CONTAINMENT INTERNAL STRUCTURE: ANCHORAGE AND CONNECTION DESIGN AND DETAILING

MUAP-11020

Revision 1

Non-proprietary Version

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REVISION HISTORY

Revision	Page	Description
0	All	Initial Issue
1	All	Responses to RAI 879-6196 Question 03.08.04-52 (submitted by UAP-HF-12054 and UAP-HF-12076) and RAI 931-6467 Questions 03.08.03-81 through 03.08.03-90 (submitted by UAP-HF-12197) have been incorporated into Revision 1. The changes are summarized as follows: • Clarified definitions for connections, connection regions, and connected parts. • Incorporated changes to clarify use of, and design criteria for full strength and overstrength connections. • Clarified load increase factor application for overstrength connection design procedure relative to HCLPF requirements. • Revised SC wall basemat anchorage example in Chapter 7 and provided summaries of wall-to-wall and slab-to-wall connections in Appendices A and B. • Provided additional explanation of confirmatory testing in Appendix C.

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DESIGN APPROACH AND EXECUTIVE SUMMARY

Scope

This Technical Report (TeR) presents the design criteria and approach for connections involving Steel Concrete (SC) walls only. Connections involving only Reinforced Concrete (RC) walls, slabs, or other structures are designed in accordance with the requirements of ACI 349-06 (Reference 1). Connections of steel members and structures are designed in accordance with the requirements of AISC N690 (Reference 2).

Relationship with MUAP-11019 for SC Walls

This TeR addresses anchorage and connection design and detailing for SC walls in the US-APWR Containment Internal Structure (CIS). The SC walls themselves are designed according to TeR MUAP-11019 Rev. 1 (Reference 3). As presented in TeR MUAP-11019 Rev. 1, the SC walls are detailed to prevent SC specific failure modes such as local buckling, interfacial shear failure, etc. The SC walls are designed using ACI 349-06 code requirements for RC structures with added conservatisms based on experimental results.

Connection Regions

As explained in TeR MUAP-11019 Rev. 1, the SC wall connection regions are those portions of the walls outside of the connections that will dissipate energy through ductile inelastic response in the event of overloading. Force transfer from the composite SC wall to the supports or connected structures occurs within these connection regions. Additionally, the connection regions also serve as transition regions wherein the steel faceplates and concrete infill of SC walls redistribute forces according to their relative stiffness and develop composite action.

The concept of connection regions for SC walls is similar to that of load transfer regions for composite columns specified by AISC 360-10, Chapter I, Section I6 (Reference 4). The SC walls themselves are designed according to TeR MUAP-11019 Rev. 1 (Reference 3). The connections and the connection regions of SC walls are designed and detailed according to this TeR.

Detailing of Connection Regions to Achieve Local Ductility

The connection regions of SC walls are detailed to prevent SC specific failure modes such as local buckling, interfacial shear failure, and section delamination or splitting failure. They are also detailed to prevent non-ductile failure modes such as out-of-plane shear failure from governing behavior. This is achieved by providing adequate shear reinforcement (tie bar area and spacing) in the connection region so that the flexural yielding limit state governs for shear span ratios greater than 2.0, i.e., the out-of-plane shear design strength ($\phi_v V_n$) of the SC wall connection region is greater than its flexural capacity divided by two times the section thickness ($M_r/2T$).

Thus, all non-ductile failure modes are prevented from occurring in the connection region, and the connection has the ductility to undergo inelastic deformations and dissipate energy, if needed, for beyond design-basis events.

Connection Design Philosophy to Achieve Global Ductility

Capacity design is a fundamental aspect of the seismic design philosophy for structures (as discussed in ASCE 41-06, Reference 5). It can be achieved by: (i) designing full strength connections, i.e., connections that are stronger than the expected strength of the weaker of the two connected parts, and (ii) detailing the connected parts to have adequate ductility to undergo inelastic deformations and dissipate energy for beyond design-basis events.

As explained in TeR MUAP-11019 Rev. 1 (Reference 3) and in this TeR, the entire expanse of each SC wall is detailed to prevent non-ductile failure modes. The connection regions are detailed explicitly to prevent out-of-plane shear failure modes from governing the behavior. This is done because connection regions are located close to supports or reaction points, and are expected to have large out-of-plane shear forces due to the associated discontinuities. Then, the specific section detailing selected for the connection regions is also applied to the remainder of the SC wall, for simplicity and good detailing practice. As a result, the entire expanse of each SC wall utilizes the same section detailing, and non-ductile failure modes are prevented from occurring in any region of the wall.

Full Strength Connection Design

The full strength connection is first designed to transfer the individual expected strengths (axial tension, in-plane shear, out-of-plane shear, or bending moment) of the weaker of the two connected parts. Clearly identifiable force transfer mechanisms are used to transfer each of the individual strengths. These force transfer mechanisms involve connectors that are well established in practice; for example, steel welding, welded rebar couplers, direct shear, etc. The connectors are then designed using applicable design codes; for example, ACI 349-06 or AISC N690.

The full-strength connection is then checked for the design force and moment demands calculated from the Linear Elastic Finite Element (LEFE) analyses of the CIS for design-basis load combinations. These design force and moment demands are assumed to occur concurrently, which is conservative. The connection adequacy is assessed by: (i) calculating the concurrent (superimposed) demands on the connectors that are involved in the force transfer mechanisms for the design force and moment demands, and (ii) checking the connector strength while accounting for interaction effects in accordance with the applicable design codes. The codes applicable to the load combination demand checks include ACI 349-06 and AISC N690, as well as ASME Section III Division 2 (Reference 6) in the case of the basemat anchorage connection design.

In summary, the full strength connection is designed to have predominantly elastic behavior and adequate strength for design basis loads and load combinations. It is further designed to be stronger than the expected strengths of the weaker of the connected parts. Therefore, for beyond design-basis loads and load combinations, inelastic deformations and energy dissipation will occur in the weaker of the connected parts. The connected parts are detailed accordingly to have good ductility and to prevent non-ductile failure modes (e.g. out-of-plane shear) and SC specific failure modes.

This full strength connection design philosophy is consistent with the concrete anchorage design philosophy in ACI 349-06 Appendix D. For example, Section D.3.6.2 indicates the need

for full strength connection design that is capable of developing the expected yield strength of the connected part.

Overstrength Connection Design Philosophy

In some limited situations, it is not feasible to provide a full strength connection. For example, a full strength connection design may not be feasible if the associated SC wall or RC slab is significantly overdesigned with respect to the linear elastic finite element (LEFE) calculated force and moment demands due to radiation shielding requirements. In such cases, the connection is designed to provide direct overstrength in the connection design with respect to the calculated force and moment demands.

The overstrength connection design philosophy, adopted here, requires the connection to be designed for 200% of the seismic demands plus 200% of non-seismic demands calculated from LEFE analyses of the CIS for loads and load combinations.

The goal of this connection design philosophy is to provide High Confidence of Low Probability of Failure (HCLPF) for 1.67 times the Safe-Shutdown Earthquake (SSE), which is accomplished conservatively by increasing the SSE force and moment demands by a factor of 2.0. This factor of 2.0 achieves the seismic margin of 1.67 while accounting for the slight difference in the failure probability levels required by HCLPF and those provided inherently by ACI 349-06 or similar code equations for component or connector strength. Further discussion of the basis for the 2.0 load increase factor relative to minimum HCLPF requirements is provided in response to RAI 879-6196 Question 03.08.04-52.

