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Design Methodology for Standard Steel-Composite (SC) Wall Attachments for Nuclear Power Plants

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ABSTRACT

Use of Steel-Composite (SC) walls is an efficient option to modularize construction and accommodate the large amount of commodity supports present in nuclear power plants. SC wall strength is derived from two faceplates (FPs) composite with structural concrete using steel anchors and connected by ties. The SC wall FPs can directly support a variety of attachments such as commodity supports or structural beams. Since the FPs act as the flexural reinforcement for the wall, SC walls present a unique situation where concentrated loads will be directly attached to the flexural reinforcement. This paper discusses an analysis and design procedure for qualifying local demand from SC wall attachments, while meeting current ANSI/AISC N690-18 (2018) design and detailing provisions for SC walls. Appendix N9 of ANSI/AISC N690-18 (2018) addresses the requirements for SC walls in safety-related structures for nuclear facilities. While Appendix N9 of ANSI/AISC N690-18 (2018) addresses the global design and analysis requirements for SC walls, Appendix N9 does not provide guidance for qualifying localized demand from attachments to SC wall FPs. Accounting for additional demand from individual local attachments is impractical for large scale, finite element building analysis models, due to the large number of attachments, relatively coarse finite element (FE) mesh and lengthy analysis runtimes. Alternatively, a portion of the available SC wall design strength can be allocated for attachments by reducing the available global strength in the SC wall components (e.g. FP, anchor, tie, etc.) by a predetermined amount. A local evaluation of a panel section with an attachment can then be performed using finite element analysis to determine the maximum attachment reaction that utilizes the allocated design strength. This paper discusses an analysis and design methodology that satisfies the intent of ANSI/AISC N690-18 (2018) for the design of SC walls with FP welded attachments while providing opportunities for improved economy.

CURRENT CODE REQUIREMENTS

Appendix N9 of ANSI/AISC N690-18 (2018) specifies that the required strength for SC walls should be determined from FE analysis. Commentary of ANSI/AISC N690-18 (2018), Section CN9.2.1, indicates that FEs larger than two times the SC section thickness (t_{sc}) are not recommended for interior regions and t_{sc} for connection regions or section penetrations. ANSI/AISC N690-18 (2018) Appendix N9.2.5 stipulates, "the required strength for each demand type shall be calculated by averaging the demand over panel sections that are no larger than twice the section thickness in length and width. In the vicinity of openings and penetrations, and in connection regions, the required strength shall be calculated by averaging the demand over panel sections no larger than the section thickness in length and width." For expansive SC walls that use FE sizes equal to the SC section thickness, this is reasonable for capturing global building behavior. However, for modelling of the effects of local attachments on SC walls, the element sizes are too large to accurately capture the local stress distribution. Alternatively, element sizes can be reduced to accurately capture localized effects from attachments. However, for large scale structural analysis models, this is impractical as there are typically a large number of attachments and will result in prohibitive analysis times. Furthermore, changes to any number of parameters such as attachment location, loading, and size may invalidate a structures' global finite analysis model and result in numerous iterations.

During design development of a new nuclear facility, the global structural analysis precedes the commodities routing and support design. Inherently, this approach requires local attachment evaluations utilize only the margin remaining in a structural element based on the global analysis. In cases where structural element utilization ratios are high, limitations are places on attachment locations, or additional detailing or reinforcement of the primary structure is needed to accommodate attachment loading. Neither of these scenarios is desirable, or efficient, when considering the thousands of attachments to structural walls in a new nuclear facility. To that end, decoupling the global structural analysis and local attachment analysis is an efficient methodology to progress both aspects of the overall SC wall design in parallel.

