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DESIGN REQUIREMENTS FOR STEEL-COMPOSITE WALL RIB AND STIFFENER CONNECTIONS

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ABSTRACT

Use of Steel-Composite (SC) walls is increasingly common in the design and construction of new nuclear power plants. During the course of fabrication and erection, various components of the SC wall modules serve multiple functions beyond those required by the in-service design basis. One component of an SC wall module that can provide functionality during fabrication, shipping, erection, and operation, in some cases, is an internal rib, or stiffener, attached to the faceplate. Prior to full field installation an internal rib provides stiffness to maintain the integrity and shape of the module, while during operation the rib may resist buckling of the faceplate. This paper discusses existing code requirements within ANSI/AISC N690-18 (2018) for the design of the weld between the rib and the faceplate. Additionally, opportunities to advance the code to improve economy, constructability and tolerance control by reducing warpage are identified.

CODE REQUIREMENTS FOR RIB CONNECTION DESIGN

The code requirements for the design of SC wall ribs and their connections to faceplates are provided in ANSI/AISC N690-18 (2018), Section N9.1.1(k). This section stipulates, "Steel ribs, if applicable, shall be embedded into the concrete no more than the lesser of 6 in. (150 mm) or the embedment depth of the steel anchor minus 2 in. (50 mm). The ribs shall be welded to the faceplates and anchored in the concrete to develop 100% of their nominal yield strength." No further elaboration on the term 'nominal yield strength' is provided within the SC Walls provisions (Appendix N9) of ANSI/AISC N690-18 (2018), thus a conservative interpretation of rib tension yield (normal to the faceplate), as opposed to shear yield (in-plane of the faceplate), can conservatively be used to satisfy the nominal yield strength requirement.

The nominal yield strength of a steel rib in tension is determined via evaluation of the tensile design strength provision of ANSI/AISC 360-16 (2016), Chapter D. Tensile yielding of the rib is established by Equation D2-1 of ANSI/AISC 360-16 (2016) and shown here in Equation 1. Note that the resistance factor to determine design tensile strength is included in Equation 1.

$$P_n = F_y A_g \tag{1}$$

Where P_n is the nominal axial strength of the rib, F_y is the specified minimum yield stress of the rib material and A_g is the gross area of the rib. Figure 1 illustrates the design configuration stipulated by current ANSI/AISC N690-18 (2018), Appendix N9, provisions.

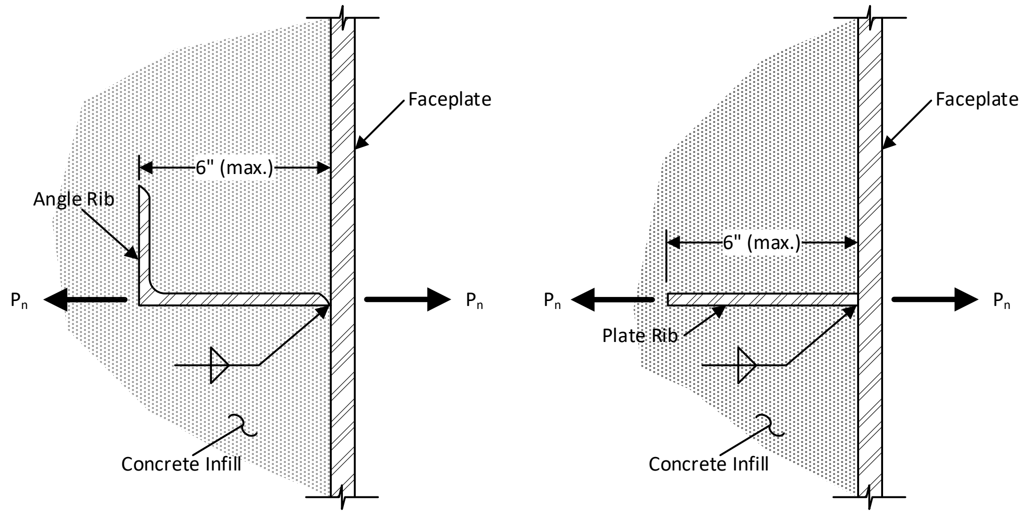

Figure 1. Typical Rib Detail per ANSI/AISC N690-18 (2018) Requirement.

