Transactions, SMiRT-24
BEXCO, Busan, Korea - August 20-25, 2017
Division VI

ON THE CONFLICTS OF BOUNDARY ELEMENT AND IN-PLANE SHEAR LIMIT REQUIREMENTS IN ACI 349

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ABSTRACT

Comprehensive finite element (FE) models with complex geometry, including multiple intersecting concrete walls with or without openings, are employed in nuclear power plant (NPP) structures. The analysis and design procedures for NPP structures are continuously emerging based on the most recent research and development. However, the applicability of some of the current design code provisions to the complex NPP FE models is not clear. The lack of guidance on how to post-process the detailed FE analysis output data and use of the current design methods available in the existing codes may result in delays in the design process and also unduly conservative designs. Accordingly, the purpose of this paper is to present code-related issues associated with boundary elements and in-plane shear limit requirements often experienced by engineers during the analysis and design of nuclear safety-related shear wall concrete structures. The discrepancies among design codes, code revisions, and chapters within a code revision pertaining to various aspects of reinforced concrete behavior are discussed. Recommendations based on experience are presented to address the specific issues of the respective code provisions.

INTRODUCTION

The design or assessment of nuclear safety-related concrete structures is performed in accordance with the requirements of special concrete codes and standards. ACI 349 covers the design and construction requirements for concrete structures that have nuclear safety-related functions. Similarly, ACI-ASME Committee 359, a joint committee of ACI and ASME, provides design and construction requirements (ACI 359-07) for concrete reactor vessels and containment structures (Section III, Division 2 of the 2007 ASME Boiler and Pressure Vessel Code). This paper addresses the existing boundary element and in-plane shear code provisions, along with the corresponding problems for which development and technical discussions might lead to clearer guidance for practicing engineers. The primary intent is to improve and facilitate the design process of NPP structures.

ACI 349 was first adopted based on the same requirements, basic parameters and equations of ACI 318. The effort was made subsequently to introduce the unique aspects of safety-related concrete structure design in the new revisions considering the major differences in performance objectives between ACI 349 and ACI 318. Even though some changes have been made and there have been continuing considerations to address the fundamental differences in design philosophy between ACI 349 and ACI 318, there are still areas that need further clarification and improvement for current design practice. This paper specifically focuses on problems in applying code provisions related to the boundary element and in-plane shear limit requirements. Reinforced concrete shear walls in conventional and safety-related concrete structures show significantly different behavior. Nuclear facilities are composed of squat walls as oppose to the tall walls of the conventional high-rise buildings. The main characteristic of squat walls is generally susceptible to shear failure rather than secondary in-plane flexure effect which is typically observed in the tall walls. The following subsections detail the significant differences in behavior of structural walls pertaining to conventional buildings and NPP structures, along with the specific issues experienced during the design stage of safety-related concrete structures.

STRUCTURAL WALLS IN CONVENTIONAL BUILDINGS AND NPP STRUCTURES

Structural concrete walls provide great strength and stiffness to resist earthquake loads and are widely used in NPP structures. Structural walls with height-to-length ratio $h_w/l_w \geq 2.0$ are slender or flexural walls in which lateral load is resisted by flexural action. On the other hand, walls with $h_w/l_w < 2.0$ are called short or squat walls and are typically shear-critical structures in which lateral load is transferred through a path similar to a truss analogy consisting of struts and ties (Wight and MacGregor, 2012). Coupled walls are structural systems consisting of individual cantilever walls, called wall piers, connected by horizontal elements, called coupling beams. The term "coupled wall" is used more frequently for walls in conventional applications, such as high-rise residential and commercial buildings, as shown in Figure 1. The aforementioned slenderness criteria for cantilever walls apply also to coupled walls. Coupled walls resist lateral loads by a combination of flexural action in wall piers and frame action generated by shear demand in coupling beams.