ANALYSIS PROCEDURE TO MEET CODE REQUIREMENTS

As an alternative approach to reducing the mesh size of the global building model, a portion of the available SC wall design strength can be allocated for attachments by reducing the available global strength in the SC wall components (e.g. FP, anchor, tie, etc.) by a predetermined amount. For example, if 10% of the SC wall strength is allotted for direct FP attachments, then a factor of 0.9 is applied to SC wall component strength (similar to a strength reduction factor) when qualifying the SC wall for global building loads. This methodology utilizes 10% of the SC wall design strength for attachments as a representative benchmark. Users of this approach must determine a strength allocation suitable for their specific circumstances, ensuring an optimal fit for each project.

The SC wall derives its strength from the composite action between the FP, ties, anchors, and concrete infill. To determine the maximum allowable load on the SC wall, the design strength for the section is first calculated by determining discrete component strength of the FP, ties, anchors, and concrete that provide composite action. The discrete component strength for the global SC wall is calculated in accordance with ANSI/AISC N690-18 (2018) and ACI 349-13 (2013), where applicable. Forces and stresses in the FP, ties, anchors, and concrete must remain within 90% of their design strength in the global building analysis to align with the strength allocation noted previously for local attachments.

A local evaluation of a panel section with an attachment can then be performed using FE analysis to determine the maximum attachment reaction that utilizes the allocated design strength. A single SC wall section FE used in a global facility analysis (with size equal to the SC wall section thickness) is modeled with smaller FEs so the local behavior from an attachment can be captured. For the evaluation presented herein, FE with size approximately equal to the FP thickness (t_p) is deemed appropriate. Furthermore, discrete elements of the SC wall can be modeled (e.g. anchors, ties, FP, and concrete) so localized forces and stresses in individual elements can be determined. See Figure 1 for an illustration.

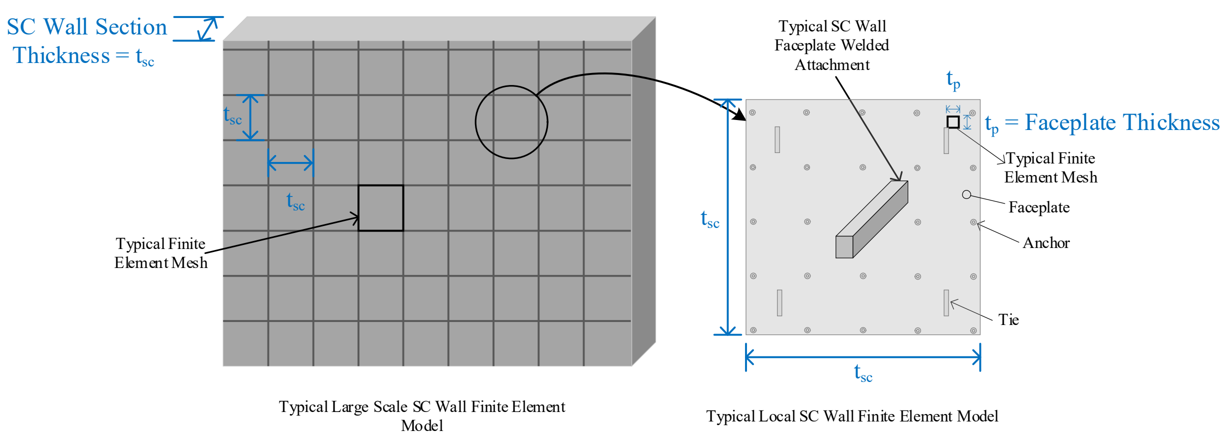

Figure 1. Typical Large-Scale FE of SC Wall versus Local FE Model.

An attachment welded to an SC wall FP can be modeled similar to an embedded plate. The SC wall FP transfers loading from an attachment to connected anchors in shear and tension and to the concrete body in bearing. For SC walls, the FP acts as a continuous embedded plate. The anchors used in SC walls function similarly to those in embedded plates and ties can be considered as tension only members.

To reconcile the difference in the modelling size and element size used in the building analysis and this local analysis, loads transmitted from attachments are limited in accordance with two different acceptance criteria simultaneously to satisfy the provisions and intent of ANSI/AISC N690 (2018).

