CONTAINMENT INTERNAL STRUCTURE: ANCHORAGE AND CONNECTION DESIGN AND DETAILING

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Mitsubishi Heavy Industries, Ltd.
16-5, Konan 2-chome, Minato-ku
Tokyo 108-8215 Japan

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DESIGN APPROACH AND EXECUTIVE SUMMARY

Scope

This Report presents design criteria and approach for connections involving steel-concrete (SC) walls only. Connections of reinforced concrete (RC) walls, slabs, and structures will be designed in accordance with the requirements of American Concrete Institute (ACI) 349-06 (Reference 3). Connections of steel members and structures will be designed in accordance with the requirements of AISC N690 (Reference 7).

Relationship with MUAP-11019 for SC Walls

This Report addresses SC walls in the US-APWR Concrete Internal Structure (CIS). The SC walls themselves are designed according to MUAP-11019 (Reference 1). As presented in MUAP-11019, the SC walls are detailed to prevent SC specific failure modes like local buckling, interfacial shear failure etc., and designed using ACI 349 code requirements for reinforced concrete structures with added conservatisms based on experimental results.

Connection Regions

As explained in MUAP-11019, the SC walls are differentiated into interior and connection regions. The connection regions are located at the supported or restrained edges, and are designed to be no longer than two times the wall thickness (2T) from the edge. 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 8). The interior regions of SC walls are designed according to MUAP-11019. The connection regions of SC walls are designed and detailed according to this Report.

Detailing of Connection Regions to Achieve Local Ductility

The connection regions of SC walls are detailed to prevent SC specific failure modes such local buckling, interfacial shear failure etc. They are also detailed to prevent non-ductile failure modes 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 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_n/2T).

Thus, all non-ductile failure modes are prevented from occurring in the connection region, and it has the ductility to undergo inelastic deformations and dissipate energy if needed for beyond design-basis events. This is similar to the intent of ACI 349-06, Section 21.3.3, which provides shear reinforcement detailing requirements to achieve ductile plastic hinge regions in flexural members of moment resisting frames.

Connection Design Philosophy to Achieve Global Ductility

Capacity design is a fundamental aspect of the seismic design philosophy for structures (ASCE 41, Reference 4). 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 earlier in MUAP-11019 and in this Report, both the interior regions and connection regions of the connected SC walls are detailed to prevent non-ductile failure modes. The major difference between their detailing is that the connection regions are detailed explicitly to prevent out-of-plane shear failure modes from governing 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.

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, shear lugs, welded rebar couplers, direct bearing, etc., and can be 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, ACI 349-06 or AISC N690.

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 (SC wall connection and interior regions) are detailed accordingly to have good ductility and prevent non-ductile failure modes (like out-of-plane shear) and SC specific failure modes.

This full strength connection design philosophy is similar to 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 strength of the connected part.

Overstrength Connection Design Philosophy

In some rare situations, it is not practical or feasible to provide a full strength connection. This will typically occur if the associated SC wall or RC slab is severely overdesigned with respect to the LEFE calculated force and moment demands due to shielding requirements. In such situations it is neither practical nor feasible to provide full strength connections. A more reasonable design philosophy would be 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 + 100% of non-seismic demands calculated from LEFE of the CIS for loads and load combinations. It distinguishes between the seismic and non-seismic demands due to potential variability and uncertainty in earthquake events (particularly beyond design-basis events) and the relative certainty of non-seismic (static, dynamic, thermal loading, etc.) loading.

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 being accomplished somewhat conservatively by increasing the SSE force and moment demands by a factor of 2.0. This increased factor (2.0 vs. 1.67) accounts 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.

This overstrength connection design philosophy is somewhat similar to the concrete anchorage design philosophy in ACI 349-06, Appendix D, Section D.3.6.3, which requires an overstrength design factor of 1.67 (i.e., 1 / 0.6) for anchorage design governed by non-ductile concrete failure modes that do not develop the full strength of the embedment.

The overstrength connection design philosophy in this report is more conservative than the requirements from ACI 349-06, Section D.3.6.3, and even the requirement for HCLPF of 1.67 SSE. It is important to note that the overstrength connection design philosophy will be used in rare situations, which will be clearly identified.

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

Anchorage Design: Example of Full Strength Connection Design

The report 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 strength is estimated using applicable material overstrength factors from ASCE 41, 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 involve connectors (welded couplers, shear lugs, etc.) that are well-established in practice and can be designed according to 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 linear elastic finite element (LEFE) of the CIS. This check involved determining the superimposed (concurrent) demands on the critical connectors, and checking their (connector) strength while accounting for interaction effects in accordance with applicable design codes.