The overstrength connection design philosophy in this TeR is more conservative than the requirement for HCLPF of 1.67 SSE. It is important to note that the overstrength connection design philosophy will be used only in limited situations where a full-strength connection cannot be provided. Furthermore, the overstrength connection design philosophy will be used only for the particular force transfer mechanisms in the connection that cannot be designed to achieve the full strength of the connected wall. Additionally, all connectors utilized in overstrength connection force transfer mechanisms will be designed to exhibit ductile failure modes involving steel yielding.

The full strength connection design philosophy is the default design philosophy unless clearly identified.

Anchorage Design: Example of Full Strength Connection Design

This TeR includes an illustrative design of the SC wall-to-concrete basemat anchorage, which embodies the full strength connection design philosophy described above. The anchorage connection is designed to be stronger than the connected SC wall. As shown, it is designed to develop (or transfer) the individual expected strengths of the SC wall in axial tension, in-plane shear, out-of-plane shear, and flexure. The expected strengths are estimated using applicable material overstrength factors from ASCE 41-06, which are similar to those in ACI 349-06 Section 21.5.1.

The anchorage connection uses clearly identifiable force transfer mechanisms for axial tension, in-plane shear, out-of-plane shear, and bending moment. The force transfer mechanisms utilize welded steel connections, welded couplers, and shear studs. These connectors are well-

established in practice and can be designed according to the appropriate AISC N690 or ACI 349-06 code provisions.

The anchorage connection design is further checked for the concurrent design force and moment demands calculated from the LEFE of the CIS. This check involves determining the superimposed (concurrent) demands on the critical connectors, and checking their (connector) strengths while accounting for interaction effects in accordance with applicable design codes.

Experimental Confirmation

As explained in this TeR, the connection design philosophy requires the use of clearly identifiable force transfer mechanisms to develop the individual expected strengths of the weaker of the connected parts. These force transfer mechanisms are provided using standard connectors that are well-established in practice, and can be designed using appropriate design codes (ACI 349-06, or AISC N690). These requirements and the full strength connection design philosophy minimize the need for experimental qualification of the SC wall connections / anchorages.

Additionally, the 1/10th scale test of a complete CIS and the 1/6th scale test of the primary shield structure (see TeR MUAP-11005 Rev. 1, Reference 7) provide valuable information regarding the overall behavior and strength of the US-APWR CIS. The design criteria and approach in this TeR will result in connection details for SC walls of the US-APWR CIS that provide equivalent or better performance than the connections utilized in the 1/10th scale test or the 1/6th scale tests conducted in Japan. This is achieved in this TeR by providing comprehensive design criteria that develop all the individual strengths of the connected walls and check for all concurrent design demands.

Finally, six series of confirmatory tests were performed as part of this project, with several tests confirming the strength and ductility of the US-APWR connections. The tests are listed in Appendix C.

The tests confirm that the calculation procedures and design assumptions proposed in TeR MUAP-11019 Rev. 1 and this TeR are conservative for the following aspects of the US-APWR SC wall design:

1. Shear strength of the steel headed studs
2. Out-of-plane shear strength of the SC walls
3. Out-of-plane shear strength of the SC walls including accident thermal conditions
4. Concrete joint shear strength for SC wall-to-wall connections
5. Direct shear strength of the rebar couplers
6. Ductility and out-of-plane shear strength of the basemat anchorage
7. Ductility and in-plane shear strength of the basemat anchorage.

Report Outline

- Section 1.0 defines key terms relative to connection design and presents the determination of the design force and moment demands for SC wall connections.
- Section 2.0 presents the connection design options and philosophies in more detail.
- Section 3.0 presents the calculation of the required individual strengths for (i) full strength connections, and (ii) overstrength connections.
- Section 4.0 presents the determination of the connection design strength by identifying the force transfer mechanisms and associated connectors.
- Section 5.0 presents the design approach for the connectors involved in the force transfer mechanisms.
- Section 6.0 presents the evaluation of the connection for the concurrent design force and moment demands from Section 1.0.
- Section 7.0 presents a design example embodying the connection design criteria and approach presented in Sections 1.0 – 6.0.
- Discussion of the methodology of connection design for SC wall-to-wall joints is presented in Appendix A.
- Discussion of the methodology of connection design for SC wall-to-RC slab connections is shown in Appendix B.
- The US-APWR confirmatory test matrix is shown in Appendix C.

LIST OF ACRONYMS

The following list defines the acronyms used in this document.

ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
CIS	Containment Internal Structure
FE	Finite Element
HCLPF	High Confidence of Low Probability of Failure
LEFE	Linear Elastic Finite Element
PCCV	Prestressed Concrete Containment Vessel
RC	Reinforced Concrete
RWSP	Refueling Water Storage Pit
SC	Steel Concrete
SSE	Safe-Shutdown Earthquake
T	Thickness
TeR	Technical Report
NRC	United States Nuclear Regulatory Commission

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1.0 INTRODUCTION AND DEFINITIONS

As discussed in MUAP-11019 Rev. 1 (Reference 3), the overall structural design and detailing process for the US-APWR Steel Concrete (SC) walls consists of three basic steps, including 1) detailing the SC walls to prevent SC-specific limit states and failure modes from controlling the design, 2) designing the SC walls for their applied forces and moments using conservative forms of ACI 349-06 (Reference 1) strength equations, and 3) designing and detailing the SC wall anchorages and connections to provide sufficient force transfer capability and ductility. Technical Report (TeR) MUAP-11019 Rev. 1 addresses the first two aspects of this overall structural design and detailing process, while this TeR focuses on the third aspect. The SC wall connection design and detailing requirements are presented relative to the following key terms:

Connection: The connection is defined as the assembly of steel connectors (including welds, shear studs, anchor bars, rebars, and rebar couplers) and the surrounding concrete materials anchoring the rebar or providing bearing resistance, that participate in the force transfer mechanisms for tension, compression, in-plane shear, out-of-plane shear, and out-of-plane flexure between two connected parts. It does not include any portions of the SC walls being connected. The SC wall basemat anchorage is one specific type of connection.

Connected Parts: The connected parts are defined as the structural members (i.e., the SC walls, Reinforced Concrete (RC) slabs, or the RC basemat) that are being connected to one another and transferring forces via the connection defined above.

Connection Region: The connection region is defined as the portion of the connected SC wall outside of the connection that is specifically designed to undergo ductile yielding and energy dissipation during overloads. Force transfer from the composite SC wall to the supports or connected structures occurs within these connection regions. Additionally, the connection regions serve as transition regions wherein the steel faceplates and concrete infill of SC walls redistribute forces according to their relative stiffness and develop composite action.

It is noted that a specific geometrical distinction of the connection regions from the interior portions of the SC walls is not necessary for the US-APWR SC wall design. As further discussed later in this TeR, the rectangular steel plate tie bar design for the US- APWR SC walls eliminates the need for this distinction. The entire expanse of each SC wall utilizes the same unique tie bar and shear stud detailing. As a result, steel faceplate yielding will be the governing failure mode, and non-ductile failure modes will not govern the behavior or strength of the SC walls in any region.

During the design process, the connection regions of the SC walls are initially designed according to the design and detailing requirements presented in this TeR. Secondly, they are subjected to the design procedures given in TeR MUAP-11019 Rev. 1 for confirming design adequacy of the SC walls for the applied loads. More specifically, the steel faceplates, tie bars, and concrete of the entire SC wall, including the portions within the connection region, are evaluated in accordance with the combined force procedures and equations presented in TeR MUAP-11019 Rev. 1, Chapter 8.