The first acceptance criteria, herein referred to as global strength criteria, are loads transmitted from attachments that are limited to the predetermined percentage of the global SC wall design strength. In this case, 10%. Because multiple discrete components are modeled, components' forces and stresses are averaged over a length and width equal to the SC wall section thickness. As discussed previously, this is consistent with ANSI/AISC N690-18 (2018) Appendix N9.2.5 guidance for element averaging. By limiting the FE model size in the local model to a typical FE in a large-scale building analysis (say length and width equal to the SC wall section thickness), averaging is performed over the same length of SC wall regardless of the smaller FE size in this calculation. As a result, forces and stresses averaged over the analysis model represent the same demand as the large-scale building analysis.

The second acceptance criteria, herein referred to as local strength criteria, are loads transmitted from attachments which may utilize a substantial portion of the SC wall component design strength. This is applicable for each individual anchor and tie, and a localized area for the FP, concrete infill, and spreader plate (where applicable). This differs from the first acceptance criteria in that individual discrete components are evaluated rather than all the components averaged within the section. Each individual tie is limited to 90% of its design strength in this evaluation. The FP and concrete bearing are limited to 90% of their design strength over a localized area. Each anchor is limited to 20% of its design strength. Methodology detailing why 20% is appropriate is discussed in a later section. Because FE sizes in this analysis are much smaller in comparison to a large-scale SC wall analysis, local stress concentrations that would otherwise not be captured are present.

ANSI/AISC N690-18 (2018) Appendix N9.2.5 specifies that SC wall demand should be determined by averaging over one or two times the SC wall thickness. For an FE size equal to the SC wall thickness as evaluated in a large-scale SC wall analysis, stresses are implicitly averaged within the section and therefore, its discrete components. Localized stresses/forces that are close to the design strength would still be acceptable for attachment loads as the average stress over the entire element (section thickness in length/width) will remain within the 10% allotted design strength in this example. Areas of localized higher stresses would cause strains that would in turn redistribute and alleviate stresses. The redistribution length would still be on a smaller magnitude in comparison to the element size utilized in the global SC wall analysis. By limiting forces and stresses to the two acceptance criteria described, stresses stay within the predetermined design strength.

GLOBAL STRENGTH CRITERIA

Faceplate

ANSI/AISC N690-18 (2018) Appendix N9.3.1 through Appendix N9.3.4 specify the strength of the SC wall section as a function of the FP per unit length. To evaluate the global strength in the FP, horizontal and vertical section cuts are made through the FP at the attachment center and attachment outer limits. When a spreader plate is used, additional section cuts at the plate edges are also evaluated. The section cut is one section thickness in length and is centered on the attachment. The forces are summed over the section and compared to 10% of the FP strength over the section thickness in this evaluation. This evaluates the membrane forces which is appropriate for global strength evaluation as ANSI/AISC N690-18 (2018) Appendix N9.3 considers the FP as tension reinforcement for SC wall axial tension and flexure. Additionally, the element Von Mises stress is averaged for the elements along the section cut and compared to 10% of the FP strength. This combines the bending and membrane stresses along the section and is prudent as local attachments cause local bending stresses that are otherwise not captured in a large-scale SC wall analysis.

The FP strength as a stress, ΦF_{FP}, is determined in accordance with Equation 1. In Equation 1, F_y, is the yield strength of the FP. And Φ_b is the resistance factor taken as 0.9. The 0.9 resistance factor is in conformance with tensile, in-plane shear and out-of-plane flexural resistance factors for SC walls as prescribed in AISC N690 Appendix N9.3.1, N9.3.3 and N9.3.4. Additionally, it is consistent with the resistance factor for plate bending in ANSI/AISC 360-16 (2016) and ACI 349 (2013) Appendix D4.4.

$$\Phi F_{FP} = \Phi_b F_y \tag{1}$$

The maximum allowable stress/forces from an attachment reaction in the FP section cuts and elements, $\Phi F_{FP.glb}$, is shown in Equation 2. In Equation 2, $\Phi_{att.glb}$ corresponds to 0.1 (10% is allocated for attachments) while ΦF_{FP} is the FP strength.

