equations are based directly on the ACI 349-06 code recommendations for RC walls.

Section 6 of this Report presents design strength equations for out-of-plane shear. These equations are based on the ACI 349-06 code recommendations for RC walls, but they include further conservatism to address size and scale effects.

Section 7 of this Report presents the design strength equations for in-plane shear. These equations are also based on the ACI 349-06 code recommendations for RC walls, but they also include further conservatism based on existing SC wall test results.

Section 8 of this Report presents the approach for designing these SC walls for combined forces (axial tension or compression, flexure, and in-plane shear in either direction). It also includes the approach for designing these SC walls for out-of-plane shears in both (horizontal and vertical) directions.

Section 9 of this Report discusses SC design considerations for accident thermal loading. It is shown that accident thermal loads do not have a significant influence on the strength (axial tension, compression, flexure, out-of-plane shear, and in-plane shear strength) of SC walls. This is demonstrated using test results from Japan and the U.S.

Appendix 1 presents the International Conference of Building Officials (ICBO) report for the deformed wire A496 tie bars that will be used throughout the structure.

Appendix 2 presents sample calculations of reinforcement requirements that demonstrate the implementation and use of the design approach for combined forces outlined in Section 8.0.

Appendix 3 provides a matrix of confirmatory tests for the SC design criteria presented herein.

Appendix 4 provides the additional design criteria specific to the primary shield structure.

Appendices 5 through 7 include three important research reports that are referenced in support of the presented SC design criteria. Topics addressed include out-of-plane shear behavior of SC structures, in-plane shear behavior of SC walls, and design of SC structures for thermal effects.

2.0 SC SPECIFIC DESIGN ISSUES AND SECTION DETAILING

2.1 Section Details

The most prevalent composite SC modules are typically 48 in. thick with two 0.5 in. thick steel faceplates made from A572 Gr. 50 steel and filled with 4000 psi concrete. The steel plates are anchored to the concrete using 0.75 in. diameter A108 steel headed shear connectors (studs) with spacing of 8 in. The steel plates are connected to each other using 0.75 in. diameter A496 steel deformed wire tie bars with spacing of 24 in. The grid pattern for shear studs and tie bars is shown in Figure 2.1-1.

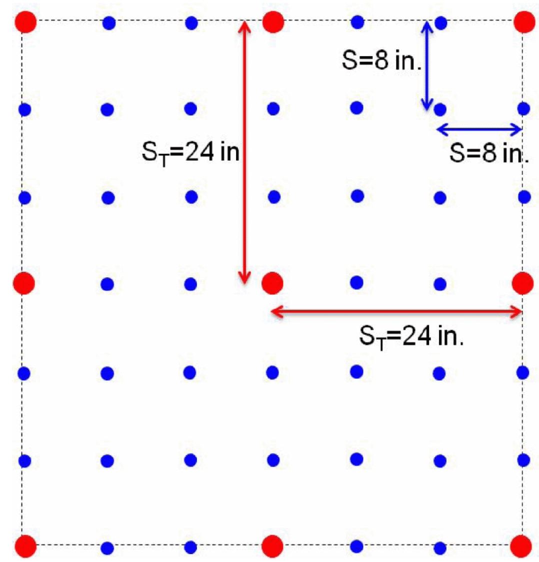


Figure 2.1-1 Stud and Tie Bar Grid Pattern on Steel Faceplates

The behavior and design of SC modules is similar to that of RC walls, as will be discussed in detail in subsequent sections. However there are several SC specific design issues involving behaviors that are different from typical RC behavior, including: (i) local buckling of steel faceplates, (ii) shear connector strength and spacing, (iii) development length, (iv) tie bar spacing, (v) tie bar ductility, and (vi) faceplate penetration detailing. The following design criteria are used to address these SC specific design issues.

2.2 Local Buckling of Steel Faceplates

The steel faceplates of SC walls are outside of the concrete infill and can potentially undergo local buckling when subjected to compressive stresses. Since the steel plates are anchored to the concrete infill using shear studs and tie bars, local buckling can only occur in between these anchor points as shown in Figure 2.2-1. The ratio s/t_p defines the slenderness ratio and controls the critical buckling stress of the steel faceplate.

Researchers in Japan have evaluated the effects of s/t_p ratio on the local buckling behavior of steel faceplates anchored to concrete. Several specimens have been tested in axial compression as reported in MUAP-11005 (Reference 3) and the behavior of a few representative and relevant specimens is discussed in more detail here to provide insight into the local buckling behavior of SC steel faceplates. These specimens are listed in Table 2.2-1.

(a) (b) (c)

Figure 2.2-1 Local Buckling of Steel Faceplates: (a) Buckling Mode, (b) Test Observation, (c) Close-up of Test

Table 2.2-1 Representative Test Matrix of Steel Local Buckling Tests (Reference 3)

These specimens had a cross-section of 1000 mm x 300 mm (39.3 in. x 11.8 in.) in plan. The specimen length was approximately equal to 1200 mm (47.2 in.), which makes them stub or short column specimens where length effects are negligible. The steel plates were made from 6 mm (0.24 in.) thick steel faceplates with measured yield stress equal to 403 MPa (58.6 ksi). The concrete infill had measured compressive strength equal to 39 MPa or 5.6 ksi.

The axial compressive strength of these stub column specimens can be estimated using ACI 349-06 (Reference 1)[1] Equation 10-2. The corresponding ACI 318-05 (Reference 2) code and commentary excerpts are included in Figure 2.2-2. As explained in the commentary the additional reduction factor of 0.80 in front of the Equation 10-2 account for accidental eccentricities that are possible in the actual structure. The test specimens were tested as

[1] Previous technical documents related to CIS design, including MUAP-11013, MUAP-11018, and the current version of the Design Control Document (DCD), refer to the ACI 349-01 code. This Report refers to ACI 349-06, in anticipation of a forth coming revision to the DCD in which ACI 349-06 (Reference 1) will become the reference code. To maintain consistency with the previous load and resistance factors, the alternate factors permitted by ACI 349-06 Appendix C will be used. It is also noted that ACI 349-06 is based upon ACI 318-05, and utilizes much of the 318-05 commentary.

concentrically as possible. Therefore, the additional 0.80 reduction factor and the phi (ϕ) factor are not included in calculating the specimen strength for comparison. The axial compressive strength of the specimens was calculated as shown below in Equation 2.2-1. In this Equation, f'_c is the compressive strength of the concrete, A_c is the area of concrete, which is equal to the total gross area (A_g) minus the steel area (A_{st}), f_y is the steel yield stress, and A_{st} is the area of the steel.

Equation 2.2-1 $$P_{no} = 0.85 f'_c A_c + A_s f_y$$

It is important to note that the US-APWR SC modules have tie reinforcement conforming to ACI 349-06 Section 7.10.5 as required here. ACI 349-06 Section 7.10.5 requires that the maximum tie bar spacing be limited to 48 times the tie bar diameter (48 in. x 0.75 in. = 36 in.) or the section depth (48 in.), whichever is smaller. The US-APWR SC modules have maximum tie reinforcement spacing of 24 in.

CODE

10.3.6 — Design axial strength ϕP_n of compression members shall not be taken greater than $\phi P_{n,max}$, computed by Eq. (10-1) or (10-2).

10.3.6.1 — For nonprestressed members with spiral reinforcement conforming to 7.10.4 or composite members conforming to 10.16:

$$\phi P_{n,max} = 0.85\phi[0.85 f_c'(A_g - A_{st}) + f_y A_{st}] \quad (10\text{-}1)$$

10.3.6.2—For nonprestressed members with tie reinforcement conforming to 7.10.5:

$$\phi P_{n,max} = 0.80\phi[0.85 f_c'(A_g - A_{st}) + f_y A_{st}] \quad (10\text{-}2)$$

10.3.6.3 — For prestressed members, design axial strength, ϕP_n, shall not be taken greater than 0.85 (for members with spiral reinforcement) or 0.80 (for members with tie reinforcement) of the design axial strength at zero eccentricity, ϕP_o.

COMMENTARY

R10.3.6 and R10.3.7 — The minimum design eccentricities included in the 1963 and 1971 codes were deleted from the 1977 code except for consideration of slenderness effects in compression members with small or zero computed end moments (see 10.12.3.2). The specified minimum eccentricities were originally intended to serve as a means of reducing the axial load design strength of a section in pure compression to account for accidental eccentricities not considered in the analysis that may exist in a compression member, and to recognize that concrete strength may be less than f_c' under sustained high loads. The primary purpose of the minimum eccentricity requirement was to limit the maximum design axial load strength of a compression member. This is now accomplished directly in 10.3.6 by limiting the design axial strength of a section in pure compression to 85 or 80 percent of the nominal strength. These percentage values approximate the axial strengths at eccentricity to depth ratios of 0.05 and 0.10, specified in the earlier codes for the spirally reinforced and tied members, respectively. The same axial load limitation applies to both cast-in-place and precast compression members. Design aids and computer programs based on the minimum eccentricity requirement of the 1963 and 1971 codes are equally applicable.

Figure 2.2-2 ACI 318-05 Code and Commentary Excerpt for Axial Compressive Strength

The experimental behavior of the tested specimens is shown in Figure 2.2-3. The Figure includes the axial force – axial strain plots for all the specimens with s/t_p ratios of 20, 25, 30, 35, and 40. The occurrence or local buckling in the steel plates is also indicated on the axial force-strain plots. The experimental results are summarized in Table 2.2-1. The Table includes the axial failure load and strain. The comparison of the experimental axial load capacity with that calculated using Equation 2.2-1 is also included. The axial strains corresponding to the local buckling of the steel plates are also included in the Table.

Figure 2.2-3 Experimental Behavior of Specimens in Table 2.2-1

The behavior shown in Figure 2.2-3 and the experimental results and comparison in Table 2.2-1 are representative of all the test results obtained in Japan (see Reference 3) for different steel plate thicknesses (4.5 mm or 0.18 in. and steel yield stresses (240 MPa or 35 ksi). The steel faceplates with slenderness (s/t_p) values less than or equal to 20 do not undergo local buckling before developing their full compressive strength estimated using ACI 349-06 Equation 10-2. Steel faceplates with slenderness (s/t_p) values greater than 20 undergo local buckling before reaching their axial strength, but still tend to develop their full axial compressive strength because the concrete stress contribution and behavior dominates.

The US-APWR design has steel plate slenderness ALWAYS less than 20, thus eliminating the failure mode of local buckling before developing full compressive strength. The US-APWR design uses a maximum tie bar spacing of 24 in. for 0.75 in. diameter tie bars, which meets the requirements of Reference 2 Section 7.10.5 as explained earlier (maximum tie spacing = 48 in. x 0.75 in. = 36 in.). The maximum axial compressive strength of the US-APWR SC modules can be estimated according to ACI Equation 10-2 shown in Figure 2.2-2.

2.2.1 Summary

The discussion presented here applies to all three categories of SC-type modules. All of the SC modules in the US-APWR have plate slenderness (s/t_p) less than 20 and tie bar spacing less than 48 times the tie diameter. This prevents the SC specific limit state of steel plate local *buckling, and provides an ACI equation (10-2) for estimating the maximum compressive strength.*

2.3 Shear Connector Strength and Spacing

The US-APWR SC modules use 0.75 in. diameter A108 steel headed stud anchors. The strengths of the steel headed stud anchors can be estimated using Reference 1, Appendix D. The ϕ factors for calculating the design strength are given in Section D.4.5 of ACI 349-06, and shown in Figure 2.3-1. The nominal shear strength of steel headed stud anchors (shear studs) governed by ductile steel yielding limit state can be computed using ACI 349-06 Section D.6.1 Equation D-18 (excerpt shown in Figure 2.3-2.) The limit states associated with concrete breakout in shear usually do not govern for these shear studs because they are in the interior regions (i.e., not located close to edges). However, the limit state associated with concrete pryout in shear for the groups of shear studs with spacing of 8 in. will be evaluated using ACI 349-06 Section D.6.3.