Design of the weld at the rib connection to the faceplate is based on ANSI/AISC 360-16 (2016), Chapter J requirements. The code does not require a specific weld type for this connection, though due to the relative ease of execution of a fillet weld a double-sided fillet weld is shown in Figure 1. A partial-joint-penetration (PJP) or complete-joint-penetration (CJP) weld may also be used. The minimum number of weld passes required for a fillet weld is per AISC-325 (2017), Chapter 8, Table 8-12.

Table 1: Required double sided fillet weld sizes based on rib design tensile strength.

Rib Thickness [in]	Minimum Faceplate Thickness [in]	Double -Sided Fillet Weld Size [in]	Number of Weld Passes Required
1/4	1/4	3/16	1
3/8	3/8	1/4	1
1/2	1/2	5/16	1
5/8	5/8	3/8	3
3/4	3/4	1/2	4
7/8	7/8	1/2	4
1	1	5/8	6

Applying the fillet weld requirements of ANSI/AISC 360-16 (2016), Chapter J, Equations J2-4 and J2-5, and base metal tensile yielding and rupture requirements of Equations J4-1 and J4-2, respectively, to the connection capacity requirements dictated by the rib design tensile strength results in the weld sizes for each specified rib thickness shown in Table 1. Note that F_y is taken as 36 kips per square inch (ksi) and represents commonly used ASTM A36 material for either an angle or flat plate rib. An E70 weld electrode is assumed which is standard practice for welds joining ASTM A36 material. Further, the minimum faceplate thickness for each rib thickness is provided such that the minimum faceplate thickness is equal to the thickness of the rib. A minimum faceplate F_y of 60 ksi is used, which is within the minimum yield strength requirements of ANSI/AISC N690-18 (2018), Section N9.1.1(d).

DESIGN IMPLICATIONS OF CURRENT CODE REQUIREMENTS

While there is no codified limitation on the thickness of a rib, practical limitations exist due to warpage of the faceplate. Thicker rib plates/angles require increasingly large weld sizes as shown previously in Table 1. Even substitution of a PJP or CJP weld for the double-sided fillet weld is cumbersome as these are still substantial welds. Execution of these larger welds requires more passes of the welding implement to complete the weld, which correlates to more heat input for a longer time. This lengthy process to perform substantial welds results in significant deformation of the faceplate as shown in Figure 2.

Waviness of the faceplate is limited by provisions of ANSI/AISC N690-18 (2018), Chapter NM. The total waviness of a faceplate is comprised of two deformation mechanisms; deformation due to fabrication, such as warpage from execution of large rib welds, and deformation due to concrete placement since the faceplate serves as formwork when concrete infill is placed. As a result, designers should specify fabrication and erection waviness limitations based on project specific parameters. For a more flexible erection process it is recommended that fabrication tolerance be held to minimal values, preferably one-eighth inch (1/8″) to three-eighth inch (3/8″) based on wall thickness. These minimal waviness limitations will be challenging to achieve when large rib welds are required.

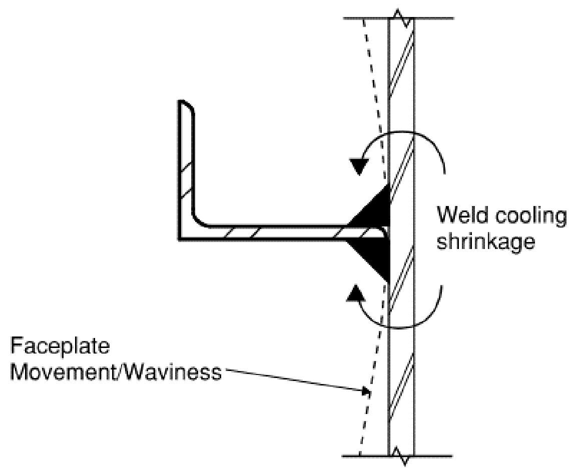

Figure 2. Faceplate Waviness Induced in Fabrication Due to Large Rib Welds.