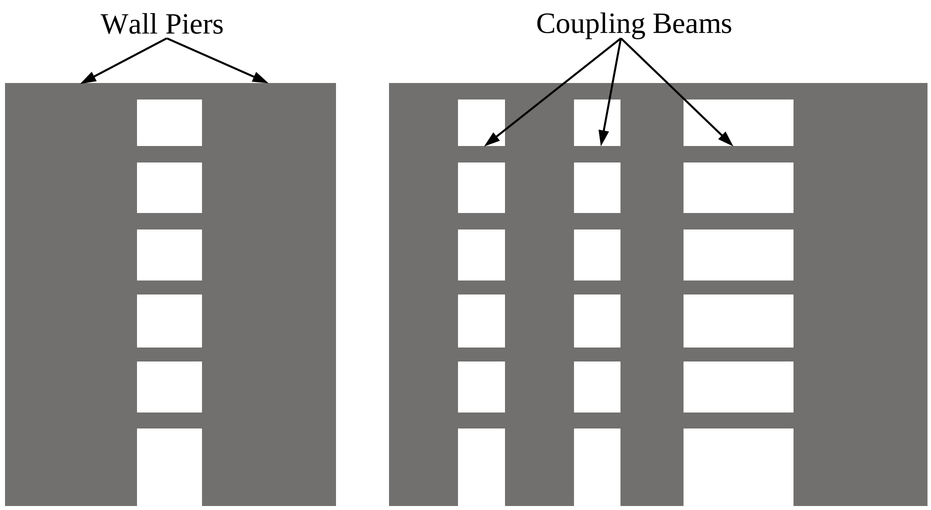

Figure 1. Examples of coupled wall geometry in conventional (residential and commercial) buildings (Harries et al., 2000).

The NPP structures have quite complex geometries, consisting of intersecting walls and openings. Most of the walls in these structures fall in the squat wall ($h_w/l_w < 2.0$) category. These structures are analyzed using rigorously developed FE models to represent their global behavior. Unlike conventional buildings, openings in NPP structural walls are of various sizes and are usually scattered to serve different purposes such as piping and equipment penetrations, as shown in Figure 2. Thus, considering the fundamental differences between walls in conventional buildings and safety-related nuclear structures, it is important for design engineers to understand the technical basis of the relevant code provisions.

Figure 2. Example of shear wall geometry in NPP structures.

BOUNDARY ELEMENTS IN SHEAR WALLS

Under lateral earthquake loading, flexural coupled wall systems exhibit inelastic behavior where large deformations and eventually plastic hinges form in coupling beams or horizontal wall segments. This is followed by large deformations near the bottom of the walls (Eljadei and Harries, 2014). This behavior is illustrated in Figure 3, which shows the conventional behavior of a coupled shear wall structure subjected to earthquake loading. As can be seen in Figure 3, tension stresses are generated on one side of the wall pier and compressive stresses are generated on the other as shown Figure 3.

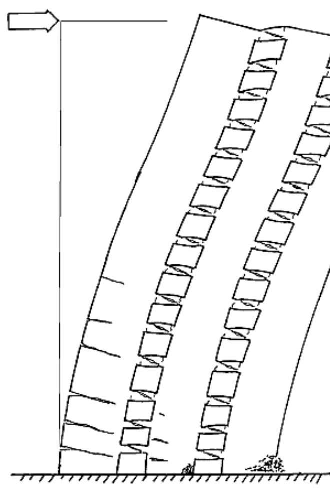

Figure 3. Coupled wall geometry and conventional yield mechanism (Subedi et al., 1999).

In addition, spalling of the unconfined concrete and buckling of longitudinal reinforcement occur at large compressive strains near the base of the wall if sufficient lateral support is not provided. The need of a boundary element arises when the compression strain of concrete at the described critical sections exceeds the spalling strain of concrete. Boundary elements are portions of the structural wall, near wall openings and edges, which are strengthened by longitudinal and transverse reinforcement. The transverse reinforcement provides adequate confinement and increases ductility and deformability at critical section locations. The boundary elements resist cyclic loads and prevent reinforcement buckling near the wall edges. As shown in Figure 4, boundary elements do not necessarily require an increase in wall thickness.