The design shear strength of the shear studs can be calculated by applying the shear strength reduction factor to the product of the specified tensile strength of the anchor steel (f_{uta}) equal to 65 ksi, and the effective cross-sectional area (A_{se}) equal to 0.441 in^2, as follows:

Equation 2.3-1 $$\phi V_{sa} = 0.75 \times A_{se} \times f_{uta} = 0.75 \times 0.441 \times 65 = 21.5 kips$$

D.4.5 Strength-reduction factor ϕ for anchors in concrete shall be as follows when the load combinations referenced in Appendix C are used:

(a) Anchor governed by strength of a ductile steel element
 i) Tension loads 0.80
 ii) Shear loads 0.75

(b) Anchor governed by strength of a brittle steel element
 i) Tension loads 0.70
 ii) Shear loads 0.65

Figure 2.3-1 Strength Reduction Factors For Anchors in Concrete from ACI 349-06

D.6—Design requirements for shear loading

D.6.1 *Steel strength of anchor in shear*

D.6.1.1 The nominal strength of an anchor in shear as governed by steel, V_{sa}, shall be evaluated by calculations based on the properties of the anchor material and the physical dimensions of the anchor.

D.6.1.2 The nominal strength of a single anchor or group of anchors in shear, V_{sa}, shall not exceed (a) through (c):

(a) for cast-in headed stud anchors

$$V_{sa} = nA_{se}f_{uta} \quad \text{(D-18)}$$

where n is the number of anchors in the group and f_{uta} shall not be taken greater than the smaller of $1.9f_{ya}$ and 125,000 psi.

D.6.3 *Concrete pryout strength of anchor in shear*

D.6.3.1 The nominal pryout strength, V_{cp} or V_{cpg}, shall not exceed

(a) for a single anchor

$$V_{cp} = k_{cp}N_{cb} \quad \text{(D-28)}$$

(b) for a group of anchors

$$V_{cpg} = k_{cp}N_{cbg} \quad \text{(D-29)}$$

where

k_{cp} = **1.0** for h_{ef} < 2.5 in.; and

k_{cp} = **2.0** for $h_{ef} \geq$ 2.5 in.

N_{cb} and N_{cbg} shall be determined from Eq. (D-4) and (D-5), respectively.

Figure 2.3-2 Nominal Shear Strength of Studs

2.4 Steel Faceplate Development Length

The length over which the shear studs can develop the yield strength of the steel faceplate is considered as the faceplate development length. In the US-APWR design, the spacing of the studs is 8 in. in both orthogonal directions. However, as shown in Figure 2.1-1, every third stud is replaced with an equivalent diameter (0.75 in.) tie bar. The shear strength contributions of the tie bars are not considered in the development length calculation, which is a conservative approach.

Based on the grid shown in Figure 2.1-1, it is evident that over each 48 in. x 48 in. steel panel, there are 32 shear studs and four effective tie bars. Thus, the replacement of shear studs with tie bars reduces the effective number of shear connectors from 36 to 32, which reduces the effective shear strength contribution to 32/36 = 88.9%. For the development length calculation, the strength of each shear stud is reduced to 88.9% to account for this replacement, which allows the use of the same orthogonal grid spacing of 8 in.

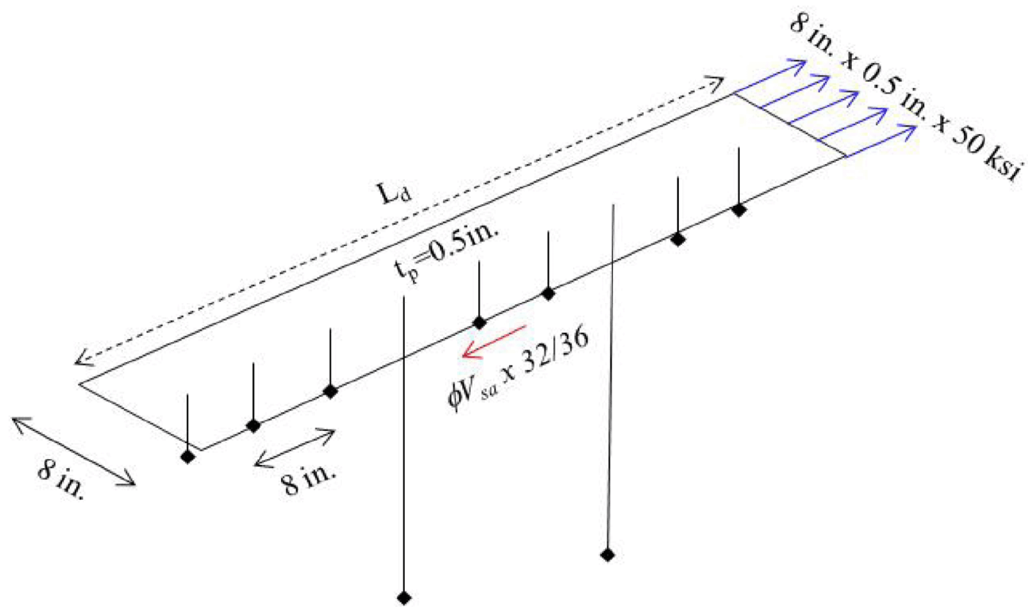


Figure 2.4-1 Free Body Diagram for Calculating Development Length

Since the transverse spacing of the studs is 8 in., the development length is calculated using the free body diagram of an 8 in. strip of the plate thickness shown in Figure 2.4-1. The yield strength of the 8 in. strip of the steel faceplate is equal to:

8 x 0.5 x 50 ksi = 200 kips

The interfacial shear strength contributed by each connector is assumed as 0.889 times ϕV_{sa}, and the total interfacial shear strength over the development length (L_d) is equal to the number of connectors (n) multiplied by the strength of each connector:

$$0.889 \times \phi V_{sa} \times n = 0.889 \times 21.5 kips \times \frac{L_d}{8in.} = 2.39 \times L_d \ kips / in.$$

The resulting development length (L_d) calculated as:

$$L_d = \frac{8in.\times 0.5in.\times 50ksi}{21.5kips \times 0.889}\times 8in. = 83.7in.$$

The development of 0.5 in. thick A572 Grade 50 plates with 8 in. x 8 in. grid of 0.75 in. diameter shear studs is approximately equal to 84 in., which is about two times the typical section thickness (48 in.). For comparison, the development length of deformed reinforcement (rebars) can be calculated in accordance with ACI 349-06, Section 12.2. As stated in the ACI commentary for Section 12.2.3, for rebars that satisfy minimum clear cover and spacing requirements, the development length for Grade 60 rebars in 4000 psi concrete can be calculated as 47 times the bar diameter.

The development lengths for #11, #14, and #18 bars are equal to 66.3 in., 79.6 in., and 106.1 in. respectively. As shown here, the development length for the 0.5 in. thick A572 Grade 50 plates is approximately equal to that of #14 bars.

2.4.1 Summary

The calculated development length for steel faceplate of SC modules is approximately equal to the typical development lengths for #14 rebars calculated using ACI code provisions, and used typically in RC nuclear structures. The calculated development length is also equal to approximately 2 times section thickness, which is an important aspect of the basic engineering approach for SC modules.

2.5 Interfacial Shear Strength

The interfacial shear strength of the shear connectors between the steel faceplates and the concrete infill can be calculated by continuing the discussion presented in Section 2.4. The interfacial shear strength of each shear stud was calculated as 0.889 times ϕV_{sa}. As explained earlier, the reduction accounts for the fact that every third shear stud in either direction is replaced with an equivalent diameter tie bar, which is assumed to contribute to only the out-of-plane shear strength, not the interfacial shear strength.

The reduced interfacial shear strength of each 0.75 in. diameter shear stud is equal to 0.889 times 21.5 kips, or 19.1 kips. The orthogonal grid spacing of the connectors is 8 in. x 8 in. Therefore, the interfacial shear strength can be expressed as an interfacial shear stress capacity of 19.1 kips/ 64 in^2, which is equal to 298 psi. For 4000 psi concrete, this is greater than four times the square root of f'_c (in psi). Although this is not meant to represent any specific equation or formula for the interfacial shear strength for SC modules, it is indicative of the relative strength of the SC sections for interfacial shear. This calculation does not include the contribution of the tie bars, which is part of the conservative design approach presented here.

The design philosophy for SC modules is to maintain the requirement that the interfacial shear strength must be greater than the out-of-plane shear strength of the SC section. As a result, an interfacial shear failure mode will not occur prior to an out-of-plane shear failure mode under applied transverse loading. Thus the SC specific limit state of interfacial shear failure is prevented from occurring.

The out-of-plane shear strength can be calculated as outlined in Section 6.0. For typical SC modules with 48 in. thickness, 4000 psi concrete, 0.5 in. thick A572 Gr. 50 plates, and 0.75 in. diameter A496 steel tie bars at 24 in. spacing, the out-of-plane shear strength can be computed as shown below using equations from Section 6.0. For example, Equation 2.5-1 shows the calculation of the out-of-plane shear strength for the typical SC modules. As shown below, the calculated out-of-plane shear strength is equal to 58.6 kip/ft., which for a 12 in. wide and 48 in. deep SC section is equal to 70.1 kips/(12 in. x 48 in^2) = 122 psi.

Equation 2.5-1 $$\phi V_n = \phi_v \left(V_c + V_s\right) = 0.85\left(12in. \times (T - 2t_p) \times 1.5 \times \sqrt{f_c'} + A_{sv} \times f_{yt} \times \frac{T}{S_T} \times \frac{12in.}{S_t}\right)$$

$$\phi V_n = \phi_v \left(V_c + V_s\right) = 0.85\left(12in. \times 47in. \times 1.5 \times \sqrt{4000psi} + 0.414in^2 \times 70ksi \times \frac{48in.}{24in.} \times \frac{12in.}{24in.}\right) = 70.1kip/ft$$

As demonstrated above, the interfacial shear strength (298 psi) is greater than the corresponding out-of-plane shear strength (122 psi) for typical SC modules. This prevents the interfacial shear failure mode for the typical SC modules, but it must be checked for all SC walls and locations.

2.5.1 Additional Summary for Sections 2.2 to Section 2.5

The interfacial shear strength of SC modules is designed to be larger than the corresponding out-of-plane shear strength, which prevents the SC specific interfacial shear failure mode.

The shear stud spacing of 8 in. x 8 in. has been designed to prevent the limit states of (i) local buckling before compression yielding of steel faceplates in Section 2.2, (ii) interfacial shear failure in Section 2.5. Additionally, the stud spacing has been designed to provide steel faceplate development length similar to that for RC structures calculated using ACI 349-06 code provisions, and less than three times the section thickness (T), which is part of our basic engineering approach.

2.6 Tie Bar Spacing and Size

Steel tie bars are needed in SC modules for the following reasons: (1) Supporting the faceplates during concrete placement. (2) Connecting the two steel faceplates through the concrete thickness, which can be larger than 3 ft. for nuclear structures. (3) Providing structural integrity by preventing delamination failure of the concrete infill. (4) Providing shear reinforcement when needed for resisting out-of-plane shear force.