Cost and schedule are practical factors that must also be considered when specifying such large welds at rib connections to faceplates. The rib-to-faceplate connection is continuous, occurring along the entire length of a module. Typically, several ribs are required on each faceplate of a module to ensure sufficient module rigidity during fabrication, shipping, and erection. Due to the size of new nuclear plant structures, the total linear footage of rib-to-faceplate weld required can be substantial.

The time required to execute large rib welds is a critical step in fabrication of SC modules. With erection schedules dependent on the delivery of modules on-site, fabricators must plan their work with sufficient time to execute substantial rib welds, perform the necessary inspections for weld quality and faceplate waviness, and apply any corrective measures necessary when a module does not pass the required inspections or meet the applicable acceptance criteria.

ALTERNATE INTERPRETAION OF CURRENT CODE REQUIREMENTS

In addition to the practical challenges associated with large rib welds required to develop the nominal tensile capacity of the rib, there are analytical aspects of the current code requirements that may be conservative when considering actual geometry or required functions for a particular design. Commentary of ANSI/AISC N690-18 (2018), Section CN9.1.1(k), indicates that ribs may be used to address local buckling of the faceplate. In this instance the rib serves as an embedment or anchorage to restrain the faceplate.

Use of the ribs for buckling restraint during operation assumes full development of the rib in tension. However, as discussed previously, ANSI/AISC N690-18 (2018), Section N9.1.1(k), limits the length of a rib to 6-inches. Achieving full development of the embedded rib, is unlikely with such a limited rib length. In order to reach the nominal tensile strength of the rib, the rib must remain anchored in the concrete infill. From a failure mode perspective, the concrete breakout strength of the rib must exceed the nominal tensile strength of the rib to achieve the level of tensile loading in the rib that corresponds to the code required nominal tensile strength.

While ribs and shear studs may be credited to perform the same faceplate buckling resistance function during SC module operation, the concrete breakout surface for the two is significantly different. The breakout surface for a single headed stud extends in all directions, leading to a conical shaped breakout structure which is based on the size/diameter and length of the headed stud as shown in Figure 4. As the ribs of SC modules are linear and continuous the breakout surface extends in one or two directions, per unit length. For example, an angle rib placed in the horizontal plane will have a breakout surface extending only above the rib as shown in Figure 3, and a plate rib would have no breakout strength. A breakout strength in direct tension can be calculated according to ACI 349-13 (2013) Appendix D, conservatively considering infinite edge distance, uncracked concrete, concentric loading, no strength reduction factors or reductions for non-ductile failure modes, the maximum embedment depth (6″), and the minimum angle leg considered herein (1/4″). Performing a comparison of rib steel tensile strength, to concrete breakout strength using the conservative assumptions above, results in a ratio of concrete tensile breakout strength to anchor strength of 3.9% for 5,000 psi concrete, and 6.9% for 7,000 psi concrete. Therefore, even when considering substantial conservatism, the concrete breakout strength of a linear rib will be substantially less than the design tensile strength currently required by ANSI/AISC N690-18 (2018) to evaluate the rib-to-faceplate weld, and the weld is not governing.

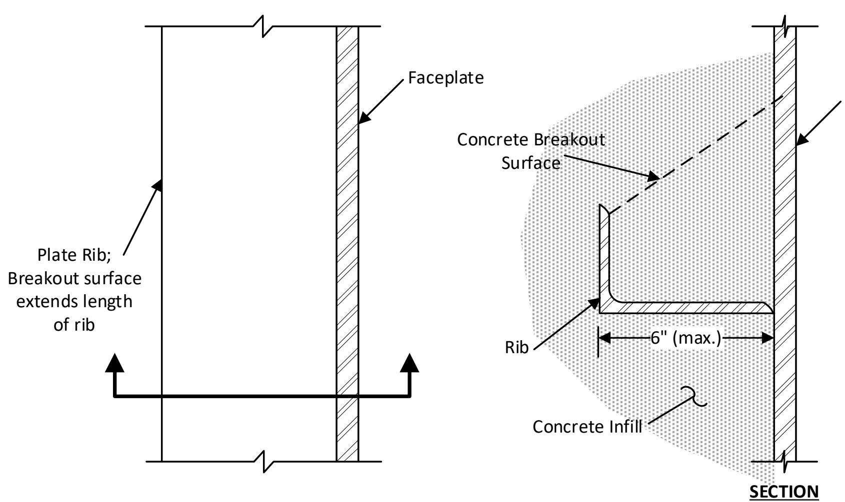

Figure 3. Breakout Surface of rib.