Experimental Confirmation

As explained in the report, 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 standard and 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 the complete concrete internal structure (CIS) and the 1/6th scale test of the primary shield structure (MUAP-11005, Reference 9) provide valuable information regarding the overall behavior and strength of the US-APWR CIS structure. The design criteria and approach in this report 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 so because the design criteria and approach in this report are more comprehensive (develop all the individual strengths of connected walls) and robust (check for all concurrent design demands).

Finally, as part of this project, we propose to perform two confirmatory tests.

One test focuses on the in-plane shear strength and behavior of the SC wall-to-basemat anchorage. As shown in the report, the anchorage connection has been designed using clearly identifiable force transfer mechanism and standard connectors (shear lugs or direct shear on welded rebars) to develop the expected in-plane shear strength of the connected SC wall. As a result, there is a high level of confidence in the performance of the anchorage connection. A confirmatory test is proposed to showcase the in-plane shear performance of the connection, and to confirm the conservatism of the design approach.

The second test focuses on estimating the in-plane and out-of-pane shear force-slip stiffness of the SC wall-to-basemat anchorage. This stiffness may be needed in the LEFE model of the CIS for Loading Condition ‘B’ (MUAP-11018, Reference 2) involving accident thermal loads. Assuming the basemat anchorage to be rigid results in extremely large design demands at the base due to the unrealistic and mathematical restraint of thermal deformations. The measured shear force-slip stiffness will be used to confirm the spring stiffness provided in the LEFE model of the CIS to realistically model the physical restraint to the thermal deformations at the basemat.

Report Outline

- Section 1.0 presents the determination of the design force and moment demands for the SC wall connection or anchorage.
- 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 available 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.
- The figures from Section 7.0 are presented with more clarity (larger size etc.) in Appendix A.
- The confirmatory test matrix is shown in Appendix B.

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
CIS	Concrete Internal Structure
HCLPE	High Confidence of Low Probability of Failure
LEFE	Linear Elastic Finite Element
NRC	Nuclear Regulator Commission
RC	Reinforced Concrete
SC	Steel-Concrete
SSE	Safe-Shutdown Earthquake
T	Thickness

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1.0 CONNECTION DESIGN FORCE DEMANDS

As explained in MUAP-11019 (Reference 1), each steel-concrete (SC) wall is differentiated into *interior regions* and *connection regions* as shown in Figure 1.1. The *connection regions* are designed to be the transition or force transfer regions located close to each supported edge of an SC wall. The connection regions are designed to be no more than two times the wall thickness (2T) in length.

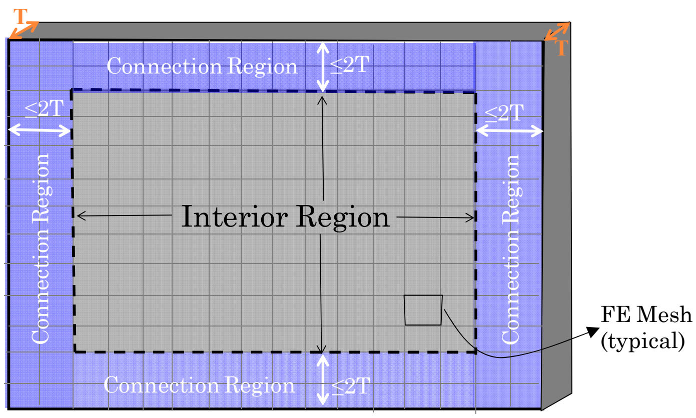

Figure 1.1 SC Wall Identifying Interior Region and Connection Region

The *connection design demands* for all loading combinations will be determined from the results of finite element analyses conducted in accordance with MUAP-11018 (Reference 2). As explained in MUAP-11018, 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 MUAP-11018 (Reference 2) Table 3-1.

The results from the finite element 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 1.2.