11.5.5 *Spacing limits for shear reinforcement*

11.5.5.1 Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed ***d*/2** in nonprestressed members or **0.75*h*** in prestressed members, nor 24 in.

11.5.5.2 Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-degree line, extending toward the reaction from mid-depth of member ***d*/2** to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

11.5.5.3 Where V_s exceeds $4\sqrt{f_c'}b_w d$, maximum spacings given in 11.5.5.1 and 11.5.5.2 shall be reduced by 1/2.

7.10.5 *Ties*—Tie reinforcement for compression members shall conform to the following:

7.10.5.1 All nonprestressed bars shall be enclosed by lateral ties, at least No. 3 in size for longitudinal bars No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars. Deformed wire or welded wire reinforcement of equivalent area shall be permitted.

7.10.5.2 Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member.

Figure 2.6-1 Maximum Spacing Requirement for Tie Bars

The maximum spacing requirement for tie bars is based on ACI 349-06 Section 11.5.5, with excerpt shown in Figure 2.6-1. As shown the maximum spacing is equal to 24 in., which is also equal to section thickness divided by two for the typical 48 in. SC modules. The maximum tie bar spacing also meets the requirement of Section 7.10.5 in ACI 349-06 (excerpt included in Figure 2.6-1). The maximum tie bar spacing has to be less than 48 times the tie bar diameter (0.75 in.) or 36 in.

11.5.6.3 — Where shear reinforcement is required by 11.5.6.1 or for strength and where 11.6.1 allows torsion to be neglected, $A_{v,min}$ for prestressed (except as provided in 11.5.6.4) and nonprestressed members shall be computed by

$$A_{v,\,min} = 0.75\sqrt{f_c'}\frac{b_w s}{f_{yt}} \qquad (11\text{-}13)$$

but shall not be less than $(50 b_w s)/f_{yt}$.

R11.5.6.3 — Previous versions of the code have required a minimum area of transverse reinforcement that is independent of concrete strength. Tests[11.9] have indicated the need to increase the minimum area of shear reinforcement as concrete strength increases to prevent sudden shear failures when inclined cracking occurs. Equation (11-13) provides for a gradual increase in the minimum area of transverse reinforcement, while maintaining the previous minimum value.

Figure 2.6-2 Minimum Shear Reinforcement Requirement

The minimum tie bar area is based on the requirement of Section 11.5.6.3, excerpt shown in Figure 2.6-2. Assuming maximum tie bar spacing (s) of 24 in., <u>the minimum required tie bar area is equal to 0.41in^2,</u> obtained as shown in Equation 2.6-1 with b_w and s equal to 24 in., f'_c equal to 4000 psi, and f_{yt} of the rebar equal to 70,000 psi for A496 deformed wire.

Equation 2.6-1
$$A_{v,\min} = 0.75\sqrt{4000\,psi}\,\frac{24\times 24 in^2}{70{,}000\,psi}(=0.39) \geq 50\frac{24\times 24 in^2}{70{,}000\,psi} = 0.41\ in^2$$

Typical SC modules include 0.75 in. diameter (0.414 in^2 area), A496 deformed wire with yield stress equal to 70,000 psi and tensile stress equal to 80,000 psi, and orthogonal spacing grid of 24 in. x 24 in. This tie bar layout satisfies the ACI code maximum spacing and minimum shear reinforcement requirements. It also satisfies the tie spacing requirements for axial compression members, and provides connectivity between the two opposite steel faceplates.

The discussion in this section relates to the minimum tie bar requirements. Additional tie bars are provided wherever required to achieve the design requirements for out-of-plane shear strength. Additionally, the tie bar spacing in connection regions is more refined as explained later.

2.6.1 Summary

The minimum tie bar spacing and area requirements are discussed with reference to the applicable sections of the ACI code. The typical tie bar grid of 24 in. x 24 in. with 0.75 in. diameter A496 deformed wire tie bars meets the minimum requirements of the code.

2.7 Tie Bar Detailing

The tie bars described in Section 2.6 are made from ASTM A496 deformed wire, the use of which is endorsed by the ACI 349-06 code. These tie bars have to designed in accordance with ICBO Evaluation Report ER-5217 (Reference 4) (Reference also included in Appendix 1).

Some unique aspects of these tie bars are as follows:

- The deformed wire tie bar is slightly different from deformed rebar in that it is a cold drawn wire material with closely spaced divots instead of the helical protrusion in deformed rebars.
- The 0.75 in. diameter ASTM A496 deformed wire has slightly smaller area (0.414 in.^{2}) due to the divots as compared to the area or #6 rebar (0.44 in.^{2}).
- The deformed wire tie bar has minimum yield stress of 70,000 psi, and minimum tensile stress of 80,000 psi.
- The tie bars must be welded to the steel faceplates according to Chapter 7 of AWS D1.1 (Reference 5) using a stud welding gun.
- The tie bar can be stud welded to plates thicker than 0.5 times the tie bar diameter.
- The plate material must comply with the prequalified Group 1 or Group 2 base metals specified in Table 3.1 of AWS D1.1 such as ASTM A36 or A572 Gr. 50.
- The tension development length for nelson deformed bars is 33.2 in. for 0.75 in. diameter bars in 4000 psi compressive strength concrete.
- The minimum edge distance for groups of tie bars is 5 d_b (3.75 in.) for tension loads, and 15 d_b (11.25 in.) for shear loads.
- The minimum spacing between groups of tie bars is 3.75 d_b (2.8 in.) for tension loads, and 20 d_b (15 in.) for shear loads.
- The *nominal* tension capacity for deformed wire bars with the minimum development length identified above is equal to 28.7 kips for the limit state of yielding and 32.80 kips for the limit state of fracture
- The *nominal* shear strength for deformed wire tie bars with 10 d_b minimum embedment length and 10 d_b minimum free edge distance is equal to 24.5 kips for embedment in 4000 psi concrete.

These tie bars will be used in the US-APWR SC modules as follows. Two 24 in. long tie bars will be stud welded to each of the two opposite steel faceplates. These tie bars will be spliced during fabrication of the modules using a LENTON® LOCK B-Series mechanical rebar splicing system (Reference 7). This coupler is designed to develop the complete tensile strength of the connected tie bar system, and cause yielding and fracture failure in the spliced tie bar away from the coupler as shown in Figure 2.7-1. The spliced tie bar system is expected to have about the same strain ductility as a continuous tie bar.

Figure 2.7-1 Fracture Failure of Tie Bar with Coupler

2.8 SC Faceplate Penetration Detailing

The steel faceplates used in SC construction may have numerous penetrations, especially in nuclear plants with extensive piping and other utility penetration requirements. In reinforced concrete, additional reinforcing bars are typically provided in each direction at the faces of a penetration, with cross-sectional area of the additional bars equal to that of the reinforcing bars interrupted by the penetration. For penetrations through SC walls, a similar approach shall be taken in which the width of plate removed by the penetration is provided by an additional cover plate of equal thickness and width on all sides of the penetration (see Figure 2.8-1). The cover plate shall be welded to the faceplate along all edges, and the welds shall be sized and detailed to ensure adequate force transfer.

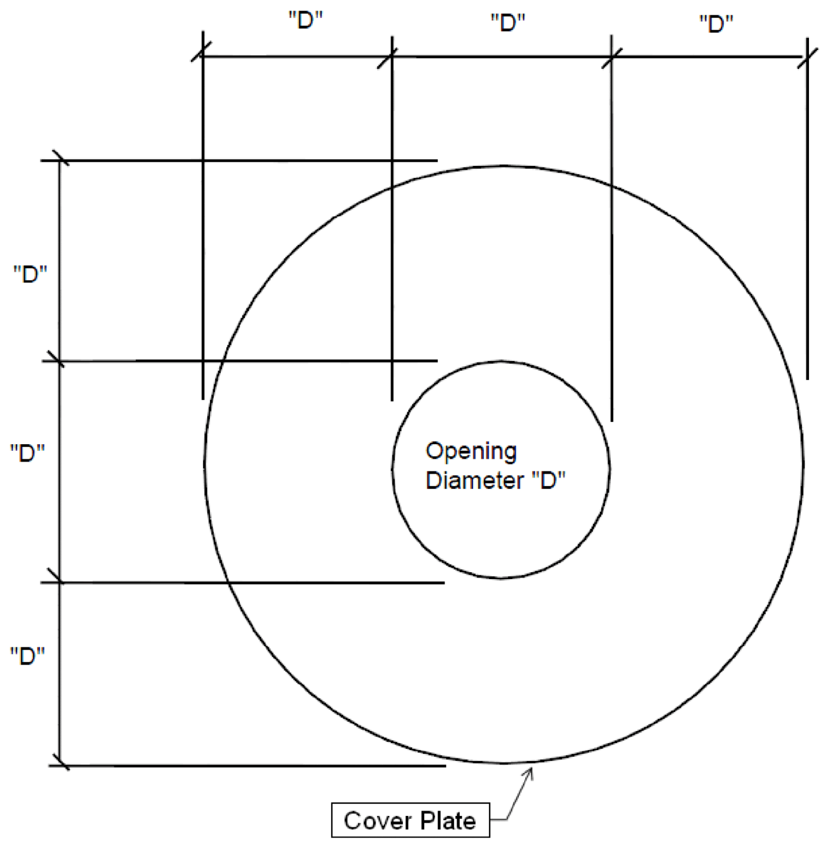


Figure 2.8-1 Typical Faceplate Penetration Detail

3.0 AXIAL TENSION STRENGTH

3.1 ACI 349-06 Code Recommendation

Section 10.2.5 of ACI 349-06 states that the tensile strength of concrete shall be neglected in axial strength calculations for reinforced concrete. As a result, the uniaxial tensile strength of reinforced concrete sections is given as follows:

Equation 3.1-1 $$\phi T_n = \phi A_{st} F_y$$

where the strength reduction (ϕ) factor is defined as 0.9 for tension in Section C3.2.1 of ACI 349-06, A_{st} is the total cross-sectional area of reinforcing steel, and F_y is the yield strength of the reinforcing steel.

There are several basic reasons for the disregard of concrete tensile strength in the ACI code: i) the tensile strength of concrete is highly variable compared to the compressive strength, ii) the tensile capacity of concrete is relatively small compared to that of the reinforcing steel, and iii) the presence of any concrete cracking eliminates the ability of the concrete section to resist direct membrane tensile forces. Consequently, neglecting the concrete capacity in membrane tensile strength calculations for reinforced concrete is conservative under limited loading conditions.

3.2 Applicability to SC Design

Since the aforementioned considerations behind ACI 349-06 Section 10.2.5 are also applicable to SC wall behavior, Equation 3.1-1 is appropriate for use in SC design. With regard to concrete tensile strength, the concrete cores of the US-APWR SC wall sections are to be constructed using standard concrete mixes with specified compressive strength (f'_c) equal to 4000 psi. As such the assumption of a relatively small concrete tensile capacity prior to cracking remains applicable to the US-APWR SC walls.

Neglecting concrete tensile capacity is also appropriate for SC sections since they are known to experience a higher degree of cracking due to curing shrinkage than is typically observed in RC sections. This is due to locked-in tensile stresses in the SC concrete cores that result from restraint of curing shrinkage by the steel faceplates, and also the discrete nature of the bond between the reinforcing steel and the concrete core. As described in MUAP-11018 (Reference 6) and illustrated in the tests performed by Ozaki et al (Reference 8), these characteristics result in reduction of cracking tensile stress for SC sections from the $4\sqrt{f'_c}$ (psi) value used in RC design to approximately $2\sqrt{f'_c}$ (psi) in SC sections.