$$\Phi F_{FP.glb} = \Phi_{att.glb} \Phi F_{FP} \tag{2}$$

Anchors

The sum of forces in all the anchors considered engaged in a group shall be limited to 10% of the anchor group strength. The only applicable limit states considered in shear are steel anchor yielding and concrete pryout. In tension, the steel anchor yielding, concrete breakout, and concrete pullout are applicable. Concrete side-face blowout strength of anchor in tension and concrete breakout in of anchors in shear are not applicable failure modes as the SC walls have continuous FPs that surround and confine the concrete infill on all faces. Anchors considered in this methodology considers only shear yielding steel anchors per ANSI/AISC N690 (2018) Appendix N9.1.4a. Determination of anchor shear strength is in accordance with ACI 349-13 (2013) Appendix D. This is appropriate as Appendix N9.4.3a permits ACI 349-13 (2013) Appendix D to be used in accordance with ANSI/AISC N690 (2018) Section I8.3 for anchor strength and is represented as ΦV_{ng} in Equation 3.

For anchors in tension, ANSI/AISC N690 (2018) does not consider their tensile strength for the SC wall design strength. However, for small attachments on FPs, out-of-plane loads on the FP would pull the FP out from the concrete infill through tension in the anchors. Therefore, it is prudent to limit the tension in the anchors. The anchor tensile strength is evaluated in accordance with ACI 349-13 (2013) Appendix D similar to the anchor shear strength and is ΦN_{ng} in Equation 4. In Equation 3 and 4 respectively, $\Phi V_{ng.glb}$ and $\Phi N_{ng.glb}$ are the maximum shear and tensile force in the anchor group allowed from an attachment reaction.

$$\Phi V_{ng.glb} = \Phi_{att.glb} \Phi V_{ng} \tag{3}$$

$$\Phi N_{ng.glb} = \Phi_{att.glb} \Phi N_{ng} \tag{4}$$

Ties

In engaged ties, the sum of all tie forces shall be less than or equal to 10%/2 of the sum of the tie strength within the section length. The forces in the ties are limited to1/2 of the 10% allocated SC wall strength to account for attachments being placed on both sides of the same SC wall. This is only applicable to the ties as they are the only element that span between opposite FPs. The anchor embedment is typically not deep enough to reach half the wall thickness as to engage the concrete infill on the opposite half of the SC wall. The ties provide shear reinforcement for out-of-plane shear strength in SC walls in accordance with ANSI/AISC N690 (2018) Appendix N9.3.5a as a function of the tie tensile strength. The contribution of ties to the out-of-plane shear strength is equated in ANSI/AISC N690 (2018) Equation A-N9-23 and is used in this methodology as the tie strength, ΦF_{tie}, see Equation 5 for its derivation. In alignment with ANSI/AISC N690 (2018), this methodology considers the ties as tension members.

$$\Phi F_{tie} = \Phi_{vo} \xi \frac{t_c}{s_{tl}} A_t f_y \frac{l}{s_{tt}} \leq \Phi_{vo} 0.25 {f'_c}^{0.5} t_c l \tag{5}$$

In Equation 5, Φ_{vo} is the resistance factor for SC wall out-of-plane shear strength (0.75) per ANSI/AISC N690 (2018) Appendix N9.3.5a, ξ is taken as 1.0 for a yielding shear reinforcement per ANSI/AISC N690 (2018) Appendix N9.3.5a, t_c as the concrete infill thickness, A_t is the cross-sectional area of the tie, f_y is the yield strength of the tie, s_{tl} is the spacing of the ties along the direction of one-way shear, s_{tt} is the spacing of the tie transverse to the direction of one-way shear, f'_c is the concrete compressive strength, and l is the unit width.