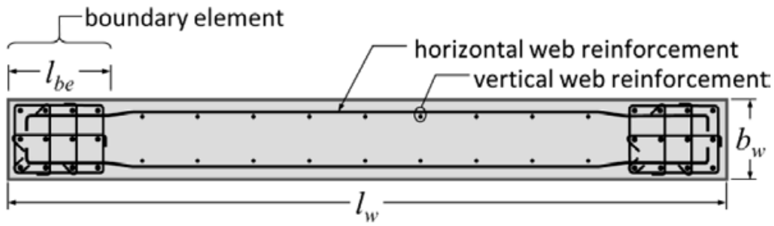

Figure 4. Typical reinforcement for cantilever wall with boundary elements (Moehle et al., 2012).

According to ACI 349-01, Section 21.6.1, boundary elements are not required for shear walls with h_w/l_w less than 2.0, and the term “wall pier” does not exist in the provision. ACI 349-01 requires boundary elements at boundaries and edges around openings of structural walls with $h_w/l_w \geq 2.0$ when the maximum extreme fiber stress, which corresponds to factored forces that include earthquake effects, exceeds $0.2f_c'$ (index method). The boundary elements are permitted to be discontinued at a section where the calculated compressive stress is less than $0.15f_c'$, where f_c' is the compressive strength of the concrete.

The ACI 349-06 and ACI 349-13 revisions exclude the index method because the method is indirect and produces conservative estimates of the need for boundary elements. ACI 349-06 (Section 21.7.6.1) and ACI 349-13 (Section 21.9.6.1) also state that boundary elements are not required for squat walls, and the term “wall pier” is also included in the same provisions, unlike ACI 349-01 (Section 21.6.1). With respect to walls and wall piers with $h_w/l_w > 2.0$, the need for boundary elements should be established by cross-section analysis for flexural and axial loads, and the boundary elements should be provided if the maximum compression strain in the cross-section analysis exceeds 0.002.

In general, the spalling process in reinforced concrete occurs gradually, and it is actually difficult to determine the strain at which concrete spalling starts. The most commonly accepted approach is that the concrete cover spalls when the extreme compression fiber reaches a strain of 0.004. However, the strain limit of 0.002 is adopted by ACI Committee 349 for combined actions, including safe-shutdown earthquake (SSE), assuming that the beyond-design-basis earthquake (BDBE) displacement is less than twice the SSE displacement. Thus, there is no need to perform seismic analysis for BDBE, recognizing that 0.004 is a threshold for boundary element requirement. In fact, the displacement-based approach (Moehle, 1992) in current industry practice provides a more rational assessment than the $0.2f_c'$ stress limit to determine whether the special boundary elements are required.

The opening sizes and wall pier definition are other important issues for boundary element evaluation. As described previously, openings in regular residential and commercial buildings are usually the same size along the height of the buildings, and the opening size is the typical door or window size. Considering the wall and opening dimensions in conventional buildings, one can see that there is discontinuity in load transfer and stress concentration around these openings. In NPP walls, openings are of various sizes, and several small-size openings may be present in one wall. The small openings may not cause discontinuity in load transfer and can be ignored. However, no guidance is provided by ACI 349 as to under what conditions these openings can be excluded in the design process regarding boundary elements.

Design engineers need to make some assumptions considering the opening sizes throughout a wall to identify the wall piers since previous versions of ACI 349 do not include an explicit definition for wall piers, but instead, only a general description with figures, without guidelines, e.g., dimension limits, to determine whether corresponding wall segments should be considered as wall piers or not. A wall can be idealized as a wall pier, assuming that the opening is large enough to disrupt the shear flow in the wall if the vertical in-plane shear capacity is less than the demand across the opening. ACI 349 neglects the remaining vertical in-plane shear capacity of the wall above and below the opening and requires reinforcing the edges around the opening with boundary elements. Thus, the corresponding wall section with opening is designed as independent wall piers that share a common shear force, which means that each wall pier has its own tension and compression sides. However, the entire wall with openings functions as a single unit if the vertical in-plane shear capacity above and below the wall is sufficient for the demand across the opening. In this case, for example, a threshold value representing the opening sizes along a certain path through the wall could be proposed by the design engineer such that the edges around the openings can be disregarded for determination of the boundary element requirement. Therefore, the design engineer should first determine whether the vertical wall segments bounded by openings can be considered as wall piers or the entire wall section can be considered as a single unit.