In terms of reinforcement bond, the faceplates used in SC construction are attached to the concrete core only at the anchorage stud locations, unlike standard reinforcing bars which are continuously bonded to the concrete by reinforcement bar deformations. As a result, the cracks developed in the concrete cores of SC sections are generally wider and at larger intervals relative to those in RC sections with similar reinforcement ratios. Thus the neglect of concrete tensile strength due to cracking is appropriate for SC design.

4.0 AXIAL COMPRESSIVE STRENGTH

4.1 ACI 349-06 Code Recommendation

Section 14.4 of ACI 349-06 (Reference 1) specifies that reinforced concrete walls designed as compression members shall be designed in accordance with the provisions of Chapter 10. Section 10.3.6 of Reference 1 specifies that the design axial load strength of reinforced concrete compression members with standard tie reinforcement shall not be taken greater than the following (ACI Equation 10-2):

Equation 4.1-1 $$\phi P_n = 0.8 \cdot \phi \left[0.85 f'_c \,(A_g - A_{st}) + F_y A_{st} \right]$$

where the strength reduction (ϕ) factor is defined as 0.7 for compression members without spiral reinforcement in Section C.3.2.2 of Reference 1, A_g is the gross area of the section, and A_{st} is the total area of reinforcement provided in the direction of the applied force. Additionally, the strength reduction factor is permitted to be increased to a maximum of 0.9 for low values of compression, in the manner discussed in Section C.3.2.2.

The aggregate strength reduction factor in Equation 4.1-1 is equal to 0.8 x 0.7 = 0.56. The conservatism provided by this factor accounts for a number of concerns specific to the safety of reinforced concrete compression members in frame structures. The first 0.8 factor accounts for accidental eccentricities typically encountered in columns that use standard tie reinforcement (a 0.85 factor is specified for spiral columns). The centroid of the steel and concrete resistance forces in the as-built column is often offset slightly from the geometric centroid of the column, resulting in an additional moment that reduces the maximum axial force the column can carry. The lower factor (0.7 for tied columns) recognizes that material nonuniformity in the concrete has a larger impact on axial compressive strength than it does on flexural strength, and that concrete strength may be less than f'_c under sustained high axial loads (Reference 1). The 0.7 resistance factor also addresses the lower ductility of compression failures, and the more serious consequences of compression member failure in the frame structures for which these provisions were primarily written.

The 0.85 factor applied to the concrete compressive resistance in Equation 4.1-1 is based on the results of numerous tests on axially loaded reinforced concrete members. This factor addresses the experimentally observed effects of less than ideal concrete consolidation and curing in actual compression members as compared to the conditions provided for compressive strength test cylinders.

4.2 Applicability to SC Design

It is apparent that the basis for the low strength reduction factor applied to RC compression members is partially applicable to SC behavior, but the compression performance of SC walls is arguably better. For example, the effects of concrete material nonuniformity and imperfect field consolidation/curing on member compressive strength may also be present in SC wall construction, but the lack of reinforcement congestion will certainly reduce concrete placement issues. As demonstrated by the SC compression member tests summarized in Table 2.2-1, the 0.85 factor applied to concrete compressive resistance is acceptably conservative for SC construction. Similarly, the reduction of compression forces in the concrete due to creep under sustained loading and the subsequent transfer of forces to the steel will also occur in SC sections, but to a lesser degree given the discontinuous bond of the steel to the concrete. The issues related to accidental eccentricity due to construction practices for tied columns will not be as significant in SC faceplate construction, although accidental moments may still be present from other sources. In terms of the failure consequences considered in the ACI strength reduction factor, compression failures of SC walls would certainly be of serious consequence to structural integrity, although the large SC cross sections selected for radiation shielding purposes typically will not be challenged in terms of compressive strength. The comparison of SC wall behavior to RC column behavior indicates that the aggregate compressive strength reduction factor given in ACI 349-06 Equation 10-2 is appropriately conservative for SC wall design.

With regard to compression failure modes, Equation 4.1-1 (ACI Equation 10-2) also assumes that the steel reinforcement will yield before buckling. This assumption is based upon the detailing requirements given for columns, which include specific tie spacing and bar engagement (hook) requirements to ensure the longitudinal reinforcement is sufficiently braced. In SC construction, similar detailing requirements are established to ensure the steel faceplates do not buckle before yielding. As discussed in Section 2.2, compression tests on SC wall sections have been performed to verify that the size and spacing of the anchorage studs and tie bars in the US-APWR design will prevent the occurrence of local faceplate buckling and ensure the compressive yield capacity of the plates can be achieved.

4.3 Additional Considerations for SC Compressive Strength

The wall design provisions of Reference 1 Section 14.4 also require the slenderness provisions for compression members in Section 10.10 to be addressed. Within the non-sway frame provisions applicable to a shear wall structure such as the CIS, Section 10.12.2 permits slenderness effects to be ignored when the following equation (ACI 349-06 Equation 10-7) is satisfied:

Equation 4.3-1 $$\frac{k \cdot l_u}{r} \leq 34 - 12(M_1 / M_2)$$

where k is the effective length factor, l_u is the unsupported length, r is the radius of gyration, M_1 is the smaller factored end moment (positive for single curvature and negative for double curvature), and M_2 is the larger factored end moment (always positive). Since M_1 is the smaller end moment and M_2 is always positive, the worst case value for the quantity (M_1/M_2) is 1.0. For the end restraint conditions encountered by most of the SC walls in the CIS, an effective length factor of k = 0.7 (fixed-pinned) may be used. This factor is reasonable and conservative for all of the walls below the operating deck (Elevation 76’-5”), as they are bounded by the basemat and the major elevated floor slabs. Using radius of gyration r = 0.3T as permitted by ACI 349-06 Section 10.11.2 (T = overall wall thickness), maximum unbraced lengths for ignoring slenderness may be computed as follows:

$$\frac{k \cdot l_u}{r} \leq 34 - 12(M_1 / M_2);$$

$$M_1 / M_2 = 1.0$$

$$\frac{k \cdot l_u}{r} \leq 22;$$

$$l_u \leq \frac{22 \cdot r}{k}$$

Maximum unbraced lengths are computed in the manner above for the various SC wall thicknesses as follows:

Table 4.3-1 Maximum SC wall unbraced lengths for ignoring slenderness.

The unbraced lengths for the SC walls between the basemat and the operating deck at elevation 76'-5" are presented in the table below:

Table 4.3-2 Unbraced lengths of SC walls in the US-APWR CIS.

The tables above show that slenderness is not an issue for the walls below the operating deck. For the remainder of the walls in the CIS, which includes the Steam Generator (SG) and pressurizer compartment walls above the operating deck, the walls are connected to one another in relatively narrow cylindrical or box configurations, such that each wall is fully braced along its height by the adjacent walls. It is therefore concluded that all SC walls in the CIS may be considered as non-slender. As a result, slenderness effects will be ignored in the calculation of SC wall demands and capacities.

5.0 OUT-OF-PLANE FLEXURAL STRENGTH

5.1 ACI 349-06 Code Recommendations

Section 10.2 of ACI 349-06 specifies a series of design assumptions that form the basis for calculating flexural capacity of reinforced concrete sections. These assumptions are as follows:

1. Section 10.2.2: *A linear strain distribution is assumed*. The strains in the reinforcement and concrete are assumed directly proportional to the distance from the neutral axis. The code states this assumption is not applicable to deep flexural members with clear spans less than or equal to four times the overall member depth.
2. Section 10.2.3: *A maximum usable strain at the extreme concrete compression fiber is assumed equal to 0.003.*
3. Section 10.2.4: *Stress in reinforcement below the specified yield strength f_y is taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement is taken equal to f_y.*
4. Section 10.2.5: *Tensile strength of concrete is neglected in flexural calculations.*
5. Section 10.2.6: *The relationship between concrete compressive stress distribution and concrete strain is permitted to be assumed as rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.*
6. Section 10.2.7: *An equivalent rectangular concrete stress distribution meets the requirements of Section 10.2.6, when the following parameters are used:*
   a. Uniform concrete stress equal to $0.85f'_c$.
   b. Depth of equivalent rectangular stress block equal to $\beta_1 c$, where c is the depth from the extreme compression fiber to the neutral axis, and β_1 is equal to 0.85 for $f'_c \leq 4000$ psi.

In addition to specifying that design of an RC cross section for flexure shall be in accordance with these assumptions, Section 10.3 of Reference 1 requires verification of the ductility of flexural members with the reinforcement provided. Specifically, Section 10.3.5 requires that flexural members with low axial compressive loads have net tensile strains in the extreme tension steel layer (ε_t) not less than 0.004. The commentary for this section provided in Reference 1 states that this limit restricts the reinforcement ratio (ρ) to about the same ratio as that defined in previous code editions, or 0.75 times the balanced reinforcement ratio (ρ_b). The balanced reinforcement ratio is defined as that which would produce balanced strain conditions for the section under flexure without axial load. In other words, the balanced reinforcement ratio results in a failure mode in which the compression concrete crushes simultaneously with first yielding of the tension reinforcement. Limiting the reinforcement ratio to $0.75\rho_b$ ensures that the strain in the tension steel will substantially exceed the yield strain when the ultimate concrete compressive strain (0.003) is reached, with accompanying ductile behavior consisting of large deflections and ample warning of an impending failure.

The use of compression reinforcement is known to improve the ductility of flexural members by strengthening the compression zone of the member and reducing the depth of the concrete compressive stress block (Reference 9 - "Reinforced Concrete Mechanics and Design"). ACI 349-06 recognizes this in Section 10.3.5.1, which states that compression reinforcement is permitted to be used in conjunction with additional tension reinforcement to increase the strength of flexural members.

5.2 Applicability to SC Design

5.2.1 Applicability of Design Assumptions

With regard to the basic assumption of a linear strain distribution given in Section 10.2.2., the clear span to depth ratios of the SC walls in the CIS are evaluated using the wall thicknesses and clear span calculations given in Section 4.3. In accordance with ACI 349-06 Section 10.2.2 and the deep beam definition given in Section 10.7.1, the majority of the SC walls in the CIS are calculated to have clear span-to-depth ratios larger than 4, as follows:

Table 5.2-1 Depth to span ratios of SC walls in the US-APWR CIS.

It is seen that all walls except the lower span of the 67 in. thick walls meet the stated criterion for assuming a linear strain distribution. There is only one 67 in. thick wall in the first vertical span of the CIS, below the refueling canal. This wall is supported by massive reinforced concrete (i.e. Category 5 in Reference 10) over much of its height, such that it is not expected to experience significant flexure. Thus, the assumption of a linear strain distribution is deemed appropriate for design of the US-APWR SC walls for flexure.

It is also recognized, however, that the assumption of perfect compatibility of steel and concrete strains stated in Section 10.2.2 is not fully achieved in SC members, due to imperfect bond of the steel plates to the concrete described in Section 3.2. Nevertheless, the experimental data discussed below illustrates that this assumption remains valid for SC design. It is also important to note that the development of the steel faceplate reinforcement occurs over a finite length of the wall from each connected edge, such that the assumption of steel yielding given in ACI Section 10.2.4 is only appropriate for sections of the wall beyond this development length. Thus the uniaxial flexural capacity for SC design defined herein is only applicable to the interior regions of a given wall. As discussed in Section 2.4, the boundaries of the interior region may be considered as occurring at approximately two times the section thickness from the connected edges of a given wall.