The maximum allowable tensile force in the tie due to an attachment reaction is $\Phi F_{tie.glb}$, and is shown in Equation 6. In Equation 6, $\Phi_{att.glb}$ corresponds to 0.1 (10% is allocated for attachments), n_{tie} is the number of ties within a modeled section, and ΦF_{tie} is the tie strength.

$$\Phi F_{tie.glb} = \Phi_{att.glb} 0.5 \Phi F_{tie} n_{tie} \tag{6}$$

Concrete Infill

The concrete infill is considered as a bearing only, compression surface. For a localized model, this is appropriate as the force normal to the plane of the SC wall would be transmitted as compression on the FP and thereafter concrete surface similar to an embedded plate. The average bearing stress within the averaging section is limited to 10% of the bearing strength. The bearing strength of the concrete infill is not specified in ANSI/AISC N690 (2018) but alternatively evaluated in accordance with ACI 349-13 (2013) Section 10.14 and provided in Equation 7 as a permissible stress, ΦB_{conc}. Ignoring the contribution of the concrete infill for the SC wall out-of-plane shear strength is conservative and simplifies modeling parameters. In Equation 7, Φ_{bear} is the strength reduction factor for concrete bearing taken as 0.65 in accordance with ACI 349 (2013) Section 9.3.2.4 and f'_c is the compressive strength of the concrete infill.

$$\Phi B_{conc} = \Phi_{bear} 0.85 f'_c \tag{7}$$

The maximum allowable bearing stress for the concrete infill due to an attachment reaction is shown in Equation 8 ($\Phi B_{conc.glb}$). In Equation 8, Φ_{att} corresponds to 0.1 (10% is allocated for attachments) while ΦB_{conc} is the concrete strength.

$$\Phi B_{conc.glb} = \Phi_{att.glb} \Phi B_{conc} \tag{8}$$

LOCAL STRENGTH CRITERIA

Ties

Each individual tie shall be limited to 45% of its respective design strength. The 45% is equal to 1/2 of the maximum 90% local strength to account for attachments being placed on both sides of the same SC wall. The design strength of individual ties is evaluated the same as ties in the global strength criteria, in accordance with ANSI/AISC N690 (2018) equation A-N9-23. Therefore, the maximum allowable force in any individual tie due to an attachment reaction is $\Phi F_{tie.loc}$ and provided in Equation 9.

$$\Phi F_{tie.loc} = \frac{0.9}{2} \Phi F_{tie} \tag{9}$$

Anchors

As previously stated, this evaluation only considers yielding steel anchors per ANSI/AISC N690 (2018) Appendix N9.1.4a. This is for composite action and preventing interfacial shear failure before out-of-plane shear failure of the SC section. In shear, each individual anchor force shall be limited to 20% of its design strength. As a yielding steel anchor, deformation is allowed to redistribute forces. For a SC wall stressed to 90% of its design strength, any single anchor can achieve 110% its design strength (90% + 20% = 110%) from a local attachment. However, this is acceptable if the individual anchor is governed by a ductile failure and as a result, yielding would occur before local concrete failure. Local redistribution is not affected and the global design strength on the SC wall is not adversely affected as the global capacity of all the anchors is also limited to 10% of the group anchor strength. This is achieved by specifying anchor material, size, and spacing criteria such that yielding failure modes control.

In tension, individual anchors shall be limited to 20% of their design strength. As previously mentioned, ANSI/AISC N690 (2018) only considers the steel anchor shear strength and does not consider the anchor tensile strength. However, concrete failure still needs to be prevented for adequate distribution of forces. Globally, the tensile group forces are limited to 10% of the group anchor strength. Individually, limiting anchor forces to 20% of the design strength is acceptable as shear and tension interaction are not checked until each is greater than 0.2 per ACI 349-13 (2013) Appendix D.7. As a result, local concrete failure does not occur. The shear and tensile design strength of individual anchors is evaluated the same as the global strength criteria, in accordance with ACI 349-13 (2013) and are included in Equation 10 and Equation 11 as ΦV_n and ΦN_n, respectively. Therefore, the maximum allowable shear and tensile force for any individual anchor due to an attachment reaction is provide in Equation 10 ($\Phi V_{ng.loc}$) and Equation 11 ($\Phi N_{ng.loc}$), respectively.