ANSI/AISC N690-18 (2018) currently does not require a minimum embedment to achieve any specified tensile strength, nor does it allow a reduction in weld design based upon the embedment. If the concrete breakout strength is less than the nominal tensile strength of the rib, and there is no codified provision for weld reduction based on the governing failure mode, design of the welded connection of the rib to the faceplate would exceed the direct tensile capacity of the concrete breakout. Further, if the rib is not designed as a restraint for faceplate local buckling, meaning is it designed solely as a construction aid (faceplate buckling restraint remains the function of shear studs only), the design of rib welds to the faceplate based on the nominal tensile strength of the rib is considered only to avoid local damage of the concrete or steel which would be detrimental to the performance of the components which are relied upon for strength.

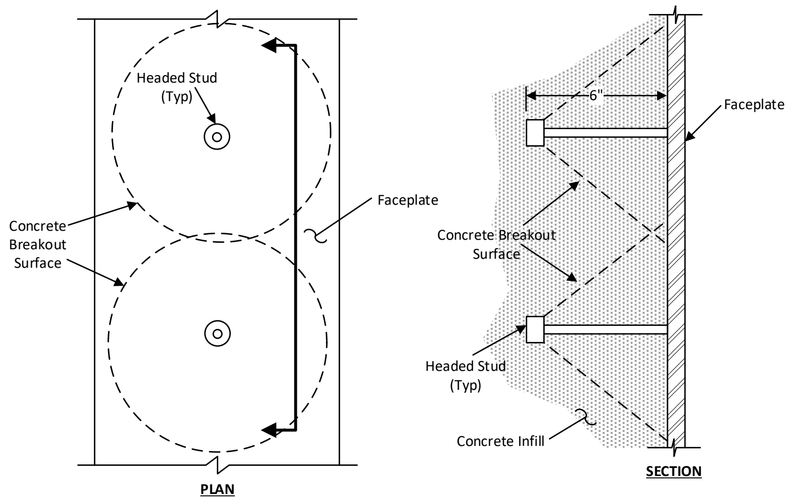

Figure 4. Breakout Surface of headed studs.

WELDDESIGN BASED ON ALTERNATE INTERPRETATION OF CODE

When the rib of an SC module cannot reach its nominal tensile strength prior to rib breakout from concrete infill, or the rib is not designed as a restraint against faceplate local buckling, the use of the shear design strength for the weld between the rib and the faceplate is an alternate approach. While the intent of ANSI/AISC N690-18 (2018), Section N9.1.1(k), the language does not preclude use of the rib shear design strength.

The nominal strength of a steel rib in shear is determined via by the shear design strength provision of ANSI/AISC 360-16 (2016), Chapter G using Equation G3-1 of ANSI/AISC 360-16 (2016) and shown here in Equation 2. Note that the resistance factor to determine design shear strength is included in Equation 2.

$$V_n = 0.6F_y btC_{v2} \tag{2}$$

Where V_n is the nominal shear strength of the rib, F_y is the specified minimum yield stress of the rib material, b is the width of the angle leg or tee stem, t is the thickness of the angle leg and C_{v2} is the web shear buckling coefficient. Due to the size limitation placed on rib elements, and since the ribs are encased in concrete during operation, parameter C_{v2} is taken as 1.0 per provisions of ANSI/AISC 360-16 (2016), Section G2.2. In cases where ribs are flat plates Equation 2 is still valid since only a single angle leg or tee stem is considered to determine the design shear strength.

The weld evaluation previously completed for the rib-to-faceplate weld connection based on rib nominal tensile strength is repeated, though updated to reflect the weld capacity meets or exceeds the shear design strength of the rib. All other parameters from the previous weld evaluation are held constant, and the ANSI/AISC 360-16 (2016), Chapter J requirements are used again to determine weld size. Table 2 provides the double-sided fillet sizes that result from use of the shear design strength.

Table 2: Required double sided fillet weld sizes based on rib design shear strength.