As stated in ACI Section 10.2.6, the applicability of the rectangular compressive stress block assumptions given by ACI 349-06 Section 10.2.7 must be evaluated by comparing the calculated capacities to the results of tests. Comparisons of calculated uniaxial flexural capacities to the results of flexural tests on SC wall sections are discussed below in Section 5.4.

5.2.2 Applicability of Ductility Requirements

The flexural reinforcement of the SC sections in the US-APWR CIS always consists of steel plates of equal thickness and yield strength (50 ksi) on each face. This gives a compression reinforcement ratio (ρ') that is identical to the tension reinforcement ratio (ρ). As a result, none of the flexural tension force in the tension reinforcement is balanced by compression in the concrete, which means that the reinforcement limitation inherent in ACI 349-06 Section 10.3.5 does not apply to any portion of the tension reinforcement area. Because of the balance of tension and compression reinforcement in SC walls, the limiting concrete compression strain (0.003) cannot be reached before the tension reinforcement has yielded. This behavior comprises an essential benefit of SC construction; it is not possible for a properly detailed SC wall subjected to pure flexure to experience a nonductile (brittle) failure mode.

5.3 Definition of SC Wall Uniaxial Moment Capacity

Using the stress and strain assumptions stated in ACI 349-06 Section 10.2, equations for uniaxial moment capacity are readily developed for reinforced concrete members with various reinforcement configurations. For the SC sections in the US-APWR CIS, a conservative evaluation of uniaxial moment capacity may be performed in which only the contribution of the tension face reinforcement is considered. The theoretical uniaxial moment capacity may also be calculated in a manner similar to that used for reinforced concrete members with equal areas of tension and compression reinforcement. The following sections evaluate the results of these approaches using the actual US-APWR SC section properties, in order to determine the most appropriate methodology for design.

5.3.1 Uniaxial Moment Capacity—Tension Steel Only

If the flexural contribution of compression reinforcement is neglected, the uniaxial moment capacity for SC walls is calculated as follows:

Equation 5.3-1
$$\phi M_n = \phi\left[\frac{A_{st}}{2}F_y \cdot \left(T - \frac{3t_p}{2} - \frac{a}{2}\right)\right]$$

where $A_{st}/2$ is the area of the tension reinforcement (half of the total reinforcement area) per unit length of wall, $(T-3t_p/2)$ is the depth from the top of the concrete section to the centroid of the tension reinforcement (analogous to 'd' in reinforced concrete design), and a is the depth of the equivalent rectangular compressive stress block. In accordance with ACI 349-06 Section 10.2.7, this is calculated as 0.85 times the depth to the neutral axis (c). The strength reduction factor (ϕ) for flexure in tension-controlled sections is given in Reference 1 Section C.3.2.1 as 0.9.

5.3.2 Uniaxial Moment Capacity—Including Compression Reinforcement

The uniaxial moment capacity for SC sections may be computed to include the compression reinforcement in the following manner:

- Determine the neutral axis of the composite section based upon yielding of the tension faceplate and a linear strain diagram.
- Sum the moments caused by the forces in the steel and concrete acting about the centroid of the section.

This is illustrated graphically in Figure 5.3-1 below. The resulting uniaxial moment capacity is calculated as follows:

Equation 5.3-2 $$\phi M_n = \phi\left[\left(T_s + C_s\right)d_p + C_c \cdot d_c\right]$$

where Ts and Cs are the forces in the tension and compression faceplates, respectively, d_p is the moment arm between either plate and the centroid of the section, Cc is the force resultant of the concrete in compression, and d_c is the moment arm from the centroid of the section to concrete compressive force resultant. The magnitudes of each of these variables are shown in Figure 5.3-1 in terms of the defined section properties.

Figure 5.3-1 Strain diagram and stress resultants for computing moment capacity with compression reinforcement.

5.3.3 Comparison of Results

Using the formulations stated above, moment capacities are calculated for each of the various US-APWR SC sections with and without the contribution of compression reinforcement. The results of these calculations are presented in Table 5.3-1 below. It is seen that neglecting the compression reinforcement results in only a small reduction of uniaxial flexural capacity. As a result, the uniaxial moment capacity for the SC wall sections will be computed neglecting the compression reinforcement, and the area of reinforcement provided on the tension face shall also be provided on the compression face. This ensures sufficient capacity of the section for load reversals occurring as a result of seismic loading, and enhances the ductility of the section as discussed above.

Table 5.3-1 Comparison of SC wall moment capacities with and without compression reinforcement.

5.4 Experimental Verification of SC Flexural Capacity

The out-of-plane flexural capacity of SC walls has been determined experimentally in tests performed in Japan (Reference 11 – "Experimental Study on a Concrete Filled Steel Structure, Part 4 Shear Tests"). Figure 5.4-1 below compares the experimentally observed flexural capacities to those obtained using Equation 5.3-1 above (i.e. disregarding compression reinforcement.) It is shown that the calculated values are slightly conservative relative to the experimentally observed values.

Figure 5.4-1: Comparison of uniaxial flexural capacity of SC sections to experimentally observed capacities .

6.0 OUT-OF-PLANE SHEAR STRENGTH

The behavior and design of SC modules for out-of-plane shear force is similar to that of RC beams. The design equations developed for RC can be used for SC design with some modifications.

6.1 ACI 349-06 Code Recommendations

According to ACI 349-06 Section 11.1.1, the out-of-plane shear strength of RC cross-section can be estimated as the summation of the shear strength contributions of the concrete (V_c) and the shear reinforcement (V_s). The concrete contribution (V_c) depends on direction of axial load (tension or compression) as shown in the ACI code excerpts included in Figure 6.1-1.

11.1.1 Except for members designed in accordance with Appendix A, design of cross sections subject to shear shall be based on

$$\phi V_n \geq V_u \qquad (11\text{-}1)$$

where V_u is the factored shear force at the section considered and V_n is nominal shear strength computed by

$$V_n = V_c + V_s \qquad (11\text{-}2)$$

where V_c is nominal shear strength provided by concrete calculated in accordance with 11.3, 11.4, or 11.12, and V_s is nominal shear strength provided by shear reinforcement calculated in accordance with 11.5, 11.10.9, or 11.12.

11.3.1 V_c shall be computed by provisions of 11.3.1.1 through 11.3.1.3, unless a more detailed calculation is made in accordance with 11.3.2.

11.3.1.1 For members subject to shear and flexure only,

$$V_c = 2\sqrt{f'_c}\, b_w d \qquad (11\text{-}3)$$

11.3.1.2 For members subject to axial compression,

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\sqrt{f'_c}\, b_w d \qquad (11\text{-}4)$$

Quantity N_u/A_g shall be expressed in psi.

11.3.1.3 For members subject to significant axial tension, V_c shall be taken as zero unless a more detailed analysis is made using 11.3.2.3.

11.3.2.3 For members subject to significant axial tension,

$$V_c = 2\left(1 + \frac{N_u}{500A_g}\right)\sqrt{f_c'}\, b_w d \qquad (11\text{-}8)$$

but not less than zero, where N_u is negative for tension. N_u/A_g shall be expressed in psi.

11.5.7.2 Where shear reinforcement perpendicular to axis of member is used,

$$V_s = \frac{A_v f_{yt} d}{s} \qquad (11\text{-}15)$$

where A_v is the area of shear reinforcement within spacing s.

Figure 6.1-1 Concrete and Shear Reinforcement Shear Strength Equations

In these equations, f'c is the compressive strength of the concrete, bw is the web width, d is the distance from extreme compression fiber to centroid of longitudinal tension reinforcement, Nu is the factored axial force normal to cross section taken positive for compression, Ag is the gross area of concrete section, fy is the specified yield strength of transverse reinforcement.

6.2 Recommendation for SC Walls

Experimental results for RC beams (without shear reinforcement) indicate that the ACI 349-06 code equation 11-3 overestimates the concrete shear strength contribution (V_c) for members subjected to shear and flexure only (Reference 12). For example, Figure 6.2-1 taken from Reference 12 illustrates the decreasing trend for concrete shear strength (V_c) as the specimen depth increases. It is also pointed out that the shear strength for large scale beams are about half of is the value given for V_c in the ACI code.

Figure 6.2-1 Ratio of Tests to ACI 349-06 Shear Strength Equation (Reference 12)

This trend has also been observed for SC beam cross-sections by Varma et al (Reference 13) who indicated that the size of the specimen has a significant influence on the concrete shear strength contribution (V_c). The experimental results indicated that as the specimen depth got larger, the shear stress carried by concrete portion (V_c) reduced. Therefore, lower values for the concrete shear strength (V_c) will be conservatively used for design of SC sections as shown below:

Equation 6.2-1 $$V_c = 1.5\sqrt{f'_c}A_c$$ (for shear and flexure only)

Equation 6.2-2 $$V_c = 1.5(1+\frac{N_u}{2000A_g})\sqrt{f'_c}A_c$$ (members subjected to axial compression)

Equation 6.2-3 $$V_c = 1.5(1+\frac{N_u}{500A_g})\sqrt{f'_c}A_c$$ (members subjected to axial tension)

The contribution of the shear reinforcement (V_s) to the total shear strength can be evaluated based on the equation 11-15 given in ACI 349-06 as,

Equation 6.2-4
$$V_s = \frac{A_v f_y d}{s}$$

Where A_v is the area of transverse reinforcement within spacing *s, f_y* is the specified yield strength of reinforcement and *d* is the total section depth.

6.3 Verification Using Experimental Data

Figure 6.2-1 demonstrates the conservatism of Equation 6.2-1 when used with the ACI 349-06 strength reduction factor for shear (0.85). As seen the equation is a lower bound when compared with the test database that includes several reinforced concrete beams without shear reinforcement tested in the past.

The proposed equations for shear strength have been compared with tests performed by Takeuchi et al (Reference 14) and Varma et al (Reference 13) in Figure 6.3-1. The beams Takeuchi et al tested, S4 and S6, had 3.6% longitudinal reinforcement ratio, stud spacing-to-plate thickness ratio equal to 27.8, shear span to depth ratio equal to 2.6 and section depth of 19.7 inches. The specimen had shear reinforcement in the form of 50 ksi yield strength tie bars, and their contribution to the shear strength (V_s) was calculated using Equation 6.2-4. The specimen depths were less than half of the values used typically for SC walls in US-APWR., which is expected to have significant influence on the concrete shear strength contribution (V_c) as explained earlier.

The beam tested by Varma et al (Reference 13) was large-scale in terms of the current application sizes, having; 2.8% longitudinal reinforcement ratio, stud spacing to plate thickness ratio equal to 20, shear span to depth ratio equal to 3.5 and specimen depth equal to 36 inches. This beam did not have any shear reinforcement, and the Vs contribution is taken equal to zero. Figure 6.3-1 shows the comparisons of the shear strength values measured experimentally and those calculated using the proposed equations. As seen in the figure the equations underestimate the shear strength for the Takeuchi beams due to their shallow depth (19.7 in.), but provide a conservative lower bound shear strength in general.

Figure 6.3-1 Comparisons of Shear Strength

6.4 Shear Strength Contribution (V_s) of Tie Bars

As mentioned in ACI 349-06 Section 11.3.1.3 (excerpt shown in Figure 6.1-1), in the presence of significant axial tension (greater than 500 psi on the gross section area or 288 kip/ft. on a 4 ft. thick section) the out-of-plane shear strength can be calculated by considering the contribution of the shear reinforcement (V_s) alone and neglecting the contribution (V_c) of the concrete.