$$\Phi V_{ng.loc} = 0.2 \Phi V_n \tag{10}$$

$$\Phi N_{ng.loc} = 0.2 \Phi N_n \tag{11}$$

Faceplate and Concrete Infill

In the FP and concrete infill, average stress shall be limited to 90% of their respective design strength locally averaged over a small area equal to two times the FE element size in width and length. For FEs sized equal to the FP thickness, this is appropriate to remove local maxima. In the FP, the Von Mises stress criteria is considered. This accounts for localized membrane and bending stresses. For the concrete infill, the bearing pressure is considered. The design strength of the FP and concrete is evaluated the same as the global strength criteria. Therefore, for the maximum allowable FP and concrete bearing stress due to an attachment reaction is provided in Equation 12 ($\Phi F_{FP.loc}$) and Equation 13 ($\Phi B_{conc.loc}$), respectively.

$$\Phi F_{FP.loc} = 0.9\Phi F_{FP} \tag{12}$$

$$\Phi B_{conc.loc} = 0.9\Phi B_{conc} \tag{13}$$

SC WALL FINITE ELEMENT MODELING METHODOLOGY

The local SC wall model representing one finite element from the large-scale SC wall analysis can be modeled in a Finite Element Analysis (FEA) program such as ANSYS APDL. Due to the attachments being limited to a portion of the SC wall design strength below the yield limit, linear elastic FE elements can be used. The FP is modeled as a shell element with mesh size approximately equal to the FP thickness. Meanwhile, the concrete infill is represented using compression-only springs. Anchors, on the other hand, are modeled as uniaxial link elements. For each anchor, three link elements are positioned along the axial and shear directions, allowing for control over their respective stiffness properties. Ties are modeled as axial-only link elements. This choice is made for modelling convenience, as it simplifies the analysis. Additionally, ties are considered as axial members contributing to the SC wall's design strength. Connection of the ties and anchors to the FP are with a bonded connection at their interface. The boundary condition for the FP is from the anchors, concrete infill elements, and along the FP edges. The anchors restrain the FP as spring elements. The concrete infill only supports compression bearing. Along the FP edges, displacements are free. However, rotation about the axis parallel to each edge is restrained. These boundary conditions are conservative as all loading is supported within the modeled area. See Figure 2 for typical FEA model.

The attachment profile is modeled with solid, linear elastic, isotropic FEs. The nodes at the interface corresponding to the weld profile are bonded to the FP. The interface between the attachment profile face and FP are not connected as a bearing surface conservatively to ensure all load transfer is through the weld nodes. Load application on the attachment is through a mater node at the geometric centroid of the cross-section. To apply loading, a master node is created at the end of the attachment, at the geometric centroid of the cross-section, see Figure 3. The master node controls all the nodes at the attachment face. All displacement and rotational degrees of freedom are controlled by the master node and all loads are applied to the master node.

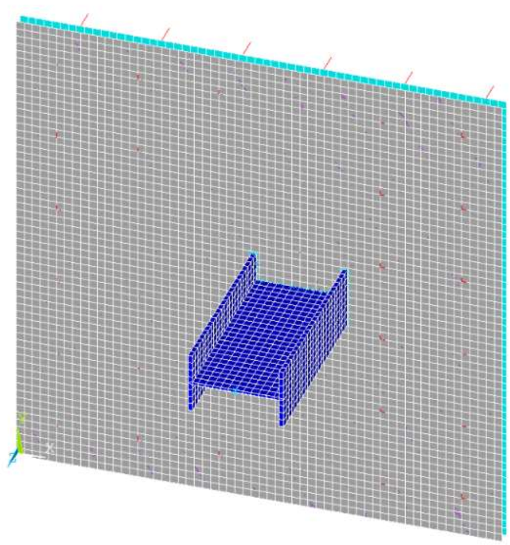

Figure 2. Sample FEA Model.