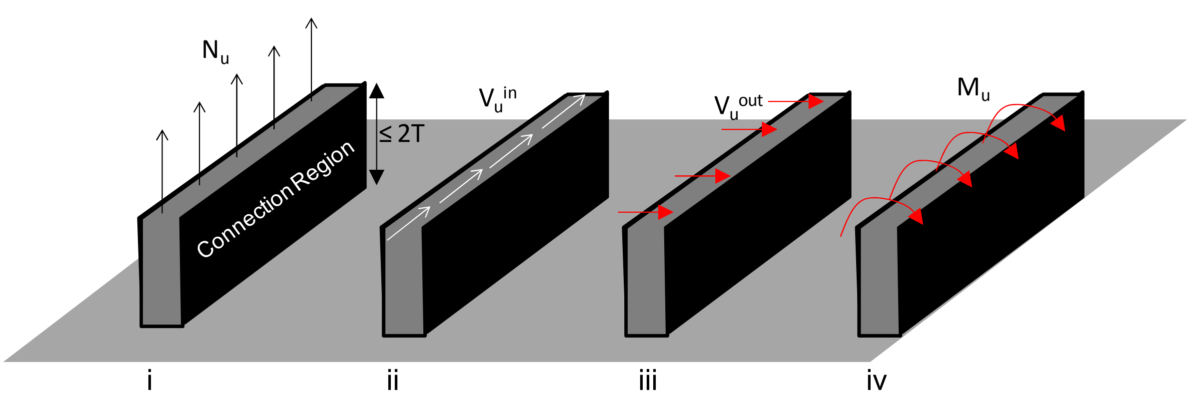

Figure 1.2 Connection Region Design Demands Per Unit Length

Each *connection design demand* ((i) – (iv) above) will be 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 will be considered:

1) Full strength connection design philosophy will develop the nominal 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 Reinforced Concrete (RC) slabs as the case may be). This ductile design approach is consistent with the concrete anchorage design provisions given in American Concrete Institute (ACI) 349-06 Appendix D (Reference 3).

2) Overstrength connection design philosophy will develop significant overstrength with respect to the design force demands on the connection. For each demand type, the connection will be designed for 200% of the seismic force demands plus 100% of the non-seismic force demands on the connection.

The full strength connection design philosophy will be used for almost all connection regions. The overstrength connection design philosophy will be used very sparingly, and only in circumstances where SC walls have been significantly overdesigned (by a factor of two or more) with respect to the design force demands. In such situations, it may not be practical to design full strength connections, and overstrength connections will be adequate. It is anticipated that the overstrength design approach will be necessary primarily for transfer of in-plane shear, given the excessive capacity of various walls that have been sized for shielding purposes rather than structural strength.

3.0 CONNECTION REQUIRED STRENGTHS

The *connection required strengths* will be calculated as described in Sections 1.0 and 2.0 of this Report. Most of the connections will be designed using the full strength connection design philosophy.

3.1 Full Strength Connection Required Strengths

For full strength connections, the required strengths for individual demands (i) – (iv) will be as follows:

(i) The required axial tension strength (N_r) per unit length of the connection will be at least 125% of the nominal yield strength of the weaker of the connected parts in tension. This is based on ACI 349-06 (Reference 3), Chapter 21.5.1. Compressive stresses are transferred through the connection via direct bearing of the concrete.

(ii) The required in-plane shear strength (V_r^{in}) of the connection will be at least 10 A_{cv} $(f'_c)^{0.5}$, which is the upper bound OF the in-plane shear strength of the SC wall based on ACI 349-06 (see MUAP-11019, Reference 1). A_{cv} is the area of concrete in in-plane shear. The concrete compressive strength f'_c will be assumed to be 150% of the nominal specified value (4000 psi for US-APWR) in accordance with American Society of Civil Engineers (ASCE) 41-06 recommendations (Reference 4, Chapter 6.2)

(iii) The required flexural strength (M_r) of the connection region will be at least the nominal flexural strength of the weaker of the connected parts in tension. This will be achieved by developing at least 125% of the nominal yield strength of the primary tension reinforcement (1.25 F_yA_s), which is also based on ACI 349-06 Chapter 21.5.1, and specified in (i) above.

(iv) The required out-of-plane shear strength (V_r^{out}) of the connection region will be 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.

This more conservative requirement (iv)(b), ensures that flexural yielding will occur before out-of-plane shear failure in the connection region for shear spans ratio greater than or equal to two. 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 code equations due to arch effects (Takeuchi et al. 1999, Reference 5). 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 6).

The requirement (iv)(b) eliminates this out-of-plane shear failure mode altogether for shear span ratios greater than or equal to two, and thus allows the SC wall to have ductile behavior in flexure. This is also the intent of the detailing requirements in ACI 349-06 Section 21.3.3 for flexural members of concrete moment fames, which also ensures that ductile failure modes like flexural yielding will govern over non-ductile failure modes like out-of-plane shear.