Rib Thickness [in]	Minimum Faceplate Thickness [in]	Double -Sided Fillet Weld Size [in]	Number of Weld Passes Required
1/4	1/4	1/8	1
3/8	3/8	3/16	1
1/2	1/2	1/4	1
5/8	5/8	5/16	1
3/4	3/4	3/8	3
7/8	7/8	7/16	4
1	1	1/2	4

DESIGN/FABRICATION IMPLICATIONS FOR ALTERNATE INTERPRETATION OF CODE

A comparison of double-sided fillet weld sizes from Table 1, based on the rib tensile design strength, and Table 2, based on the rib shear design strength, is provided in Table 3. Reductions in weld sizes range from one-sixteenth inch (1/16″) to 1/8″. While these weld size reductions seem minimal, the practical implications of this reduction are significant.

Table 3: Reduction in double sided fillet weld sizes through use of rib design shear strength.

Rib Thickness [in]	Minimum Faceplate Thickness [in]	Reduction in Double - Sided Fillet Weld Size [in]	Reduction in Number of Weld Passes
1/4	1/4	1/16	0
3/8	3/8	1/16	0
1/2	1/2	1/16	0
5/8	5/8	1/16	2
3/4	3/4	1/8	1
7/8	7/8	1/16	0
1	1	1/8	2

The lesser weld sizes help reduce warpage at the rib interface with the faceplate. Fewer weld passes are required to execute the reduced weld sizes, meaning the time required to complete the weld is reduced and heat will not spread in the faceplate far beyond the connection to the rib. A reduction in warpage also means it is more likely the faceplate waviness limit will be met, or that fewer corrective measures (i.e. use of a jig) are required during, or upon completion, module cooling after welding is done.

As noted previously, there may be linear miles of rib within the SC modules of a new nuclear facility. Reducing the size of the weld has substantial implications for fabrication in terms of time and cost savings. Requiring a lesser amount of weld metal to lay down at a given welded connection permits the welder to move more quickly. Further, the weld size reductions due to use of the rib shear design strength reduce the number of weld passes to complete the fillet weld. For 5/8" and 1" thick ribs, two fewer weld passes are required, while for the 3/4" rib, one less weld pass is required. For large SC wall modules these thicker ribs are expected to be common. Considering the welded connection of ribs to faceplate are double-sided, the savings in weld passes is doubled as well. When applied over miles of rib connection welds, the time and cost savings during the fabrication process is substantial.

TECHNICAL CONSIDERATIONS FOR ALTERNATE INTERPRETATION OF CODE

When designing the rib weld, failure of the weld and subsequent damage to the faceplate could be a concern. Even if the rib is not relied on to restrain faceplate buckling during design-basis conditions, failure of the weld at the faceplate should not impact the structural integrity of the faceplate. The weld in Figure 5 was investigated by Preece (1968). The strength of the weld is taken as the least shear strength of each plane. Testing determined that stress in the fusion area is not critical to establishing the shear strength of the joint. For the weld shown in Figure 5, results from Preece (1968) indicate the weld at Plane 2-2 will not be the governing failure mechanism of this weld, thus damage to the faceplate via failure on the rib weld is not a concern.

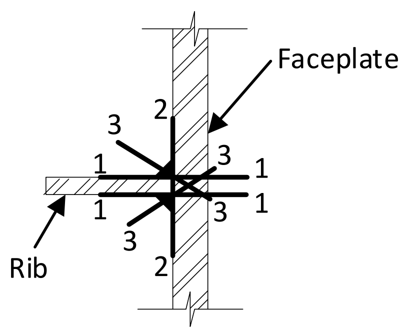

Figure 5. Fillet Weld Failure Planes.

ALTERNATE DESIGN OPTIONS

The use of ribs in SC modules presents significant challenges to efficient and cost-effective fabrication based on current ANSI/AISC N690-18 (2018) requirements for welded connections between ribs and faceplates. Provided in the following are options to mitigate the impact of rib weld code requirements for the welded connection to the faceplate.

Eliminate Use of Ribs

As noted previously, the ribs of SC modules serve multiple functions, to provide stiffness and maintain module integrity/shape during fabrication, shipping and erection, and to restrain the faceplate against local buckling during design-basis conditions (if considered by the engineer to reduce studs). If restraint of the faceplate is not required, use of alternate means to provide module stiffness during the fabrication and construction processes is recommended.