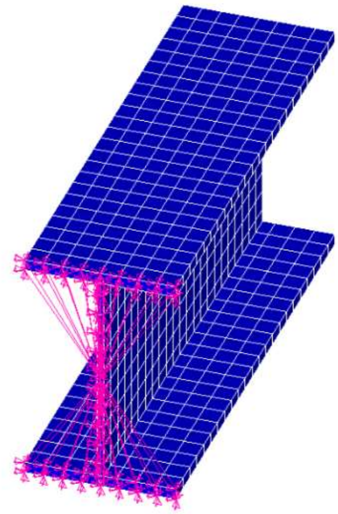

Figure 3. Attachment Master Node Constraint for Load Application.

Parametric Study Procedure

Attachments welded on FPs can theoretically be placed at any location on an SC wall. The location of the attachment plays a critical role in the forces and stresses the SC wall FP, anchors, ties, and concrete infill experience. To determine the maximum load an attachment can transmit to an SC wall, a parametric evaluation can be performed by first determining the number of analysis configurations that envelop the desired placement criteria considering, attachment profiles, attachment spacing, material specifications, connection detail, SC wall component sizes, location, tolerances, etc…Generally, the less limitations on placement results in increased analysis configurations required for a given attachment profile.

Next, for each attachment, analyze the set of controlling analysis configurations to determine a set of capacities. Analysis program ANSYS APDL was used to develop this methodology. ANSYS APDL can be developed to automate modelling, analysis, and post-processing. This substantially decreases interfacing time which reduces the likelihood of human error due to repetitive tasks while obtaining desired results. As a result, this increases efficiency and cost-savings.

Analysis of parameters and cases used to develop this methodology demonstrates that the maximum load that an attachment can transmit to an SC wall FP is governed by the anchor strength. Specifically, the allotted 10% global strength of the group anchor strength controls the capacity. This is expected as the group anchor strength is typically controlled by concrete failure that has strength that is much lower than steel failures such as FP and tie yielding. Considering a different set of predetermined SC wall strength allocation and SC wall parameters, the results and trends may vary.

EVALUATION OF PROSPECTIVE ATTACHMENTS

With a set of capacities for an attachment determined using this methodology, engineers placing attachments on SC walls can easily check that their attachment is within the permissible limits of the SC wall by inputting their actual reactions into an interaction equation. As mentioned previously, analysis results used to develop this methodology show that the anchors control the set of capacities for each attachment.

The interaction equations to check prospective attachments are provided in Equation 14, 15, and 16 below and are based on the interaction equations in ACI 349-13 (2013) Appendix D.7 for embedment plate anchors. Rather than the forces and strength of the anchors, the variables in this methodology are the capacities derived from this analysis and the reactions from the perspective attachment. This is appropriate as the capacities derived in this analysis method are based on the strength the anchors. The applicable reactions that can be considered are from load and resistance factor design (LRFD) combinations in alignment with the strength criteria that is calculated per LRFD methodology.

$$\frac{\mathrm{T}}{\mathrm{T_s}} + \frac{\mathrm{M_x}}{\mathrm{M_{xs}}} + \frac{\mathrm{M_y}}{\mathrm{M_{ys}}} \leq 1.0 \ \text{ if } \ \frac{\mathrm{V_x}}{\mathrm{V_{xs}}} + \frac{\mathrm{V_y}}{\mathrm{V_{ys}}} + \frac{\mathrm{M_z}}{\mathrm{M_{zs}}} \leq 0.2 \tag{14}$$

$$\sqrt{\frac{V_x}{V_{xs}} + \frac{V_y}{V_{ys}}} + \frac{\mathrm{M_z}}{\mathrm{M_{zs}}} \leq 1.0 \ \text{ if } \ \frac{\mathrm{T}}{\mathrm{T_s}} + \frac{\mathrm{M_x}}{\mathrm{M_{xs}}} + \frac{\mathrm{M_y}}{\mathrm{M_{ys}}} \leq 0.2 \tag{15}$$