1.1 Connection Design Demands

The *connection design demands* for all loading combinations are determined from the results of Finite Element (FE) analyses conducted in accordance with TeR MUAP-11018 Rev. 1 (Reference 8). As explained in TeR MUAP-11018 Rev. 1, Linear Elastic Finite Element (LEFE) analyses are conducted for static loading conditions, and also for condition ‘A’ (operating thermal + seismic loading) and condition ‘B’ (accident thermal + seismic loading), using appropriate stiffness values accounting for the effects of concrete cracking where applicable. A summary of the applicable loading combinations considered is provided in TeR MUAP-11018 Rev. 1, Table 3-1.

The results from the FE analyses can be used to determine the design demands per unit length of the connection, namely, (i) membrane axial force N_u, (ii) membrane in-plane shear force V_u^{in}, (iii) out-of-plane shear force V_u^{out}, and (iv) out-of-plane bending moment M_u. These are illustrated below in Figure 1.1-1.

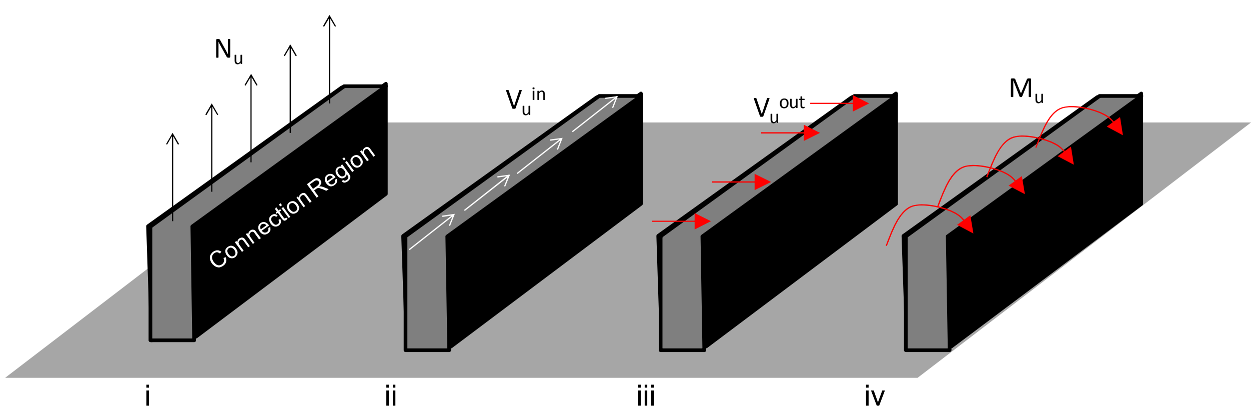

Figure 1.1-1 Connection Design Demands Per Unit Length

Each *connection design demand* ((i) – (iv) above) is differentiated into demands due to seismic loading (condition A or B), and demands due to non-seismic loading (static loads, thermal loads, etc.)

2.0 CONNECTION DESIGN OPTIONS FOR SC-TYPE WALLS

Two different connection design philosophies are considered:

1) The *full strength connection* design philosophy develops the expected strength of the weaker of the two connected parts, so that ductile behavior is ensured, with yielding and inelasticity occurring away from the connection in one of the connected SC walls (or RC slabs as the case may be). This ductile design approach is consistent with the concrete anchorage design provisions given in ACI 349-06, Appendix D.

2) The *overstrength connection* design philosophy develops significant overstrength with respect to the design force demands on the connection. For each demand type, the connection is designed for 200% of the seismic force demands plus 200% of the non-seismic force demands on the connection. Additionally, all connectors utilized in overstrength connections are designed to exhibit ductile failure modes involving steel yielding.

The full strength connection design philosophy is planned for use in all SC wall connections in the US-APWR Containment Internal Structure (CIS), including all connections between SC walls and all anchorage connections between the SC walls and the nuclear island basemat.

The overstrength connection design philosophy is to be used in limited circumstances where a full strength connection cannot be provided, such as connections involving SC walls that have been significantly overdesigned with respect to the design force demands. For example, the overstrength design approach may be necessary for connections involving very thick SC walls that have excessive capacity as a result of sizing for shielding purposes rather than structural strength. In such cases, the overstrength design approach is to be utilized only for the particular force transfer mechanisms in the connection that cannot be designed to achieve the full strength of the wall.

3.0 CONNECTION REQUIRED STRENGTHS

The *connection required strengths* for SC walls are calculated as described in Sections 1.1 and 2.0 of this TeR. The majority of the connections are designed using the full strength connection design philosophy. The full-strength connection design philosophy requires the connections to be stronger than the connected SC walls, and the connection regions to be detailed to undergo yielding and to dissipate energy during extreme seismic or accident events. Non-ductile failure modes such as out-of-plane shear failure or steel faceplate fracture (before significant yielding) are prevented in the connection regions for shear span ratios greater than 2.0, as explained below. This ensures that such failure modes do not interfere with or prevent energy dissipation by the SC walls and overall ductile behavior of the containment internal structure during extreme events.

3.1 Full Strength Connection Required Strengths

For full strength connections, the required strengths for the individual demands (i) – (iv) are defined in terms of the expected strengths of the weaker connected part, as follows:

(i) The required axial tension strength (N_r) per unit length of the connection is 125% of the nominal yield strength of the weaker of the connected parts in tension. The 1.25 factor used to obtain the expected tensile strength is conservatively selected to be consistent with the factor used for flexure, as discussed in (iii) below. Compressive stresses are transferred through the connection via direct bearing of the concrete.

(ii) The required in-plane shear strength (V_r^{in}) per unit length of the connection is 110% of the nominal in-plane shear strength of the SC wall. The nominal in-plane shear strength of SC walls is defined in TeR MUAP-11019 Rev. 1 as A_sF_y, where A_s is the total area of the steel faceplates per unit length of the wall, and F_y is the nominal yield strength of the steel. The expected strength of the A572 Gr. 50 steel faceplates is defined using the material overstrength factor of 1.1 given in ASCE 41-06 Table 5-3. This is also consistent with the R_y factor of 1.1 given in AISC 341-10 (Reference 9) for A572 Gr. 50 plate material.

(iii) The required flexural strength (M_r) of the connection and the connection region is the expected flexural strength of the weaker of the connected parts. This is defined as 125% of the nominal yield strength of the primary tension reinforcement ($1.25F_yA_s$), where A_s is the area of one faceplate per unit length of the wall. The 1.25 factor accounts for both material overstrength and strain hardening of the A572 Gr. 50 steel faceplates. Strain hardening is included in the expected flexural strength calculation because of the higher strain rates that occur in the tension steel faceplate as the plastic moment capacity of the composite section is developed. This is similar to the 1.25 factor in ACI 349-06 Section 21.5.1.1, which also accounts for both material variability and strain hardening in the longitudinal reinforcement of moment frames (see also ACI 352R-02 Section 3.3.4, Reference 10). It is important to note that rebar steel has more variability in yield stress than plate steel material (ASCE 41-06).

As mentioned above in (i), the 1.25 factor applied to flexural strength is also conservatively assigned to tension strength for consistency. This factor is conservative for expected tensile strength because direct tension on the wall, like in-plane shear, causes simultaneous yielding of both faceplates and thus corresponds to the yield strength of the faceplates before strain hardening.

(iv) The required out-of-plane shear strength (V_r^{out}) of the connection and the connection region is the larger of the following:

(a) Shear force calculated in accordance with ACI 349-06, Sections 21.3.4 and R21.3.4.