The use of a robust tie system between module faceplates may also serve to maintain module integrity in a manner similar to ribs. Ties between the faceplates are a requirement of SC module design per ANSI/AISC N690-18 (2018), Section N9.1.5. For structures with considerable demand, particularly seismic or aircraft impact demand, the size of ties is substantial and spacing between adjacent ties is limited. High structural demand, and the need for large closely spaced ties to satisfy the design-basis condition will also stiffen the SC module during fabrication and erection. The ties may be evaluated to their tensile and shear capacities in the fabrication and construction condition, since design-basis loading is not present at that time.

Integration of ribs with ties, where ribs are not in direct contact with the faceplate also eliminates the need to weld a rib to the faceplate. In this case the stiffness needed to maintain module integrity are integral with the ties required between faceplates. Figure 6 illustrates one method to achieve an integration of module ties and rib. A single flat plate is used. Horizontally spanning portions of the plate are designed as the structural between the faceplates and meet the design-basis requirements for the SC modules. The welds between the ties and faceplates are subject to the requirements for ties, as described in ANSI/AISC N690-18 (2018), Section N9.1.5. The construction ribs span vertically and utilize a portion of the structural tie. Since the ribs are credited for fabrication, handling and erection functions only, their structural demand is not concurrent with design-basis demand and the overlap of these functions is permissible.

The option provided in Figure 6 addresses module out-of-plane stiffness, though in-plane stiffness must also be addressed. Without additional in-plane stiffening measures the SC module could ‘pancake,’ meaning the faceplates of a given module displace in opposing directions within the plane of the module until the welds between the ties and faceplates fail, resulting in module collapse prior to installation. Placement of steel elements between adjacent combined tie and rib plates (i.e., braces, or similar) requires minimal additional material while providing resistance to SC module collapse due to in-plane failure.

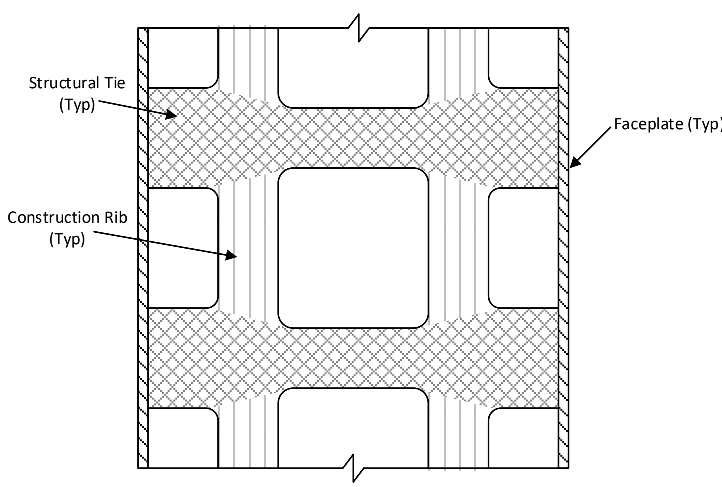

Figure 6. Use of Combined Tie and Rib System.

Specify Flow Holes in Ribs

Maintaining the design of the rib-to-faceplate weld based on the tensile design strength of the rib can be achieved, while also reducing the weld size, by reducing the tensile design strength of the rib section. The proposed configuration shown in Figure 7 maintains the rib weld to the faceplate, however large flow holes are cut out of the rib to reduce the tensile design strength of the rib section.