3.2 Overstrength Connection Required Strengths

For overstrength connections, the required strengths (N_r, V_r^{in}, V_r^{out}, and M_r) will be calculated as 200% of the seismic force demands plus 100% of the non-seismic force demands. The seismic force demands correspond to safe-shutdown earthquake (SSE), and they are increased to provide reserve margin and high confidence of low probability of failure (HCLPF) for 167% of SSE. ACI 349-06 requires design for 167% of the factored loads in the nonductile anchorage design provisions of Section D.3.6.3, wherein the nominal capacities are multiplied by a factor of 0.6 (i.e., 1/0.6 = 1.67). The design forces are increased to 200% instead of 167% to account for the fact that the ACI 349 code equations and resistance (ϕ) factors for strength are not based on HCLPF.

4.0 CONNECTION AVAILABLE STRENGTH

The *connection available strength* for each demand type (N_n, V_n^{in}, V_n^{out}, and M_n) will be calculated using the applicable *force transfer mechanism* identified in accordance with Section 4.1 and the *available strength* of its contributing *connectors* calculated in accordance with Section 5.0.

The *connection available strength* for each demand type will be demonstrated to be greater than or equal to the corresponding *connection required strength*.

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* will be 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 available strength* will be 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 will not be considered as a valid *connector* or *force transfer mechanism*.

5.0 DESIGN OF CONNECTORS

The force transfer mechanism for each demand type will be used to compute the *required strength* for its contributing *connectors*. The *available strength* for different connectors will be computed as follows:

1. For steel-headed stud anchors, the available strength shall be determined in accordance with ACI 349-06 Appendix D.
2. For welds and bolts, the available strength shall be determined in accordance with American Society of Civil Engineers (AISC) N690 (Reference 7) Section J.
3. For compression transfer via direct bearing in concrete, the available strength shall be determined in accordance with ACI 349-06 (Reference 3) Section 10.17.
4. For shear-friction load transfer mechanisms, the available strength shall be determined in accordance with ACI 349-06 Section 11.7.
5. For embedded shear lugs and shapes, the available strength shall be determined in accordance with ACI 349-06 Appendix D, Section D.11.
6. For rebar anchors and dowels, the available strength shall be determined from ACI 349-06 Section 15.8.

Connectors will be designed such that their *available strength* is greater than their *required strength*. In all cases the available strength will be calculated using the appropriate code specified strength reduction (ϕ) factors.

6.0 CONNECTION EVALUATION FOR COMBINED FORCES

The connection design performed in accordance with Sections 2, 3, and 4 of this Report will be further evaluated for the combined force demands calculated according to Section 1.0 (N_u, V_u^{in}, M_u, and V_u^{out}).

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

The *total required strength* will be compared with the *connector available strengths* (ϕR_n) calculated as described in Section 5.0 of this Report, while accounting for the effects of superposition of force demands as applicable. For example, steel headed stud anchors may be subjected to superposition of tension and shear forces, the interaction of which will be 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 design strengths (axial tension strength N_r, in-plane shear strength V_r^{in} etc.) of the weaker of the connected members.

And then,

(2) Evaluating the connection design for the combinations of design force demands (N_u, V_u^{in}, M_u, 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.

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-to-SC wall, and SC wall-to-RC floor connections in the US-APWR structure. The conceptual details for the SC wall-to-basemat connection are illustrated in Appendix A. The following discussion presents the application of the connection design approach to the development of the typical secondary SC wall-to-basemat connection (details shown in Appendix A). The location of the connection is shown in Figure 7.1.

Security-Related Information- Withheld Under 10 CFR 2.390

Figure 7.1 Secondary Shield Walls Basemat Anchorage

The anchorage design is conducted as follows:

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 finite element analyses as outlined in Section 1.0. These design demands are not used directly in the design process until much later in 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) 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 Stress Block Used to Compute M_r

Step 4) Design of Connection Region: The connection region (of length less than or equal to two times the section thickness T) will be designed in accordance with Section 3.0, which calls for detailing similar to ACI 349-06 Section 21.3.3, which is for the plastic hinge region of flexural members in moment frames. This detailing provides maximum tie bar spacing of 12 in.

Additionally, the area and spacing of tie bars will be designed to provide out-of-plane shear strength greater than the V_r^{out} calculated above. This will ensure the connection region is governed by ductile flexural yielding behavior rather than the non-ductile out-of-plane shear behavior. For example, the calculations shown below indicate that tie bar spacing of 10 in. in either direction is adequate for section detailing.