(b) Shear force required to develop flexural capacity over a shear span ratio of 2, i.e., $M_r/2T$, where T is the section thickness or depth.

Requirement (iv)(b) is generally the more conservative of the two requirements and ensures that flexural yielding will occur before out-of-plane shear failure in the connection region for shear span ratios greater than or equal to two, as discussed in greater detail below in Section 3.2. This requirement is necessary for full strength design of SC wall connections for a number of reasons:

1. The SC walls are not frame members. The seismic response of the US-APWR CIS is more complex and different from the behavior of RC building frames, which are the subject of ACI 349-06, Section 21.3.4.
2. The SC walls of the CIS must be designed for loading combinations involving simultaneous accident thermal loading and seismic loading. These loading combinations are unique to safety-related nuclear structures designed using ACI 349-06. However, ACI 349-06 Section 21.3.4 requirements are based on ACI 318-05 Section 21.3.4 (Reference 11) requirements, which were established for commercial RC building frames.
3. The double curvature bending moment diagrams shown in ACI 349-06, Section R21.3.4 may not apply to SC walls of the CIS, particularly when combined with accident thermal loading conditions.

3.2 Additional Discussion of Full Strength Design for Out-of-Plane Loading

The out-of-plane bending behavior of an SC wall depends on the detailing of the section including the steel faceplate reinforcement ratio, the tie bar area and spacing, and the shear stud size and spacing. SC walls can reach their out-of-plane strength due to limit states associated with:

1. Flexural yielding of the steel faceplates. This limit state corresponds to the flexural strength (moment capacity) of the SC wall and results in ductile behavior after yielding.

2. Flexural and shear cracking of the concrete followed by tension yielding of the tie bars. This limit state corresponds to the out-of-plane shear strength of the SC wall and results in a non-ductile failure mode.

3. Bond shear cracking of the concrete and shear fracture of the studs. This limit state corresponds to the interfacial shear strength of the SC walls and results in a non-ductile failure mode.

In accordance with the full-strength connection design philosophy, the SC wall connection regions are detailed such that the hierarchy of the above limit states is (1), (2), and (3), i.e., flexural yielding with ductile behavior will occur before out-of-plane shear failure, and out-of-plane shear failure will occur before interfacial shear failure. This is achieved as follows:

Limit state (1): The flexural capacity (M_r) of the SC wall section is computed first assuming the steel faceplate (longitudinal reinforcement) to be at a stress level corresponding to 125% f_y. As discussed above, this 125% of the nominal yield stress (f_y) accounts for the expected variability and the potential strain hardening of the steel faceplate material under flexural loading. The flexural capacity is required to be developed over a shear span ratio of 2.

Limit state (2): The out-of-plane shear behavior and strength of SC walls depends on: (a) the section details including the tie bar size and spacing, and (b) also on the shear span a/T ratio, where a is the shear span length and T is wall thickness. For shear span ratios greater than or equal to 2.0, the out-of-plane shear behavior is governed by flexural shear cracking of the concrete, where flexural cracking occurs first and then propagates with shear cracking. The cracked concrete develops compression struts and engages the steel tie bars as tension ties. The out-of-plane shear strength can be computed using ACI 349-06 code equations. TeR MUAP-11019 Rev. 1 conservatively reduces the ACI 349-06 calculated shear strength to account for size effects on the cracking strength of concrete.

As the shear span ratios become smaller than 2.0, deep beam behavior dominates and the out-of-plane shear behavior is governed by D-region effects with the formation of multiple concrete compression struts and arching action in the cracked concrete as shown below in Figure 3.1-1. The steel faceplates act as tension ties for the arching action in the cracked concrete. The resulting out-of-plane shear strength can be much higher than that calculated using ACI 349-06 code equation (modified by TeR MUAP-11019 Rev. 1) because this behavior and failure mode are significantly different from those assumed by the equations.

Experimental results indicate that the out-of-plane shear strength for shear spans less than two is much larger than that calculated using ACI 349-06 code equations, due to arch effects (Takeuchi, et al. 1999, Reference 12). For shear spans greater than two, out-of-plane shear failure can occur due to flexural shear cracking or diagonal tension cracking (Varma, et al., November, 2011, Reference 13). For example, consider the results from the experimental database (provided in TeR MUAP-11005 Rev. 1 Appendix B) on out-of-plane shear behavior. The majority of the tests in this database had shear span ratios less than or equal to 1.5. As shown in Figure 3.1-2, the experimental shear strength is much greater than that calculated using ACI 349-06 code equations (modified by TeR MUAP-11019 Rev. 1).

Thus, for shear span ratios less than 2.0, the failure mode is not governed by flexural behavior or even conventional out-of-plane shear behavior. For such cases, the out-of-plane shear strength is significantly greater than that calculated using design equations and enforced in the design process. Thus, for shear span ratios less than 2.0, there is significant reserve margin in the design.

Limit state (3): The interfacial shear strength of the shear connectors between the SC faceplates and the concrete infill is required to be greater than or equal to the out-of-plane shear strength. This is accomplished using the design procedures in TeR MUAP-11019 Rev. 1 Section 2.5.

Thus, the SC wall connection regions are designed to have ductile behavior and dissipate energy when the structure is subjected to extreme loading. The shear span ratios of the SC walls of the CIS will be calculated upon completion of the LEFE analysis for calculating demands. These shear span ratios will be calculated as the demand (M_u) divided by the corresponding out-of-plane shear demand (V_u) multiplied by the wall thickness (T). These shear

span ratios will be reported along with the basic design, and demonstrated for the connection regions to be greater than or equal to 2.0.

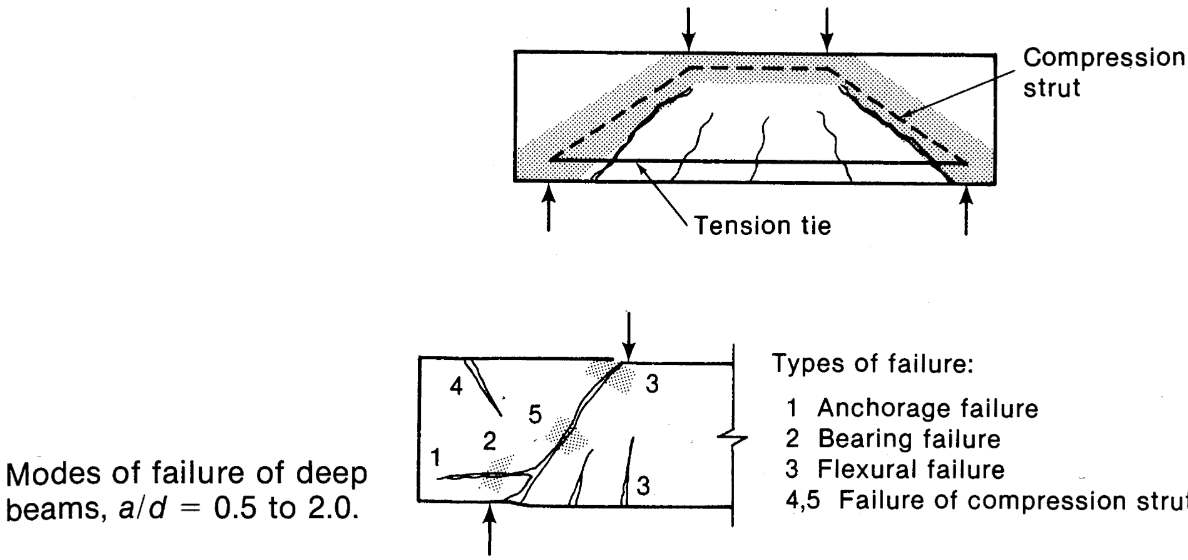

Figure 3.1-1 Modes of Failure of Deep Beams, *a*/*d* = 0.5 to 2.0.
(MacGregor, Reference 14)

Figure 3.1-2 Out-of-Plane Shear Strength vs. Shear Span Ratio

Ratio of Out-of-Plane Shear Strength (V_n^{exp}) of Specimens with Shear Span Ratio less than 2.0 from Experimental Database (see MUAP-11005 Rev. 1, Appendix B), Divided by the Out-of-Plane Shear Strength Calculated using TeR MUAP-11019 Rev. 1 Equations.