$$\text{Otherwise, } \frac{\left[\left(\frac{\mathrm{T}}{\mathrm{T_s}} + \frac{\mathrm{M_x}}{\mathrm{M_{xs}}} + \frac{\mathrm{M_y}}{\mathrm{M_{ys}}}\right) + \left(\sqrt{\frac{V_x}{V_{xs}} + \frac{V_y}{V_{ys}}} + \frac{\mathrm{M_z}}{\mathrm{M_{zs}}}\right)\right]}{1.2} \leq 1 \tag{16}$$

Where the numerator terms (V_x, V_y, T, $\mathrm{M_x}$, $\mathrm{M_y}$, and $\mathrm{M_z}$) are the actual reactions from the prospective attachment with V_x, and V_y being the shear reactions in the plane of the SC wall, T as the axial reaction normal to the plane of the SC wall and $\mathrm{M_x}$, $\mathrm{M_y}$, and $\mathrm{M_z}$ as the bending about the in-plane and normal axis of the SC wall. See Figure 4 for illustration.

The denominator terms ($\mathrm{V_{xs}}$, $\mathrm{V_{ys}}$, $\mathrm{T_s}$, $\mathrm{M_{xs}}$, $\mathrm{M_{ys}}$, and $\mathrm{M_{zs}}$) are the capacities determined from this methodology corresponding to the same force and moments as the numerator terms. The numerator and denominator terms are scalar values with positive/negative not included. If prospective attachment reactions result in equalities less than 1.0 per Equation 14, 15, and 16, then the attachment is considered acceptable to be welded to the SC wall FP. No further evaluation of the SC wall or attachment is required, otherwise, a unique evaluation will be required.

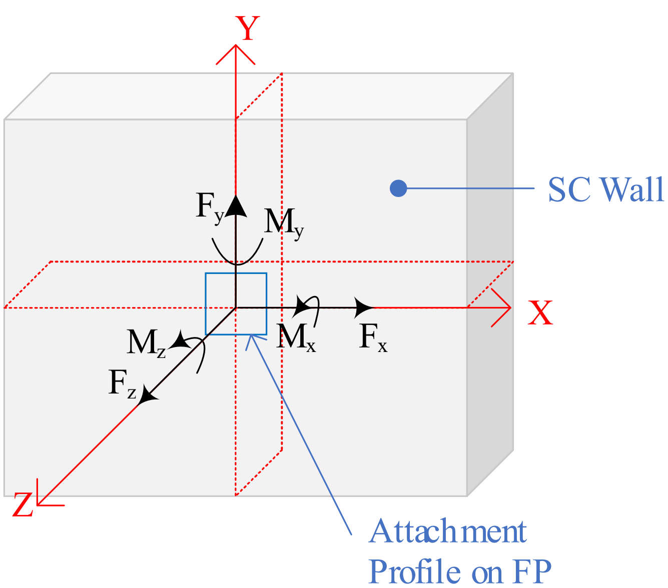

Figure 4. Loading Diagram for Typical Attachment.

CONCLUSION

Current AISC N690-18 (2018) code provisions do not provide guidance on how to efficiently qualify SC wall attachments. This paper provides a methodology for incorporating local design demand from attachments into the framework of the global SC wall analysis. FE analysis techniques and determination of allowable capacities meeting the intent and requirements of ANSI/AISC N690-18 (2018) are presented by reserving capacity in the global SC wall design to account for the local demand from attachments. This standard attachment qualification framework for FP welded attachments allows for parallel design evolution that is decoupled from the larger facility analysis. For commodity designs, this provides engineers an efficient process for determining if a given attachment with a set of reactions is pre-qualified. For the global SC wall design, this methodology substantially reduces or eliminates a highly variable input (attachment demand) in the facility analysis. When implemented during early phases of a nuclear facility design, this methodology can result in a significant cost and time savings by increasing efficiency for the design of commodity attachments on SC walls.

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