This approach may be needed when evaluating the design for seismic force demands calculated using the square-root-sum-of-the-squares (SRSS) combination method, which eliminates the sign of all forces including the axial force. It is necessary to consider both directions (axial tension and compression) if the sign of the axial force has been lost due to the SRSS combination method. The design case for axial tension + out of plane shear will govern because it reduces the shear strength to that of the shear reinforcement (tie bars) alone.

According to Section 11.5.6.2 of ACI 349-06 (excerpt shown in Figure 6.1-1), the contribution of the tie bars (Vs) to the shear strength of a unit foot wide section having 4 ft. depth, 0.75 in. diameter 70 ksi steel reinforcement at 24 in spacing can be computed using Reference 1 (Eq. 11-15 in ACI 349) as:

$$V_s = A_v f_y \frac{T}{S_L}\frac{12}{S_T} = 0.414 in^2 \times 70 ksi \times \frac{48in}{24in} \times \frac{12in}{24in} = 29.0 kip/ft$$

As discussed in Section 2.7, the tie bars used as shear reinforcement in the US-APWR SC design are made from A496 deformed wire. As explained in Section 2.5, the interfacial shear demands that occur from flexure are assumed to be resisted only by the headed shear studs welded to the steel plate. This is a conservative assumption that does not account for any interfacial shear force contribution from the tie bars that provide contributions to the out-of-plane shear strength.

7.0 IN-PLANE SHEAR STRENGTH

Design for in-plane shear is in accordance with the requirements for special structural walls of ACI 349-06 Chapter 21. The steel faceplates are treated as both the vertical and horizontal reinforcing steel.

7.1 ACI 349-06 Code Recommendations

ACI 349-06 Section 21.7.4.1 requires that nominal in-plane shear strength V_n of reinforced concrete structural walls be calculated using Equation 21-7 (excerpt shown in Figure 7.1-1).

21.7.4 *Shear strength*

21.7.4.1 V_n of structural walls shall not exceed

$$V_n = A_{cv}(\alpha_c \sqrt{f_c'} + \rho_t f_y) \tag{21-7}$$

where the coefficient α_c is 3.0 for $h_w/\ell_w \le 1.5$, is 2.0 for $h_w/\ell_w \ge 2.0$, and varies linearly between 3.0 and 2.0 for h_w/ℓ_w between 1.5 and 2.0.

Figure 7.1-1 In-Plane Shear Strength

where α_c is a coefficient defining the relative contribution of concrete strength to nominal wall shear strength which is equal to 2 when wall height (h_w) to length (l_w) ratio is larger than 2. In this equation, A_{cv} is the gross area of cross section bounded by web thickness and length of section in the direction of shear force considered, f'_c is the compressive strength of concrete, ρ_n ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{cv}, and f_y is the specified yield strength of reinforcement.

21.7.4.4 For all wall piers sharing a common lateral force, V_n shall not be taken larger than $8A_{cv}\sqrt{f_c'}$, where A_{cv} is the gross area of concrete bounded by web thickness and length of section. For any one of the individual wall piers, V_n shall not be taken larger than $10A_{cw}\sqrt{f_c'}$, where A_{cw} is the area of concrete section of the individual pier considered.

Figure 7.1-2 Upper Bound Limits for In-Plane Shear Strength of RC Walls

In addition to the previous equation additional limitations exist for walls in ACI 349-06 Section 21.7.4.4 (excerpt shown in Figure 7.1-2).

7.2 Experimental Data for In-Plane Shear Strength of SC Walls

In-plane shear loading produces principal tension and compression forces in the composite SC section. The principal tension causes cracking in concrete that significantly decreases the concrete contribution to the overall in-plane shear strength and stiffness. Ozaki et al (Reference 8) have conducted several in-plane shear tests on SC design to determine the fundamental behavior and cyclic performance. The tests included pure in-plane shear loading and slight axial compression combined with in-plane loading.

The specimens had reinforcement ratios (ρ_s) ranging from 2.3 percent to 4.5 percent, but keeping a constant shear stud spacing to plate thickness ratio (b/t_s) of 30 by adjusting the spacing. The experimental results indicated that as the steel plate become thicker, the yield strength and the maximum strength become higher. In addition to that it was found that the addition of nominal axial load (200-400 psi compressive stress) did not have a significant effect on the maximum strength.

Figure 7.2-1 Experimental results from Ozaki tests, and comparison with Equation 7.3-1

The specimens were subjected to cyclic shear strain history and maximum in-plane shear forces obtained from the tests. In Figure 7.2-1 maximum in-plane shear forces obtained from Ozaki tests by subjecting them to pure in-plane shear and shear combined with axial load are compared numerically with the steel plate uniaxial tension strengths (A_sF_y). The specimens were subjected to cyclic shear strain history and maximum in-plane shear forces obtained from the tests. It is seen that the specimens numerically exhibited maximum in-plane forces comparable to tension strength of the steel plates.

Another in-plane test was performed by Sasaki et al (Reference 15) where seven flanged shear wall specimens have been tested under in-plane lateral loading conditions. Of those, five specimens had a height of 1660 mm, one had a height of 1250 mm, and one had a height of 2500 mm. Web SC panel thicknesses were 115 mm, 230 mm, and 345 mm. The surface steel plate thicknesses were constant for all specimens and were equal to 2.3 mm. The corresponding steel reinforcement ratios were 1.33 percent, 2 percent, and 4 percent. Also, the headed stud spacing to plate thickness ratio (b/t_s) was equal to 33.

Figure 7.2-2 shows the maximum force values obtained from specimens tested by Sasaki et al (Reference 15) by subjecting them to lateral in-plane shear, also numerically comparing the maximum shear strength values to A_sF_y of the steel plates of each specimen.

Figure 7.2-2 Experimental results from Sasaki tests, and comparison with Equation 7.3-1

Some specimens exhibited responses governed by yielding and then buckling of the flange before the maximum load was reached causing premature failure of the specimen. This potentially led to shear strength values that are slightly below the in-plane shear strength of the web portions, which was the main intent of the test. The one specimen that has strength slightly below the calculated value (A_sF_y) had the aforementioned failure in flanges before achieving maximum strength.

The in-plane strength upper limits for reinforced concrete walls given in ACI 349-06 are based on test results of squat or short shear walls that had h_w/l_w ratios less than 2. Due to low aspect ratio the failure of these walls included sliding shear failure slightly above the base of squat walls or diagonal crushing of concrete in compression for thin walls. Both of these failure types are not likely to be seen for SC design due to significant contribution of the exterior steel plates under both shear and compressive forces, and the anchorage connection design details.

However, in the absence of extensive experimental data, these upper limits are enforced for the in-plane shear strength of SC walls.

7.3 Conservative Equation for In-plane Shear Strength

Based on the test results obtained from Ozaki et al (Reference 8) and Sasaki et al (Reference 15) the in-plane shear strength of SC walls can be calculated using

Equation 7.3-1 $$V_n = A_s f_y$$

Where, simplifications have been made to the original ACI equation by conservatively neglecting the concrete contribution ($A_{cv}\ \alpha_c\ f'_c{}^{0.5}$).

In the above equation, A_s is the area of the steel plates ($A_s = A_{cv}\ \rho_t$) in the composite section and f_y is the specified yield strength for the steel plates. The in-plane shear strengths calculated using the above equation and the experimental results reported by both Ozaki et al (Reference 8) and Sasaki et al (Reference 15) are compared in Figure 7.2-1 and Figure 7.2-2. These figures indicate good comparison between the experimental results and the in-plane shear strength calculated using Equation 7.3-1.

The strength reduction factor (ϕ) of 0.85 will further improve the comparison and ensure the conservatism of the in-plane shear strength calculated using Equation 7.3-1.

The upper bound for the in-plane shear strength of SC walls is still limited to

Equation 7.3-2 $$V_n = 10 A_{cv} \sqrt{f'_c}$$

for walls sharing a common lateral force. This is a conservative limitation for SC design since the failure modes that are the basis for this requirement, including excessive crack widths and localized concrete crushing, are more critical to RC walls.

8.0 DESIGN FOR COMBINED FORCES

The results from analyses include three membrane force demands (N_x, N_y, and N_{xy}) kip/ft, three moment (M_x, M_y, and M_{xy}) kip-ft./ft., and two out-of-plane forces V_{xz} and V_{yz} kip/ft. In the design process, the out-of-plane shear demands are treated separately as outlined in Section 8.1. The remaining demands are treated as follows:

(i) M_{xy} is directly added to M_x and M_y to increase their magnitude for design. M_x^{total} is the sum of M_x and M_{xy}, and M_y^{total} is the sum of M_y and M_{xy}.

(ii) N_{xy} is used to compute area of steel required in both the x and y directions, and they are equal. The shear reinforcement is equally distributed on both faces. The contribution of the concrete infill to the in-plane shear strength is not included.

(iii) N_x and M_x^{total} are used to compute the area of steel required in the x direction. If N_x is tensile, then the area of steel required for N_x is distributed equally on both faces. The area of steel required for M_x^{total} is calculated assuming no contribution from the steel reinforcement in compression, but the calculated area is added to both faces (tensile and compressive).

(iv) N_y and M_y^{total} are used to compute the area of steel required in the y direction. If N_y is tensile, then the area of steel required for N_y is distributed equally on both faces. The area of steel required for M_y^{total} is calculated assuming no contribution from the steel reinforcement in compression, but the calculated area is added to both faces (tensile and compressive).

The total area of steel required is computed on both faces, and in both directions (A_x^{req} and A_y^{req}), and compared with the area of steel available on both faces in both directions (A_x^{avail} and A_y^{avail})

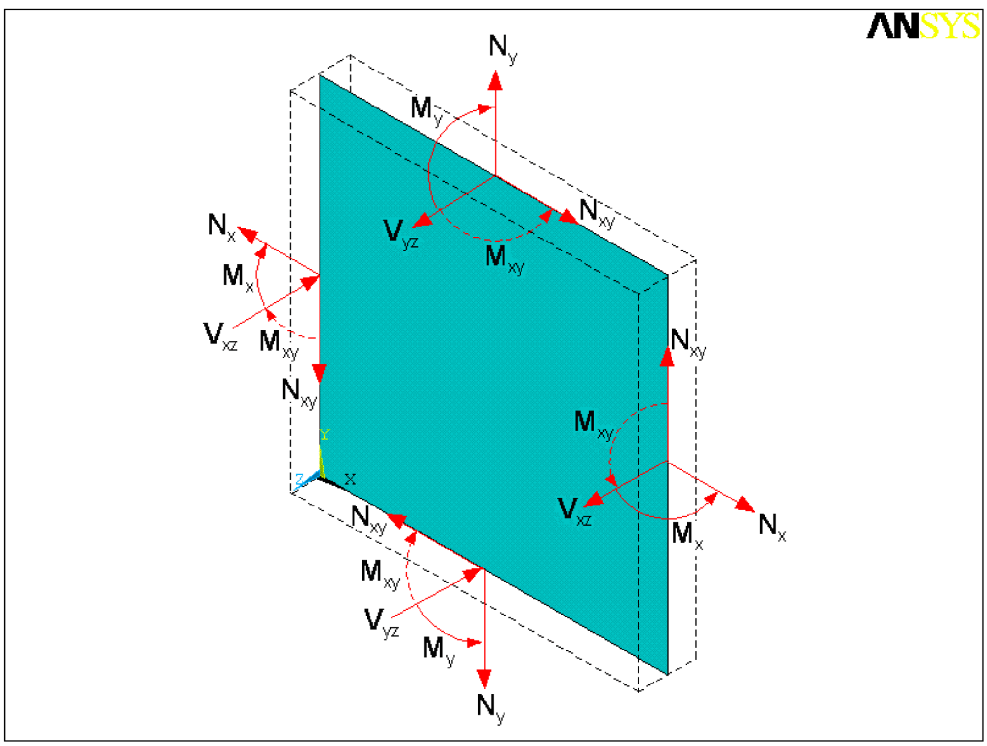


Figure 8.1-1 Force and Moment Demands for Design of SC Walls

8.1 Design for Out-of-Plane Shear Demands

The same shear reinforcement (tie bars) provides resistance (V_s) for the both the out-of-plane force demands V_{xz} and V_{yz}. The required shear reinforcement for both V_{xz} and V_{yz} can be calculated by subtracting the corresponding concrete shear strength contribution (ϕV_c), and dividing by the yield stress of the shear reinforcement (f_{yt}, which is limited to 80,000 psi for deformed wire reinforcement, in accordance with ACI 349-06 Chapter 11.5.2).