3.3 Overstrength Connection Required Strengths

For overstrength connections, the required strengths (N_r, V_r^{in}, V_r^{out}, and M_r) are calculated as 200% of the seismic force demands plus 200% of the non-seismic force demands. The seismic force demands correspond to Safe-Shutdown Earthquake (SSE), and they are increased to provide High Confidence of Low Probability of Failure (HCLPF) for 167% of SSE. Using 167% of the seismic force demands would not achieve HCLPF for 167% of SSE. The load increase factor required to achieve the seismic margin of 1.67 is in excess of 1.67 but less than 2.0. As a result, the use of a load increase factor of 2.0 ensures that a minimum HCLPF seismic margin factor of 1.67 is achieved. Further margin is obtained by also factoring the non-seismic force demands, including dead, live, fluid, accident pressure, and operating/accident thermal demands.

4.0 CONNECTION DESIGN STRENGTH

The *connection design strengths* (ϕN_n, ϕV_n^{in}, ϕV_n^{out}, or ϕM_n) for each of the corresponding connection required strengths (N_r, V_r^{in}, V_r^{out}, or M_r) are calculated using the applicable *force transfer mechanism* identified in accordance with Section 4.1 and the *design strength* of its contributing *connectors* calculated in accordance with Section 5.0.

4.1 Force Transfer Mechanism

For each of the required strengths (N_r, V_r^{in}, V_r^{out}, and M_r), a clearly identifiable *force transfer mechanism* is identified and provided. Each *force transfer mechanism* shall involve *connectors* of the same type in the connection region. If more than one *force transfer mechanism* is possible for resisting a particular demand type, then the one with the largest *connection design strength* is the governing *force transfer mechanism*.

Connectors used in this design will consist of steel headed stud anchors, tie bars, reinforcing bars and dowels, shear lugs, embedded steel shapes, steel welds and bolts, rebar mechanical couplers, and direct bearing in compression. Direct bond transfer between the steel plate and concrete is not considered as a valid *connector* or *force transfer mechanism*.

5.0 DESIGN OF CONNECTORS

The force transfer mechanism for each demand type is used to compute the *required strength* for its contributing *connectors*. The *design strengths* for different connectors are computed as follows:

1. For steel headed stud anchors, the design strength shall be determined in accordance with ACI 349-06 Appendix D.
2. For welds the design strength shall be determined in accordance with AISC N690 Section Q1.5.3.
3. For bending of the baseplate in the basemat anchorage connection, the design strength shall be determined in accordance with AISC N690 Section Q1.5.1.4.3.
4. For compression transfer via direct bearing in concrete, the design strength shall be determined in accordance with ACI 349-06 Section 10.17. Design bearing strength for concrete placed against shear lugs shall be determined in accordance with ACI 349-06 Section D.4.6.
5. For shear-friction load transfer mechanisms, the design strength shall be determined in accordance with ACI 349-06 Section 11.7.
6. For rebar anchors, the design strength in tension and in shear shall be calculated using ACI 349-06 Appendix D, Section D.5.1 and Section D.6.1, respectively.
7. For joint shear strength, the design strength shall be determined in accordance with ACI 349-06 Section 21.5.3.1.

Connectors are designed such that their *design strength* is greater than their *required strength*. In all cases the design strength is calculated using the appropriate code specified strength reduction (ϕ) factors. See Section 6.2 for additional design requirements from ASME Section III, Division 2, which are applicable to the basemat anchorage connection since it involves the Containment Pressure Boundary Liner.

6.0 CONNECTION EVALUATION FOR COMBINED FORCES

The connection design performed in accordance with Sections 2, 3, and 4 of this TeR is further evaluated for the combined force demands calculated according to Section 1.1 (N_u, V_u^{in}, M_u, and V_u^{out}).

The *force transfer mechanisms* of Section 3.0 are used with the individual force demands (N_u, V_u^{in}, V_u^{out}, and M_u) to determine the required strengths for its contributing connectors. The *total required strength* (R_u) for the *connectors* are calculated as the superposition of the required strengths from all the individual demands.

The *total required strength* is compared with the *connector design strengths* (ϕR_n) calculated as described in Sections 5.0 and 6.2 of this TeR, while accounting for the effects of superposition of force demands as applicable. For example, rebar anchors may be subjected to superposition of tension and shear forces, the interaction of which is considered explicitly.

6.1 Summary of Connection Design Approach

The connection design approach outlined in Sections 1.0 – 6.0 consists of typically:

(1) Designing the connection to transfer each of the full expected strengths (axial tension strength N_r, in-plane shear strength V_r^{in}, flexural strength M_r, out-of-plane shear strength V_r^{out}) of the weaker of the connected members.

or

designing the connection to develop significant overstrength with respect to the design demands; i.e., 200% of the seismic demands plus 200% of the non-seismic demands.

And then,

(2) Evaluating the connection design for the combinations of design force demands (N_u, V_u^{in}, M_u, and V_u^{out}) calculated from LEFE analysis of the CIS.

This design approach ensures that the connections are typically stronger than the weaker of the connected parts, and the structure will have ductile failure modes occurring in the SC walls (not the associated connections) governing the overall response if the SC wall becomes overloaded.

6.2 ASME Section III, Division 2 Design Requirements

As shown in Figure 7.2-2, the SC wall basemat anchorage connection crosses a code jurisdictional boundary. As a result, the basemat anchorage connection design must achieve both the strength requirements outline in previous section of this TeR, and the liner design requirements of ASME Section III Division 2 (Reference 6). Confirmation of the basemat anchorage design for both sets of requirements is accomplished in Step 9 of the overall full strength connection design procedure, in two basic steps; (i) confirm the design strength of the connection using the applicable provisions of ACI 349-06 and AISC N690-1994 as discussed in previous sections, and using demands from the ACI 349-06 load combinations as modified by United States Nuclear Regulatory Commission (NRC) Regulatory Guide 1.142 (Reference 15), and (ii) confirm that the baseplate and its anchorage meet the requirements presented in ASME

Section III Division 2 Article CC-3000, using demands determined in accordance with Article CC-3200. The second step involving evaluation of the connection for ASME requirements includes the following procedures:

1. Calculate demands using the load factors and load combinations defined in Article CC-3200 Table CC-3230-1.
2. Evaluate the baseplate, which also functions as part of the Prestressed Concrete Containment Vessel (PCCV) pressure boundary liner, per the liner stress and strain requirements of Article CC-3720.
3. Evaluate the rebar anchors, which also function as PCCV liner anchors, per the requirements of Article CC-3730.
4. Evaluate the baseplate and the welds between the SC faceplates and the baseplate for the additional requirements applied to brackets in Article CC-3750.