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 7.3. As shown, the force transfer is achieved using large diameter reinforcing bars that are attached to the thick steel base plate using welded mechanical couplers. These reinforcing bars are extended sufficiently into the basemat to ensure full development of the bars. Furthermore it is assumed that 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 also welded to the thick steel base plate as shown. Complete joint penetration welds are provided to ensure the full tensile capacity of the plates can be transferred. The base plate is designed with sufficient thickness to elastically 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 7.3 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 the one shown in Figure 7.3, where the tensile force in the steel faceplate of the SC wall will be transferred through the steel baseplate and welded rebars in the concrete basemat.

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 lugs 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 7.4, the force transfer from the concrete infill of the SC wall to the steel baseplate is achieved using shear lugs that are designed according to the requirements of ACI 349 Appendix D Section D.11. These shear lugs have longitudinal spacing (along the length of the wall) of approximately the wall thickness, and load path shown in Figure 7.5.

Figure 7.4 Anchorage Connection: Force Transfer Mechanism for In-Plane Shear

Figure 7.5 Anchorage Connection: In-Plane Shear Load Path Through Shear Lugs

The force transfer mechanism for transferring the in-plane shear from the steel baseplate to the concrete basemat is through direct shear on the rebar anchors attached to the steel baseplate using welded mechanical couplers. The mechanism of shear friction is not used, given the unlikelihood of the deformation required to mobilize shear friction in this assembly of large (#18) anchors welded to a thick baseplate. The design for shear is performed according to ACI 349-06 Section D6.1, which is used to determine or check the diameter, spacing, and number of rebars welded to the steel baseplate.

Step 8) Force Transfer Mechanism for Out-of-Plane Shear: The force transfer mechanism for transferring out-of-plane shear from the SC wall to the steel base plates consists of shear lugs in the concrete infill. As shown in Figure 7.6, the force transfer from the concrete infill of the SC wall to the steel baseplate is achieved by using shear lugs that are designed according to the requirements of ACI 349-06 Appendix D Section D.11.

There are two shear lugs in the SC wall thickness, and they are repeated at a longitudinal spacing of approximately the wall thickness. The far lug is assumed to resist all out-of-plane shear as shown in Figure 7.6.

Figure 7.6 Anchorage Connection: Force Transfer Mechanism for Out-of-Plane Shear

The force transfer mechanism for transferring out-of-plane shear from the steel baseplate to the concrete basemat is through direct shear on the welded rebar anchors, as shown in Figure 7.7. As described above for in-plane shear, the design for direct shear is performed according to ACI 349-06 Section D6.1.

Figure 7.7 Anchorage Connection: Out-of-Plane Shear Load Path

Step 9) Check Design for Combined Forces: 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 available 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 available strength* is checked against the *total required strength* using applicable ACI 349 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 48"-thick secondary shield wall shown in Figure 7.1, using an applicable seismic load combination (i.e. dead load + live load + fluid load + operating thermal + seismic). The demands are obtained from a linear elastic finite element analysis using appropriate 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 7.8 below.

Figure 7.8 Application of Forces to Rebar Anchors

The two anchors on the far left in Figure 7.8 experience the critical loading. The tension and shear interaction check is performed for these anchors as follows:

(It is assumed that the material used for the rebar is A706. The yield stress is 60 ksi. The ultimate strength is 80 ksi.)

Similar 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 conditions, as well as the welds that attach the couplers to the wall baseplate.

8.0 REFERENCES

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Appendix A: SC Wall to Basemat Connection Details

Security-Related Information- Withheld Under 10 CFR 2.390

Figure A1-1 Plan at Elevation 1'-11" Showing Area of Detail for SC Wall to Basemat Connection

Figure A1-2 Sectional View of Basemat Connection Showing Shear Lugs and Wall Reinforcement

Figure A1-3 Sectional Plan View of Connection Showing Shear Lug Arrangement

Figure A1-4 Force Transfer Mechanism for Vertical Axial Tension

Figure A1-5 Mechanisms for Transferring Transverse and In-Plane Shear to Base Plate

Figure A1-6 Direct Shear Mechanism for Transferring Shear from Base Plate to Concrete Basemat

Appendix B: Confirmatory Test Matrix for SC Basemat Anchorage

Table B-1 Confirmatory Test Matrix for SC Anchorage