In ACI 318-14, a wall pier is defined as vertical wall segments with dimensions and reinforcement intended to result in shear demand being limited by flexural yielding of the vertical reinforcement in the pier. Based on the definition in ACI 318-14, a wall pier, as shown in Figure 5, is a vertical wall segment within a structural wall, bounded horizontally by two openings or by an opening and an edge. In addition, ACI 318-14 introduces a new requirement for wall piers in which the horizontal length-to-wall thickness ratio (l_w/b_w) should be less than or equal to 6.0 and the clear height-to-horizontal length ratio (h_w/l_w) should be greater than or equal to 2.0. This new requirement raises the question of how to treat a vertical wall segment that does not meet these geometric limitations, which ACI 318-14 has yet to address. In the authors' view, if a vertical wall segment does not meet wall pier requirements, the entire wall can be considered as a single unit, and boundary elements are not required for the edges around openings. It is noted that wall pier definitions are the same in both ACI 349-13 and ACI 318-14. However, ACI 318-14 provides clearer guidance on how to determine wall piers, which could be incorporated in future revisions of ACI 349 to improve the shear wall design process.

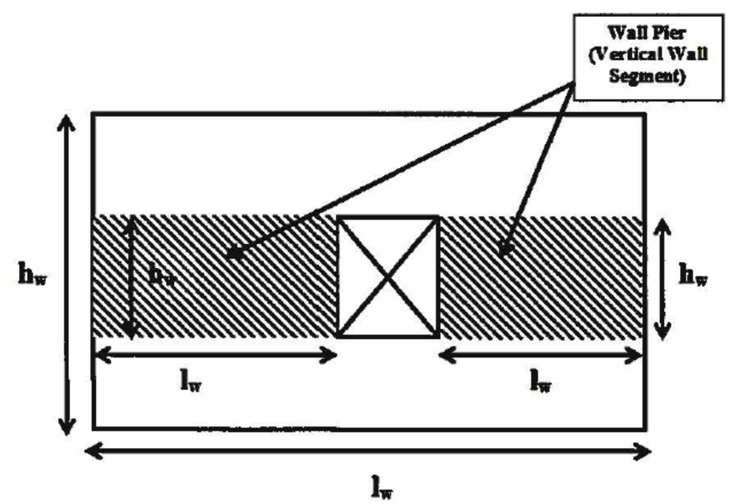

Figure 5. Vertical wall segments.

In contrast, and as described above, openings in structural walls or coupled walls in NPPs differ from those in conventional buildings as they are of various sizes and are usually scattered. This makes the stress or strain distributions more complicated and dispersed throughout a wall. Due to the high number of mechanical equipment and penetrations acting as point loads, high stresses/strains may be present in locations other than typical bottom wall edges and around openings. ACI 349 provides no guidance for such cases as to whether boundary element necessity needs to be checked and established, provided that wall pier requirements are satisfied.

Figure 6 shows an example stress contour plot for a structural wall in a nuclear facility where peak compressive stresses can be observed at other areas throughout the wall instead of the critical section locations that require boundary elements, as previously illustrated in Figure 3. These peak compressive stresses may be due to complex geometry and different load cases, such as pipe impact, pipe reactions, or accident pressure load rather than seismic forces. As can be observed from Figure 6, there is no continuity in stress distribution from the bottom edges of the wall towards the top. Therefore, providing boundary elements for these isolated areas only because of ACI 349 stress/strain limit requirements is redundant. This would lead to a very tedious design process for NPP structures and may result in an impractical wall design, leading to construction errors and delays. Furthermore, providing redundant boundary elements at the base of the wall due to the conservative index method may lead to significant shear distress prior to the yielding of boundary flexural reinforcement, even though considerable detailing is provided at the wall boundaries (Wallace and Orakcal, 2002).