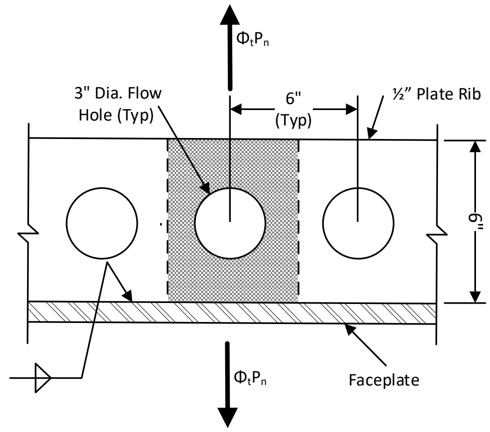

Figure 7. Use of Large Flow Hole in Rib

The reduced cross-section of the rib is evaluated for yielding of the gross section and rupture of the net section per ANSI/AISC 360-16 (2016), Section D2 requirements. The shear lag factor, U, used for evaluation of the rib in tension is 1.0 per Case 3 in Table D3.1 of ANSI/AISC 360-16 (2016). Case 3 is selected due to the rib tensile load acting perpendicular to the weld. The gross area for the sample configuration on Figure 7 is determined and applied to Equation 1, provided previously. The net section area is determined and applied to Equation 3, which corresponds to Equation D2-2 from ANSI/AISC 360-16 (2016).

$$\Phi_t P_n = F_u A_e \tag{3}$$

In Equation 3, A_e is the effective area, and Φ_t is taken as 0.75 for the net tension case. Use of 3″ diameter flow holes, spaced at 6″ on-center, in a ½″ thick rib plate results in rupture of the net section as the controlling nominal strength. The double-sided fillet weld size is determined once again per ANSI/AISC 360-16 (2016), Chapter J. The required double-sided fillet size is now 3/16″, which is the minimum weld size permitted for a 1/2" thick plate per ANSI/AISC 360-16 (2016), Chapter J, Table J2.4. Recall from Table 1, a 5/16″ double-sided fillet weld was required when yielding of the gross rib section governed the tension loading condition. The addition of large flow holes reduces the design tensile strength to the rupture strength of the net section providing a 1/8″ reduction in weld size for this sample case.

The addition of flow holes also provides the critical construction function of eliminating voids beneath the rib. Placement of concrete infill beneath horizontal SC module components has the potential for voids, particularly at the interface of the rib and faceplate. Inclusion of flow holes provides a path for the concrete and air to move around and through the rib element. Another option places semi-circular flow holes at the interface of the rib and faceplate. This configuration has the added benefit of flow holes at the location concrete voids are most likely to occur. Concrete will pass through the rib plate, shown in Figure 8, rather than create an air pocket beneath the rib at the connection to the faceplate. The caveat inherent in this option is the discontinuous weld between the rib and the faceplate, which makes the use of mechanized welding more challenging due to repeated stops and starts, and also reduces the available weld length, requiring larger welds. Additionally, the likelihood of inclusions and non-conformances in the rib-to-faceplate weld increasing with the repeated stopping and starting required for configuration in Figure 8.

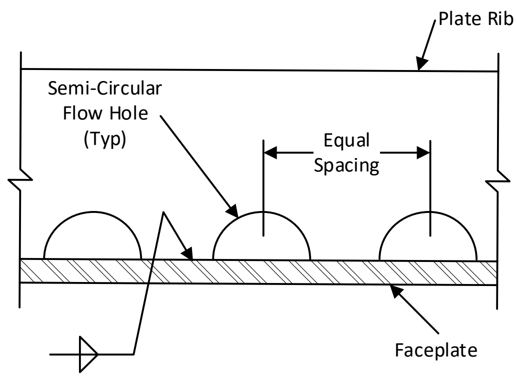

Figure 8. Use of Large Semi-Circular Flow Hole at Rib Interface With Faceplate

CONCLUSION

Current AISC N690-18 (2018) code provisions for welds of ribs to faceplate result in large welds that are time-consuming and costly while also increasing the likelihood of warping of the faceplate. Analysis provided in this paper shows that weld sizes are reduced when rib shear design strength is used, however additional testing may be required to substantiate. Alternatively, if the welds are not relied on for design strength or buckling resistance, and failure is restricted to the effective throat or angle base metal, the weld can be designed for demand in fabrication and erection. Smaller welds reduce the cost and time required to complete the weld, while also reducing the likelihood of faceplate warpage. Additionally, alternate details are provided to reduce the tensile strength of the rib, resulting in smaller welds without compromising the stiffness of the rib for module fabrication, handling, and installation. Further, use of robust ties are likely to eliminate the need for the stiffness provided by the ribs, where supplemented with bracing, thus eliminating the rib welds at the faceplate entirely.

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