7.0 DESIGN EXAMPLE: SC WALL-TO-BASEMAT ANCHORAGE

The connection design approach described in Sections 1 – 6 has been used to design and detail the SC wall-to-basemat, SC wall-to-SC wall, and RC slab-to-SC wall connections in the US-APWR structure. The following discussion presents the application of the connection design approach to the development of typical SC wall-to-basemat connections (location and details shown in Figures 7.1-1 and 7.2-2 through 7.2-6).

Figure 7.1-1 Location of RWSP Wall Basemat Anchorage

7.1 Summary of Basemat Anchorage Connection Code Requirements

The SC wall to containment basemat anchorage is a full-strength connection that develops the expected yield strength of the SC wall in tension, the expected flexural capacity of the SC wall section, the expected out-of-plane shear strength of the SC wall, and the expected yield strength of the SC wall in in-plane shear. The intent of this full-strength connection design approach is the same as that of ACI 349-06, Appendix D.3.6, and ACI 349-06, Section 21.5.1.

As mentioned in Section 6.2 of this TeR, the SC wall to containment basemat connection penetrates the PCCV steel liner plate on the concrete basemat. ASME Section III, Division 2 governs PCCV liner plate design and all welding to the liner plate. This code does not have full-strength design requirements, but it does have unique requirements including:

- Liner stress and strain requirements for the calculated design demands as outlined in ASME Section III, Division 2, CC-3720.
- Liner anchor force and displacement requirements for calculated design demands as outlined in ASME Section III, Division 2, CC-3730.
- Requirements for material and quality control of welding to the liner plate in ASME Section III, Division 2, CC-2600 and CC-4500.

The SC wall to containment basemat connection is designed and detailed to meet the intent of both ACI 349-06 for full-strength connection design, and ASME Section III, Division 2 for strength, ductility, and quality of the steel liner plate and anchors for calculated design demands.

The connection is addressed in four parts: (i) the connection of the SC wall to the **[ ]**-thick steel liner plate on the concrete basemat, (ii) the liner plate itself, (iii) the connection between the **[ ]**-thick steel liner plate and the **[ ]** rebar-coupler anchors, and (iv) the design of the **[ ]** rebar anchors in the concrete basemat. Each part is designed for the full-strength connection demands, and then checked for the combined interaction of individual force demands from the applicable code as outlined in Section 6.0.

The design of part (i) is governed by:

a) AISC N690-1994 (including Supplement 2, 2004), Table Q.1.5.3 for the design of the welds (connectors) of the steel faceplate to the liner plate.
b) ACI 349-06, Appendix D.6 for the design of the shear studs (connectors) connecting the SC wall concrete to the liner plate.
c) All welding to the liner plate is in accordance with the material and quality requirements of ASME Section III, Division 2, CC-2600 and CC-4500.

d) The SC wall faceplates and the vertical shear studs that connect to the top of the baseplate (liner) are treated as “brackets and attachments” in accordance with ASME Section III, Division 2, CC-3650 and CC-3750. These provisions require liner attachments to be checked in accordance with AISC specifications (AISC N690-1994 is applicable in this case). Checks of the faceplates, the studs, and the faceplate/stud welds to the liner are performed with the design demands calculated from the load combinations in accordance with this ASME requirement. It is noted that since AISC N690-1994 does not contain specific provisions for checks of SC faceplate stresses, the provisions given for combined force design in Chapter 8 of TeR MUAP-11019 Rev. 1 are

used.

The design of part (ii) is in accordance with:

a) AISC N690-1994, Section Q.1.5.1.4.3 and Table Q.1.5.7.1 for evaluation of flexural stress in the liner plate due to full-strength demands. Note that ASME Section III, Division 2 CC-3750 permits the liner in the vicinity of attachments to be designed in accordance with AISC specifications.

b) ASME Section III, Division 2, Sub-Section CC-3720 for evaluation of liner steel plate stress and strains due to calculated design demands.

The design of part (iii) is in accordance with:

a) AISC N690-1994, Tables Q1.5.3 and Q.1.5.7.1 for the design of the welds between the #18 rebar couplers and the steel liner plate.

b) The material and weld quality requirements of ASME Section III, Division 2, CC-2600 and CC-4500 for all welding to the liner plate.

The design of part (iv) is in accordance with:

a) ACI 349-06, Appendix D Sections D.5, D.6, and D.7 for the design of **[ ]** rebar anchors embedded in the concrete basemat.

b) Liner anchor force and displacement provisions in AMSE Section III, Division 2, CC-3730 for evaluation of the liner anchors for calculated design demands.

7.2 Summary of Full Strength Connection Design Procedures

The following is a summary of the steps involved in the full strength design procedure outlined in Sections 1 - 6 of this TeR, as applied to the basemat anchorage connection. The evaluation of the connection for combined forces is illustrated in Step 9 for the rebar anchors (part “iv” above), to illustrate the application of both ACI 349-06 and ASME Section III Division 2 requirements. The approach for evaluating other components of the anchorage connection for combined forces is similar, and is presented in detail in the CIS basic design calculations.

Step 1) Design Demands: The anchorage design demands (N_u, V_u^{in}, M_u, and V_u^{out}) can be determined from the results of the LEFE analyses as outlined in Section 1.1. These design demands are not used directly in the design process until Step 9.

Step 2) Design Philosophy: The full strength connection design approach outlined in Section 2.0 is selected for the anchorage connection.

Step 3) Full Strength Connection Required Strengths: The anchorage connection required strengths (N_r, V_r^{in}, M_r, and V_r^{out}) for the individual demands are calculated as outlined in Section 3.

Figure 7.2-1 Stress Block Used to Compute M_r

<u>Step 4) Design of Connection Region:</u> The connection region is designed in accordance with Section 3.0. The area and spacing of tie bars are designed to provide out-of-plane shear strength greater than V_r^{out} as calculated above. This ensures the connection region is governed by ductile flexural yielding behavior rather than non-ductile out-of-plane shear behavior. **[**

]

As shown, the calculations above determine the design out-of-plane shear strength for the connection region. This value is then used to design the shear studs on the SC faceplates to provide interfacial shear strength greater than the out-of-plane shear strength of the section, in accordance with the procedures described in MUAP-11019 Rev. 1, Section 2.5. **[**

] The resulting arrangement of the rectangular plate tie bars and shear studs in the connection region is shown in Figure 7.2-6, and presented in greater detail in the CIS basic design calculations. It is noted that the tie bar and shear stud size and spacing selected for the connection region are applied to the full expanse of the SC wall.

Step 5) Force Transfer Mechanism for Tension: The force transfer mechanism for transferring axial tension from the steel faceplates to the base foundation concrete is shown in Figure 7.2-2. As shown, the force transfer is achieved using large diameter reinforcing bars that are attached to the thick steel baseplate using welded mechanical couplers. These reinforcing bars are extended sufficiently into the basemat to ensure full development of the bars. Furthermore the basemat is sufficiently reinforced to prevent a concrete breakout failure mode due to tension applied to the reinforcing bar group.

The steel plates of the SC walls are welded to the thick steel baseplate using complete joint penetration welds to ensure that the full tensile capacity of the plates can be transferred. The baseplate is designed with sufficient thickness to transfer the faceplate tensile force to the two anchor rods on either side of the faceplate (the center rod is provided for shear resistance only). The diameter, number, and spacing of the rebar anchors are determined to provide design axial tension strength greater than the required axial tension strength (N_r).

Figure 7.2-2 Anchorage Connection: Force Transfer Mechanism for Tension

Step 6) Force Transfer Mechanism for Bending Moment: The force transfer mechanism for transferring bending moment from the SC wall to the concrete basemat is similar to that shown in Figure 7.2-2, where the tensile force in the steel faceplate of the SC wall is transferred through the steel baseplate and rebar anchors into the concrete basemat. Flexural compression is transferred through direct bearing of the SC wall concrete on the steel baseplate.