The areas with high local stresses/strains as shown in Figure 6 are essentially designed by performing element-by-element structural design based on FE analysis results at the node or element level under the most critical load combinations. The vertical or horizontal segment that contains the highest demand is designed entirely to resist such demand. In addition, local structural integrity at high stress locations is in fact maintained by providing sufficient transverse reinforcement as a result of element-by-element design to resist longitudinal reinforcement buckling and concrete spalling. Also, high compressive stresses can be averaged based on ACI 349 design philosophy, ultimate strength design methodology, which considers that the element forces are redistributed to the less stressed regions when the highly-stressed regions of the section reach their strength limit. Other sections at upper levels of the wall can be designed in a similar manner.

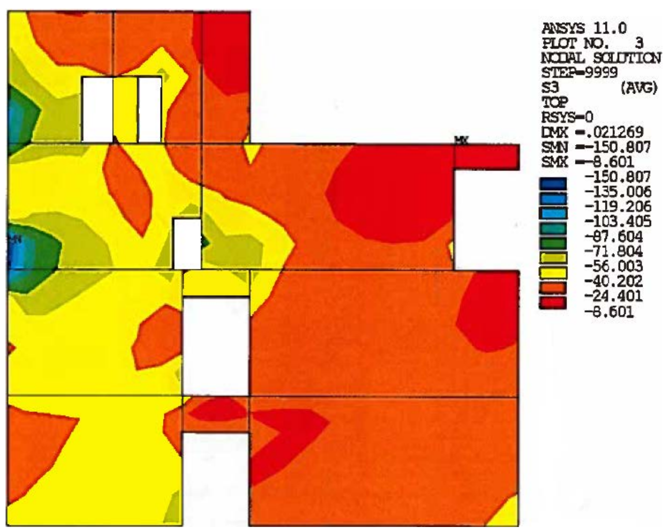

Figure 6. Stress distribution in an NPP structural wall.

IN-PLANE SHEAR LIMIT REQUIREMENTS

Another controversial issue in the design or qualification of shear walls is the peak in-plane shear strength of the reinforced concrete walls. Shear walls must resist in-plane shear forces induced by earthquakes. In coupled wall systems, a wall pier may individually resist in-plane shear, or two or more wall piers may share a common shear force. In addition, coupling beams or horizontal wall segments must resist shear forces in the vertical direction generated due to the frame action of the coupling beams. The total in-plane shear contribution from concrete (V_c) and steel reinforcement (V_s) based on Section 11.2 and Section 11.4

of ACI 349-13, respectively, is limited to the equations provided in the following sections because the total in-plane shear demand should not exceed the upper-bound in-plane shear strength limits.

ACI 349 - Chapter 11

The calculated nominal in-plane shear strength (V_n), the summation of the in-plane shear contribution from concrete and the reinforcement, at any horizontal section of a wall is bounded by the following equation provided in Chapter 11 of ACI 349-13, which is adopted from Chapter 11 of ACI 318-08:

$$V_n \leq \phi 10\sqrt{f_c'}hd \tag{1}$$

where ϕ is the strength reduction factor for shear (0.85), f_c' is the compressive strength of concrete, h is the wall thickness, and d is the distance from extreme compression fiber to centroid of longitudinal tension reinforcement. Chapter 11 of ACI 318-08 specifies the d distance as $0.8l_w$, unless a larger d value is determined by strain compatibility analysis.

ACI 349 - Chapter 21

Chapter 21 modifies Chapter 11 of ACI 349-13 for seismic design and limits the maximum in-plane shear strength of structural walls and diaphragms by the following equation:

$$V_n = \phi A_{cv}\left(\alpha_c\sqrt{f_c'} + \rho_t f_y\right) \tag{2}$$

where A_{cv} is the gross area of concrete section bounded by web thickness and length of section in the direction of shear force considered, α_c is a coefficient defining the relative contribution of concrete strength to nominal wall shear strength, ρ_t is the ratio of area of distributed transverse reinforcement to gross concrete area perpendicular to that reinforcement, and f_y is the specified yield strength of reinforcement.