Equation 8.1-1 $A_{v1}^{req} = (V_{xz} - \phi_v V_c)/\ \phi_v f_{yt}$

Equation 8.1-2 $A_{v2}^{req} = (V_{yz} - \phi_v V_c)/\phi_v f_{yt}$

where A_v and $\phi_v V_c$ are calculated using equations 6.2-1 through 6.2-4. The provided shear reinforcement A_v^{avail} must be greatern than or equal to $A_v^{req} = A_{v1}^{req} + A_{v2}^{req}$.

8.2 Design for Combined Axial Tension, Flexure, and In-Plane Shear

If the membrane force N_x or N_y is tensile, then the concrete can be assumed to be fully cracked. The area of steel required on each face to resist N_x, M_x^{total}, and N_{xy}, or N_y, M_y^{total}, and N_{xy} can then be computed as shown in Equation 8.2-1 to Equation 8.2-4 given in Table 8.2-1.

Table 8.2-1 Design for Combined Forces with N_x or N_y in Tension

	X-Direction	*Y-Direction*
Equation 8.2-1	$A_{xT}^{req} = \frac{N_x}{\phi_t f_y} \times \frac{1}{2}$	$A_{yT}^{req} = \frac{N_y}{\phi_t f_y} \times \frac{1}{2}$
Equation 8.2-2	$A_{xF}^{req} = \frac{M_x^{total}}{\phi_b f_y \left(T - \frac{3t_p}{2} - \frac{a_x}{2}\right)}$ Where, $a_x = f_y A_{xF}^{req} /(0.85b \cdot f_c')$	$A_{yF}^{req} = \frac{M_y^{total}}{\phi_b f_y \left(T - \frac{3t_p}{2} - \frac{a_y}{2}\right)}$ Where, $a_y = f_y A_{yF}^{req} /(0.85b \cdot f_c')$
Equation 8.2-3	$A_{xV}^{req} = \frac{N_{xy}}{\phi_v f_y} \times \frac{1}{2}$	$A_{yV}^{req} = \frac{N_{xy}}{\phi_v f_y} \times \frac{1}{2}$
Equation 8.2-4	$A_x^{avail} \geq A_x^{req} = A_{xT}^{req} + A_{xF}^{req} + A_{xV}^{req}$	$A_y^{avail} \geq A_y^{req} = A_{yT}^{req} + A_{yF}^{req} + A_{yV}^{req}$

In these Equations, A_{xT}^{req} and A_{yT}^{req} are the area of steel required on each face to resist N_x and N_y, respectively calculated using Equation 8.2-1. A_{xF}^{req} and A_{yF}^{req} are the area of steel required to on each face to resist M_x^{total} and M_y^{total}, respectively, calculated using Equation 8.2-2. Equation 8.2-1 and Equation 8.2-2 were obtained by considering the stress block diagrams shown in Figure 8.2-1. As shown in the Figure, the contribution of the steel in compression to the flexural capacity is not included. A_{xV}^{req} and A_{yV}^{req} are the area of steel required to on each face to resist N_{xy}, calculated using Equation 8.2-3.

As shown in Equation 8.2-4, the total area of steel available on each face in the x and y directions (A_x^{avail} and A_y^{avail}) must be greater than the corresponding required values (A_x^{req} and A_y^{req}), which are the sum of steel areas required for tension, flexure, and in-plane shear.

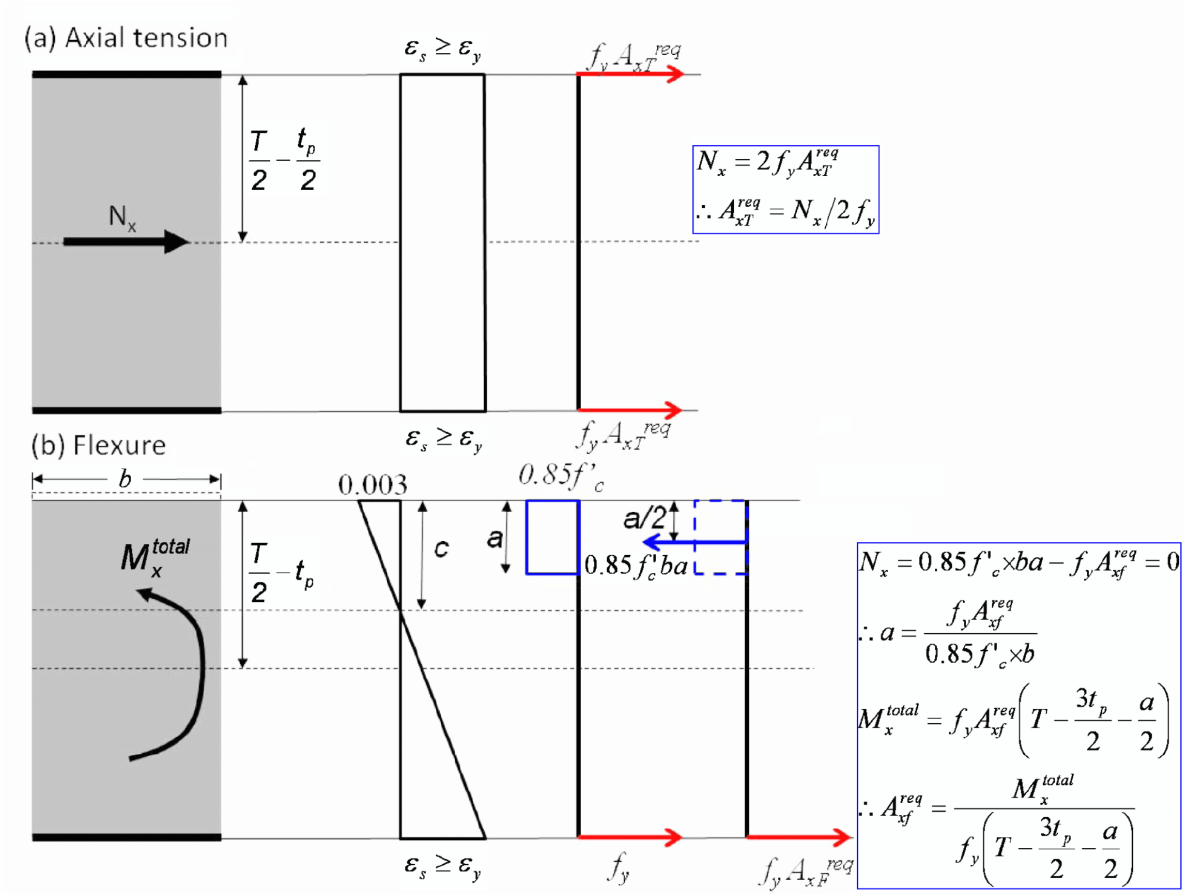


Figure 8.2-1 Section Equilibrium Used to Compute Steel Areas when N_x, N_y in Tension

8.3 Design for Combined Axial Compression, Flexure, and In-Plane Shear

If the membrane force N_x or N_y is compressive, then the concrete will have greater contribution to resisting the moments (M_x or M_y) and the in-plane shear (N_{xy}). In general axial compression increases the section moment capacity and in-plane shear strength of the wall because the concrete in compression contributes to both of these strengths.

The area of steel required on each face to resist $N_x + M_x^{total}$ and N_{xy}, or $N_y + M_y^{total}$ and N_{xy} can be computed as shown in Equation 8.3-1 to Equation 8.3-4 given in Table 8.3-1.

Table 8.3-1 Design for Combined Forces with N_x or N_y in Compression

	X-Direction	*Y-Direction*
Equation 8.3-1	$A_{xF}^{req} = \dfrac{M_x^{total} - \phi_b M_{cx}}{\phi_b f_y (T - t_p)/2}$	$A_{yF}^{req} = \dfrac{M_y^{total} - \phi_b M_{cy}}{\phi_b f_y (T - t_p)/2}$
Equation 8.3-2	$M_{cx} = 0.85 f_c' \times b \times a_x \times \left(\dfrac{T}{2} - t_p - \dfrac{a_x}{2}\right)$ Where, $a_x = \dfrac{N_x + f_y A_{xF}^{req}}{0.85 f_c' b}$	$M_{cy} = 0.85 f_c' \times b \times a_y \times \left(\dfrac{T}{2} - t_p - \dfrac{a_y}{2}\right)$ Where, $a_y = \dfrac{N_y + f_y A_{yF}^{req}}{0.85 f_c' b}$
Equation 8.3-3	$A_{xV}^{req} = \dfrac{N_{xy}}{\phi_v f_y} \times \dfrac{1}{2}$	$A_{yV}^{req} = \dfrac{N_{xy}}{\phi_v f_y} \times \dfrac{1}{2}$
Equation 8.3-4	$A_x^{avail} \geq A_x^{req} = A_{xF}^{req} + A_{xV}^{req}$	$A_y^{avail} \geq A_y^{req} = A_{yF}^{req} + A_{yV}^{req}$

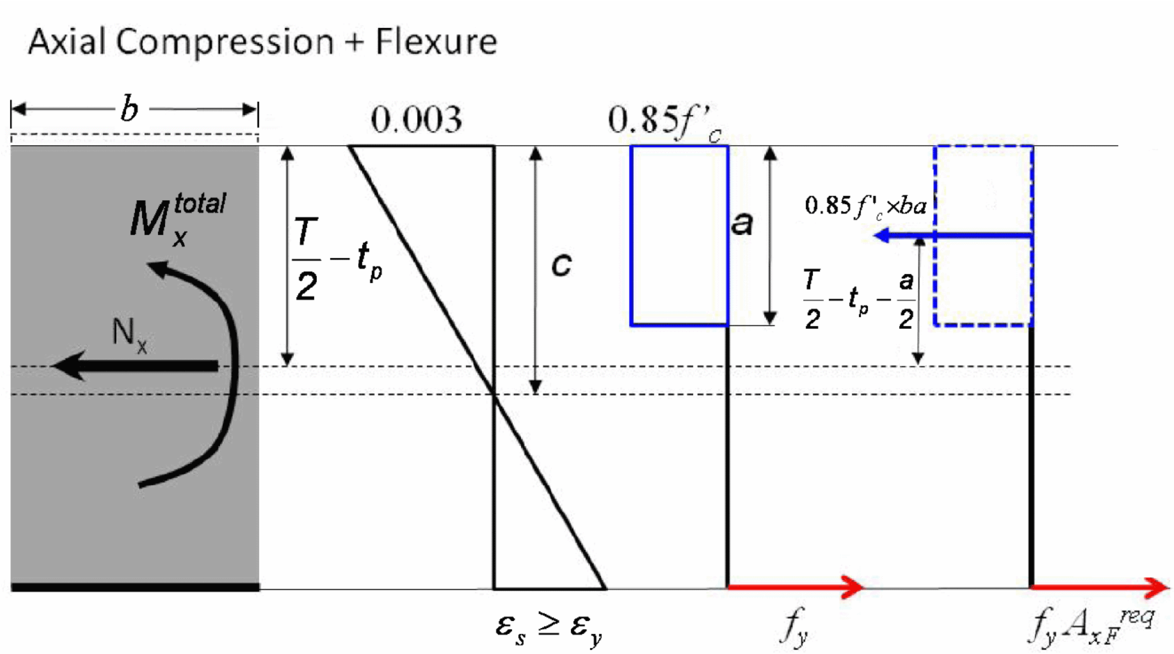
In these Equations, A_{xF}^{req} and A_{yF}^{req} are the area of steel required on each face to resist $N_x + M_x^{total}$ and $N_y + M_y^{total}$, respectively, calculated using Equation 8.3-1. In this Equation, M_{cx} and M_{cy} are the contributions of the concrete to the moment capacity, and can be calculated using Equation 8.3-2.