Step 7) Force Transfer Mechanism for In-Plane Shear: The mechanism for transferring in-plane shear from the SC wall to the steel baseplate consists of: (i) welding the steel plates of the SC wall to the steel baseplate, and (ii) shear studs in the concrete infill of the SC wall.

The force transfer from the steel faceplates to the baseplate is readily achieved by welding. As shown in Figure 7.2-3, the force transfer from the concrete infill of the SC wall to the steel baseplate is achieved using shear studs that are designed according to the requirements of ACI 349-06 Appendix D Section D.6. These shear studs have longitudinal spacing (along the length of the wall) of **[ ]** , and load path shown in Figure 7.2-3.

Figure 7.2-3 Anchorage Connection: In-Plane Shear Load Path Through Shear Studs

Step 8) Force Transfer Mechanism for Out-of-Plane Shear: Two primary mechanisms exist to transfer out-of-plane shear from the wall to the baseplate consisting of: (i) a reduced number of effective studs attached to the top of the baseplate as shown in Figure 7.2-4 (for the **[ ]**-thick RWSP wall), and (ii) friction between the baseplate and the infill concrete in the equivalent flexural compression stress block resulting from out-of-plane moment. In accordance with ACI 349-06 Section D.6.1.4, the friction in the compression block is combined with the shear capacity of the studs on the top of the baseplate. Both of these shear transfer mechanisms listed above can develop the full strength design shear capacity. Furthermore, the conservatism of these out-of-plane force transfer mechanisms is confirmed by the 5:8 scale test of the basemat anchorage connection performed in Test Series 5.2 (see the Confirmatory Test Matrix presented in Appendix C for further discussion).

The force transfer mechanism for transferring out-of-plane shear from the steel baseplate to the concrete basemat is through direct shear on the rebar anchors. As described above for in-plane shear, the design for direct shear is performed according to ACI 349-06 Section D.6.1.

Figure 7.2-4 Anchorage Connection: Force Transfer Mechanism for Out-of-Plane Shear

Step 9a) Check Design for Combined Forces per ACI 349-06: The design force demands determined in Step 1 (N_u, V_u^{in}, M_u, and V_u^{out}) are used to check the connection design strength. Each individual demand is used to compute the *required strength* on the associated *connector*. The *total required strength* for each connector is determined by superposition of the required strengths from the individual demands.

The *connector design strength* is checked against the *total required strength* using the applicable code equations that account for superimposed force demands.

For the basemat anchorage example, this step is illustrated for the rebar anchors below. The design force demands are computed for the **[ ]**-thick RWSP wall, using an applicable seismic load combination (i.e. dead load + live load + fluid load + operating thermal + seismic). The demands are obtained from an LEFE analysis using appropriate ACI 349-06 load factors. The demands to be used for illustrating the interaction check are as follows:

These loads are applied to the section as shown in Figure 7.2-5 below.

Figure 7.2-5 Application of Forces to Rebar Anchors

Step 9a) Check Design for Combined Forces per ACI 349-06, continued.

The two anchors on the far left in Figure 7.2-5 experience the critical loading. The tension and shear interaction check is performed for these anchors as follows:

(The material used for the rebar is **[ ]**. The yield stress is **[ ]**. The ultimate strength is **[ ]** .)

Step 9b) Check Design for Combined Forces per ASME Section III Division 2:

The design force demands determined in Step 1 (N_u, V_u^{in}, M_u, and V_u^{out}) are used to check the connection design strength. Each individual demand is used to compute the *required strength* on the associated *connector*. The *total required strength* for each connector is determined by superposition of the required strengths from the individual demands.

The *connector design strength* is checked against the *total required strength* using the applicable code equations that account for superimposed force demands.

The basemat anchorage example below illustrates the combined force evaluation as presented in Section 6.0. The design force demands are computed for the **[ ]** -thick RWSP wall, using an applicable seismic extreme environmental load combination (i.e. dead load + live load + fluid load + operating thermal + seismic (safe shutdown)). The demands are obtained from an LEFE analysis using appropriate ASME Section III Division 2 load factors. The demands to be used for illustrating the interaction check are as follows:

These loads are applied to the section as shown in Figure 7.2-5.

The two anchors on the far left in Figure 7.2-5 experience the critical loading. The tension and shear interaction check is performed for these anchors as follows:

(The material used for the rebar is **[ ]**. The yield stress is **[ ]**. The ultimate strength is **[ ]**.) See the following page for the ASME Section III Division 2 calculations.

Step 9b) Check Design for Combined Forces per ASME Section III Division 2, continued.

Note that ASME Section III Division 2 Table CC-3730-1 does not specifically address interaction of tension and shear forces in liner anchors; it only provides allowable anchor stresses. Hence, ACI 349-06 Equation D-30 is used to evaluate the interaction while utilizing the ASME allowable stress.

Similar combined force design checks for the reinforcing bar anchors must also be made for all other applicable loading combinations. In addition, the mechanical couplers must be checked for each of the applied loading combinations, as well as the welds that attach the couplers to the wall baseplate. These checks are performed in detail in the CIS basic design calculations.

Figure 7.2-6 Overall Sketch of Basemat Anchorage Connection

8.0 REFERENCES

1. American Concrete Institute, "Code Requirements for Nuclear Safety Related Concrete Structures," ACI 349-06, November 2006.
2. American Institute of Steel Construction, "Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities," including Supplement 2 (2004), ANSI/AISC N690-1994, 1994, and 2004.
3. Mitsubishi Heavy Industries, Ltd., "Containment Internal Structure: Design Criteria for SC Walls," MUAP-11019, Revision 1, January 2013.
4. American Institute of Steel Construction, "Specification for Steel Buildings," ANSI/AISC 360-10, 2010.
5. American Society of Civil Engineers, "Seismic Rehabilitation of Existing Buildings," ASCE 41-06, 2007.
6. American Society of Mechanical Engineers, "Boiler and Pressure Vessel Code," Section III, Division 2, 2001.
7. Mitsubishi Heavy Industries, Ltd., "Research Achievements of SC Structure and Strength Evaluation of US-APWR SC Structure Based on 1/10th Scale Test Results," MUAP-11005, Revision 1, December 2012.
8. Mitsubishi Heavy Industries, Ltd., "Containment Internal Structure: Stiffness and Damping for Analysis," MUAP-11018, Revision 1, February 2013.
9. American Institute of Steel Construction, "Seismic Provisions for Structural Steel Buildings," AISC 341-10, 2010.
10. American Concrete Institute, "Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures," ACI 352R-02, 2002.
11. American Concrete Institute, "Building Code Requirements for Structural Concrete," ACI 318-05, 2005.
12. Takeuchi, Masayuki, et al., "Experimental Study on Steel Plate Reinforced Concrete Structure Part 28 Response of SC Members Subjected to Out-of-plane Load (1&2)," Papers 2619 and 2620, Summaries of Technical Papers of Annual Meeting, Architectural Institute of Japan, 1999.
13. Varma, A.H., Sener, K.C., Zhang, K., Coogler, K. and Malushte, S.R., "Out-of-Plane Shear Behavior of SC Composite Structures," Trans. of the Internal Assoc. for Struct. Mech. in Reactor Tech. Conf., SMiRT-21, Div-VI: Paper ID# 763, 6-11, New Delhi, India, November, 2011.
14. MacGregor, J.G., "Reinforced Concrete Mechanics and Design," pp. 188, 3rd Edition, 1997.
15. U.S. Nuclear Regulatory Commission, "Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments)," Regulatory Guide 1.142, Revision 2, November 2001.
16. Mitsubishi Heavy Industries, Ltd., "Containment Internal Structure Design and Validation Methodology," MUAP-11013, Revision 2, February 2013.