The coefficient α_c in Equation 2 above recognizes the higher shear strength of walls with high shear-to-moment ratios and depends on the geometry of the entire wall or wall pier. The coefficient α_c is 3.0 for $h_w/l_w \leq 1.5$ and 2.0 for $h_w/l_w \geq 2.0$. The coefficient α_c varies linearly between 3.0 and 2.0 when h_w/l_w is between 1.5 and 2.0, where h_w and l_w are the height and the length of the entire wall or wall pier, respectively. ACI 349 and ACI 318 require that the larger h_w/l_w ratios for the entire wall and segment of the wall should be considered when determining V_n for segments of a wall.

Chapter 21 of ACI 349-13 also provides the maximum in-plane shear strength requirement for wall piers and horizontal wall segments (coupling beams). Figure 7 illustrates wall piers (vertical wall segments) and horizontal segments in a typical NPP wall, as described in ACI 349-13. The nominal in-plane shear strength is limited to $8A_{cv}\sqrt{f_c'}$ for wall piers that share a common lateral force and $10A_{cw}\sqrt{f_c'}$ for any individual wall pier, based on Chapter 21. Additionally, $10A_{cw}\sqrt{f_c'}$ is given as maximum in-plane shear strength for horizontal wall segments. A_{cw} represents the cross-sectional area of the concrete section of an individual wall pier or horizontal wall segment.

As described above, the $8A_{cv}\sqrt{f_c'}$ and $10A_{cw}\sqrt{f_c'}$ limits are given for wall piers and horizontal wall segments in ACI 349-13 but not necessarily for a continuous cross-section above the opening shown in Figure 7. Therefore, Equation 2 is the only applicable in-plane shear limit for the continuous cross-section based on Chapter 21.

Comparison of ACI 349 Chapter 11 and Chapter 21 for Maximum In-Plane Shear Requirement

Chapter 21 of ACI 349 contains design requirements for reinforced concrete structures related to seismic loads. Section 21.1.1.2 of ACI 349-13 states that the safety-related concrete structures should satisfy the requirements of Chapter 1 through 19, and if there is a conflict between these chapters and Chapter 21, Chapter 21 controls the seismic design. Therefore, in-plane shear strength limit of structural walls can be checked using both Equation 1 and Equation 2.

One of the main differences between Chapter 11 and Chapter 21 of ACI 349 regarding the in-plane shear limit requirements is the reinforcement contribution ($\rho_t f_y$) given in Equation 2. For a wall pier or a horizontal wall segment (see Figure 7), Chapter 21 of ACI 349, as previously described, imposes additional in-plane shear limits ($8\mathrm{A_{cv}}\sqrt{\mathrm{f_c'}}$ and $10\mathrm{A_{cw}}\sqrt{\mathrm{f_c'}}$) which always governs the peak shear strength. It is noted that the effective length used in calculation A_{cv} or A_{cw} in the in-plane shear limits is another major difference between Chapter 11 and Chapter 21. While Chapter 11 uses $0.8l_w$ as effective length (or a larger d value if justified), the full section length l_w is employed in Chapter 21. However, both Chapter 11 and Chapter 21 equations result in an equivalent value of an effective length, and therefore an equivalent in-plane shear strength limitation, when wall piers sharing a common lateral force are considered.

On the other hand, for a continuous cross-section (See Figure 7), Equation 2 is the only limit used to check the peak shear strength based on Chapter 21. Also, Equation 1 should be satisfied according to Section 21.1.1.2 for seismic design. When these in-plane shear limits are compared, Equation 2 usually provides significantly large upper-bound limit because of the presence of the term ($\rho_t f_y$). As high in-plane shear demand due to seismic loads requires larger steel reinforcement ratios, un-conservative upper-bound in-plane shear limit is obtained for the continuous cross-sections in Chapter 21. The difference between Equation 1 and Equation 2 in-plane shear limit values is significant and unreasonable when the reinforcement ratio in Equation 2 increases considerably with higher seismic demand. Therefore, Equation 2 should be revised in future revisions of ACI 349.