Equation 8.3-1 and Equation 8.3-2 were obtained by considering the stress block diagrams shown in Figure 8.3-1. As shown in the Figure, the contribution of the steel in compression is not included to be conservative.

A_{xV}^{req} and A_{yV}^{req} are the areas of steel required on each face to resist N_{xy}, calculated using Equation 8.3-3.

As shown in Equation 8.3-4, the total area of steel available on each face in the x and y directions (A_x^{avail} and A_y^{avail}) must be greater than the corresponding required values (A_x^{req} and A_y^{req}), which are the sum of steel areas required for compression + flexure, and in-plane shear.

It is important to note that the stress blocks assumed in Figure 8.3-1 will not be reasonable if the calculated value of (a) is greater than approximately 0.75 times the section thickness. For a 48 in. thick section with 0.5 in. thick steel plates and 4000 psi concrete, this corresponds to having axial compressive forces (N_x) less than or equal to 1170 kip/ft. This upper limit is quite reasonable because the CIS walls are not subjected to such significant axial compression.



$$N_x = 0.85f'_c \times ba - f_y A_{xf}^{req} = 0$$

$$\therefore a = \frac{N_x + f_y A_{xf}^{req}}{0.85f'_c \times b}$$

$$M_x^{total} = f_y A_{xf}^{req}\left(\frac{T}{2} - \frac{t_p}{2}\right) + 0.85f'_c \times ba\left(\frac{T}{2} - t_p - \frac{a}{2}\right)$$

$$\therefore A_{xf}^{req} = \frac{M_x^{total} - M_{cx}}{f_y\left(\frac{T}{2} - \frac{t_p}{2}\right)}$$

$$where, M_{cx} = 0.85f'_c \times ba\left(\frac{T}{2} - t_p - \frac{a}{2}\right)$$

Figure 8.3-1 Section Equilibrium Used to Compute Steel Areas when N_x, N_y in Compression

9.0 ACCIDENT THERMAL CONSIDERATIONS

The thermal effects of accident thermal conditions (T_a) on the US-APWR CIS has been presented and discussed in detail in MUAP-11018 (Reference 6). After the first few minutes, accident thermal conditions produce similar temperatures on both the exterior steel faceplates. The temperature of the concrete infill lags behind that of the steel plates due to its low thermal conductivity and large thermal inertia. The temperature profiles through the composite cross-section and the temperature-time (T-t) curves for different points within the composite cross-section can be computed by conducting 1D heat transfer analysis. The validity of such heat transfer calculation has been demonstrated by Ozaki et al (Reference 17).

9.1 Effects on Design Force Demands

The results of heat transfer analyses indicate nonlinear (parabolic) temperature profiles through the composite cross-section as shown in Figure 9.1, which is excerpted from MUAP-11018 (Reference 6) Section 7 (Reference 6). This parabolic temperature profile results in *through-section cracking* of the concrete infill for all the secondary shield SC walls of the CIS. The occurrence of this *through-section cracking* has been experimentally observed and discussed by Ozaki et al (Reference 17) for in-plane conditions, and Varma et al (Reference 18) for out-of-plane conditions. These researchers have also demonstrated the significant relief in steel plate stresses due to the cracking of the concrete infill.

Figure 9.1-1 Calculated Temperature Profile in 4ft. thick SC Walls

For the parabolic thermal gradient with similar steel plate temperatures on both exterior faces (from 1000 seconds after accident thermal loading), the steel plates will have almost no mechanical stresses in the SC wall interior regions that are away from supports, restraints, and connections. The concrete infill will have through-section cracking in orthogonal directions similar to that discussed in Appendix D of MUAP-11018 (Reference 6) and shown in Figure 9.1-2 below.

The effects of concrete cracking on reducing both the in-plane shear stiffness and the out-of-plane shear stiffness to their respective cracked values has been presented in MUAP-11018 (Reference 6) Appendix D based on the work of Ozaki et al (Reference 17) and Varma et al (Reference 18). As explained in MUAP-11018 (Reference 6), these reduced stiffness values are captured in the linear elastic finite element (LEFE) models for loading condition 'B' that are used to determine the design forces and demands for the accident thermal + earthquake loading condition. Thus, the effects of accident thermal loading and the corresponding concrete cracking have been directly accounted for in calculating the design force demands for condition 'B' according to MUAP-11018 (Reference 6).

Figure 9.1-2 Orthogonal Through-Thickness Cracking Pattern (from Reference 17)

9.2 Effects on Design Capacities

The effects of concrete cracking due to accident thermal loading on the SC wall design capacities calculated according to Sections 3 -8 are as follows:

1) The axial tension strength can still be calculated using Equation 3.1-1, because the concrete infill was assumed to have no influence on the calculated axial tension strength even before cracking induced by the accident thermal condition.

2) The axial compressive strength can still be calculated using Equation 4.1-1. Concrete cracking due to accident thermal loading will cause the compressive force to be resisted by the steel plates only before crack closure occurs. This reduction in the section resistance (stiffness) has already been accounted for in calculating the design force demand as described in MUAP-11018 (Reference 6) and Section 9.1. Concrete crack closure will occur if the axial compression demand is high, and the total axial compressive strength will still be equal to that calculated using Equation 4.1-1.

3) The flexural strength can still be calculated conservatively using Equation 5.3-1. Concrete cracking due to accident thermal loading will initially cause the compressive force of the moment couple in the cross-section to be resisted by the compressive steel plate only before crack closure occurs. However, concrete crack closure will eventually occur as the moment demand (and the corresponding compressive force produced by the moment couple) becomes large. The final moment capacity will be governed by stress state similar to that shown in Figure 5.3-1, and can be calculated conservatively using Equation 5.3-1. The section moment-curvature (M-ϕ) behavior after concrete cracking due to accident thermal loading and the moment capacity has been discussed in detail by Varma et al (Reference 18).

4) The out-of-plane shear strength can still be calculated using Equations 6.2-1 to 6.2-4. As shown by Varma et al (Reference 18), concrete cracking due to accident thermal condition does not seem to have a major influence on the out-of-plane shear strength of SC beams. This is probably because the thermally induced cracks are perpendicular to the steel plates. These cracks have to turn by 45-60^o to develop out-of-plane shear cracks in the concrete, which probably takes approximately the same amount of force as that required to develop new shear cracks due to the anisotropic cracking nature of concrete. Additionally, the V_s contribution to the out-of-plane shear strength is unchanged because the temperature of the steel shear reinforcement embedded in the concrete is not high enough to change the yield strength etc.

5) The in-plane shear strength of SC walls can still be calculated using Equation 7.3-1. The effects of thermally induced concrete cracking on the in-plane shear behavior and strength of SC walls has been investigated by Ozaki et al (Reference 17), which was also presented and discussed in MUAP-11018 (Reference 6) Appendix D. As demonstrated by Ozaki et al (Reference 17), thermally induced concrete cracking does not reduce the in-plane shear strength of SC walls, and the ambient equations for in-plane shear strength can still be used.

6) Since thermally induced concrete cracking has no significant influence on the individual design capacities of SC walls, it is reasonable to assume that the design for combined forces can also be performed according to the Tables and Equations presented in Sections 8.1, 8.2 and 8.3.

9.3 SC Specific Design Issue – Local Buckling

Testing has been performed in Japan to evaluate the occurrence of steel faceplate local buckling due to applied thermal loading. Sekimoto and Kondo (Reference 1 in MUAP-11005) conducted a series of tests on SC walls subjected to temperature changes of 50 – 500^{o}C in steps of 50 or 100^{o}C. For SC specimens with s/t less than 20, no local buckling of the steel faceplates was observed, both in specimens with and without full restraint. Concrete cracking similar to that discussed in Sections 9.1 and 9.2 above was observed in the specimens.

The accident thermal loading on the US-APWR CIS imparts temperature changes (ΔT) in the steel faceplates of the various SC walls that are well within the range of temperatures considered in the Japanese testing. For example, the steel faceplates reach a uniform temperature of 300^oF at approximately 1000 seconds after the postulated pipe rupture, as shown in Figure 9.1-1. This results in a temperature change of approximately 200^oF or 93^oC, given the winter normal operating temperature of 105^oF. Assuming a steel thermal expansion coefficient (α_s^{TH}) of $6.5x10^{-6}$ /^oF, this corresponds to a thermal expansion strain of 1300 microstrain. If the steel plate expansion is fully restrained, which only occurs in the connection regions close to rigid supports or the basemat, the steel plate compressive stress would be 37.7 ksi, which is less than the specified yield stress of 50 ksi. As explained earlier in Section 2.1, all the steel faceplates have slenderness (stud spacing/plate thickness) less than 20, which ensures that local buckling will not occur before yielding in compression. Thus, even if the accident thermal expansion is fully restrained, the steel plate will not undergo yielding or local buckling.

10.0 REFERENCES

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Appendix 1: ICC Evaluation Service, Inc. Report ER-5217

Nelson Deformed Bar Anchor Studs

LEGACY REPORT

ER-5217

Reissued February 1, 2007

Appendix 2: Sample Calculations of Reinforcement Requirements

Problem 1: Pressurizer Wall, Element 51346, Load Combination #4 (D + F + L + E_{ss})

Problem 2: Pressurizer Wall, Element 51346, Load Combination #5 (D + F + L - E_{ss})

URS

CALCULATION SHEET

SHEET: 1 OF 3

PROJ. NO. 29427

CALC. BY: DJW DATE : 9/08/11

PROJECT TITLE: US-APWR

CHKD. BY: EJB DATE : 9/09/11

SUBJECT/FEATURE: SC Wall Area of Steel Faceplate Requirements Using MUAP-11019 Ch. 8

REV.: 0

URS

CALCULATION SHEET

SHEET: 2 OF 3

PROJ. NO. 29427

CALC. BY: DJW DATE : 9/08/11

CHKD. BY: EJB DATE : 9/09/11

PROJECT TITLE: US-APWR

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REV.: 0

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CALCULATION SHEET

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REV.: 0

Appendix 3: Confirmatory Test Matrix for SC Wall Design Criteria

Appendix 4: Design Criteria for Primary Shield Structure

Appendix 5: Reference 8, "Out-of-Plane Shear Behavior of SC Composite Structures"

Appendix 6: Reference 19, "In-Plane Shear Behavior of SC Composite Walls: Theory vs. Experiment"

Appendix 7: Reference 18, "Steel-Plate Composite (SC) Walls: Analysis and Design Including Thermal Effects"