APPENDIX A:

SC Wall-to-Wall Joint Connection Design

Connection Design Summary – SC Wall-to-Wall Joint Connection

The determination of the required strengths and force transfer mechanisms for the "*T*" connection between SC walls is based on the full strength connection design philosophy. A typical "*T*" connection joining the adjacent East-West wall to the continuous North-South wall is shown in Figures A.1-1 & A.1-2. As previously derived in this TeR, the required strengths for this connection include out-of-plane shear (V_r^{out}), moment (M_r), tension (N_r) and in-plane shear (V_r^{in}).

The resulting design utility ratios for individual full strength demands as outlined in Sections 1 – 6 of this TeR are illustrated in Table A.1-1. The evaluation of all the connection components including design for combined forces is not presented here, but is presented in detail in the CIS basic design calculations.

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Figure A.1-1 Plan View Showing Area of Detail for SC Wall "*T*" Connection

Table A.1-1 SC Wall "*T*" Connection Utility Ratios

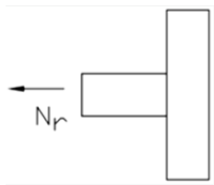
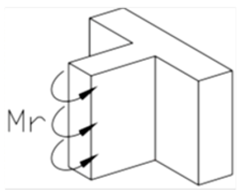
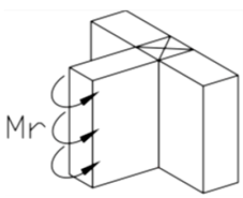
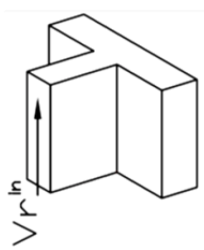
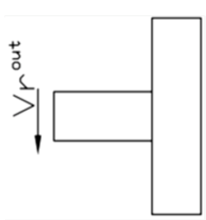
Design Step	Individual Full Strength Demand	Force Transferred	Utility Ratio	Comment
5		Axial Tension	[ ]	Controlling ratio based on shear studs in the connection. Note: N_r is limited by out of plane flexural capacity of the continuous wall.
6		Moment	[ ]	Controlling ratio based on shear studs in the connection.
Joint Shear		Moment	[ ]	Ratio based on joint shear capacity per ACI 349-06 Section 21.5.3.1
7		In-Plane Shear	[ ]	Controlling ratio based on shear studs in the connection.
8		Out-of-Plane Shear	[ ]	Controlling ratio based on shear studs in the connection.
9	Actual demands from LEFE Model	-	-	See CIS basic design calculations

Figure A.1-2 Overall Sketch of the SC Wall “*T*” Connection

APPENDIX B:

RC Slab-to-SC Wall Connection Design

Connection Design Summary – RC Slab-to-SC Wall Connection

The determination of the required strengths for the RC slab-to-SC wall connection is based on the full strength connection design philosophy. The location and conceptual details of a typical connection joining an RC slab to an SC wall is shown in Figures B.1-1 and B.1-2 below. The required individual strengths (V_r^{out}, M_r, N_r, and V_r^{in}) are based on ACI 349-06 with the applicable sections as follows:

- Tension (N_r) and Moment (M_r): ACI 349-06 Section 21.5.1.1.
- Out-of-Plane Shear (V_r^{out}): ACI 349-06 Section 21.3.4.1.
- In-plane Shear (V_r^{in}): ACI 349-06 Section 21.7.4.

The force transfer mechanisms selected to resist full strength RC slab demands are illustrated in Table B.1-1. The evaluation of all the connection components including design for combined forces is not presented here, but is presented in detail in the CIS basic design calculations.

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Figure B.1-1 Plan View Showing Area of Detail for RC Slab-to-SC Wall Connection

Table B.1-1 Force Transfer Mechanisms for Connection Between RC Slab and SC Wall

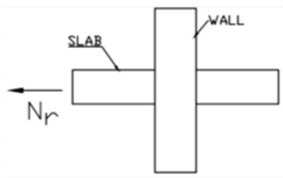
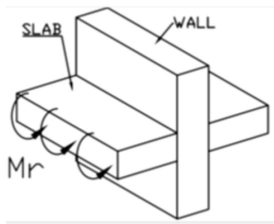
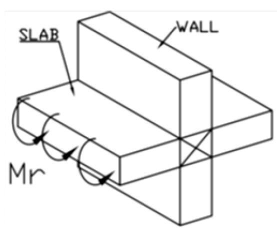
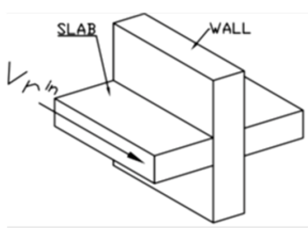
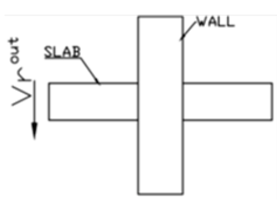
Design Step	Individual Full Strength Demand	Force Transferred	Force Transfer Mechanisms
5		Axial Tension	Axial tension is transferred from the slab into the SC wall via a direct load path consisting of rebar, rib plates, and tie bars, which can be seen in Figure B.1-2.
6		Moment	Tension resulting from moment is transferred from the slab into the SC wall via a direct load path consisting of rebar, rib plates, and tie bars, which can be seen in Figure B.1-2.
Joint Shear		Moment	Joint shear resulting from moment in the slab is resisted by concrete joint shear capacity according to ACI 349-06 Section 21.5.3.
7		In-Plane Shear	Studs within the slab and those directly located on the inside of the SC wall (shown in Figure B.1-2) provide a direct load path for in-plane shear.
8		Out-of-Plane Shear	Studs within the slab and those directly located on the inside of the SC wall (shown in Figure B.1-2) provide a direct load path for out-of-plane shear. Though not utilized, shear friction resulting from engaging slab rebar at the SC faceplate provides an additional, independent full strength force transfer mechanism.
9	Actual demands from LEFE Model		Combined interaction of applicable forces from analysis models using the forces transfer mechanisms presented in steps 5 through 8.

Figure B.1-2 Conceptual Detail of the RC Slab-to SC-Wall Connection

Note: The connection detailing shown is conceptual and for illustration purposes only.

APPENDIX C:

Confirmatory Test Matrix for SC Wall Design Strengths and Anchorage

Confirmatory Test Matrix

The confirmatory test matrix for SC walls of the US-APWR CIS is shown in Table C.1-1 on the following pages. The Series 4, 5, and 6 tests confirm the strength and ductility of the CIS connections. The Series 4 test confirms the applicability of the ACI 349-06 joint shear strength equations in Section 21.5.3 to SC wall-to-wall joints. The Series 5 and 6 tests confirm the in-plane and out-of-plane shear strength, behavior, and ductility of the SC wall basemat anchorage detail. A detailed summary of the confirmatory testing procedures and results is provided in TeR MUAP-11013 Rev. 2 Appendix B (Reference 16).

Table C.1-1 Confirmatory Test Matrix for SC Design Strengths and Anchorage

Table C.1-1 Confirmatory Test Matrix for SC Design Strengths and Anchorage (continued)