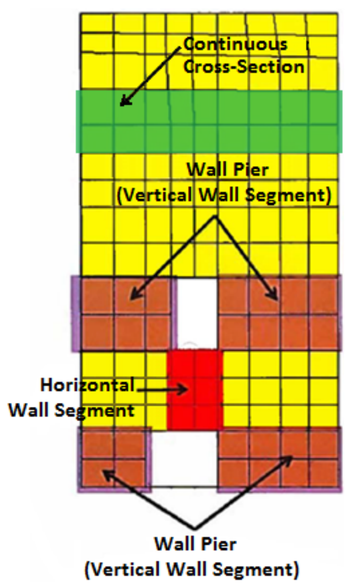

Figure 7. Vertical and horizontal segments in a typical NPP reinforced concrete wall.

In addition, ACI 349 does not address vertical in-plane shear explicitly in any revisions. Chapter 11 addresses only horizontal in-plane shear while Chapter 21 provisions include the horizontal wall segment which considers shear in the vertical direction. However, there is not any clear guidance on how to treat in-plane shear in vertical direction for other areas throughout the wall.

The seismic in-plane shear demand for wall piers is usually within code limits due to the regularity of the wall and opening arrangements in conventional buildings. On the other hand, in coupling beams or horizontal wall segments, the shear strength limit is generally exceeded under high seismic loads. To solve this issue in conventional building design practice, the design engineer typically proposes increasing the concrete compressive strength and/or reducing the effective stiffness of the coupling beams to reduce the high shear stresses so that the design meets code requirements. Therefore, current design practice, which is based on the ACI 318 strength-based design approach, may lead to unrealistic details for coupling beams and does not capture the expected behavior of coupled wall systems (McNeice, 2004). The performance-based design approach is better suited to address the high shear demand issue in high-rise non-nuclear building structures that are allowed to perform nonlinearly and experience structural damage in certain components and locations in their lateral force-resisting system.

Buildings in nuclear safety-related concrete structures are designed to behave in the elastic range and are not allowed to experience any damage. Also, the performance-based design approach, in its conventional way, is yet to be introduced in nuclear industry codes. Accordingly, the design engineer should develop a method to solve shear strength exceedance in wall piers and horizontal wall segments when designing reinforced concrete shear walls. Stress or force/moment averaging can be performed using the FE analysis output provided for all elements that have common nodes (Ozkan et al., 2015). The shear demand from each element is averaged along a critical shear section in the vertical or horizontal wall segment. The design engineer can consider different section cut lengths after reviewing the corresponding contour plots to reduce the potentially excessive conservatism. However, the averaged shear demand might still be high and require heavy steel reinforcement. This could be due to the combination of high seismic forces and insufficient wall-segment length (See Figure 7).

It should be noted that the in-plane shear limit can be improved by increasing the concrete compressive strength or the thickness of the wall piers or coupling beams since the shear strength limit equations include material and geometric parameters, such as compressive strength of concrete, f_c', and gross area of concrete for wall piers, A_{cv}, or for horizontal wall segments, A_{cw}. These options might be implemented easily in non-nuclear projects where teamwork and collaboration among architects and design engineers is emphasized and organized in the early stages of design. However, in nuclear safety-related projects, these options cannot usually be used by the design engineer as such properties are decided at the licensing stage and are based on different layout requirements, such as accommodation of mechanical equipment and penetrations. This in turn will significantly delay the design process of special reinforced concrete walls in particular for site-specific applications.

CONCLUSIONS

In this paper, common design problems for special reinforced concrete shear walls are presented for nuclear safety-related structures. The ACI 349-13 code provisions are heavily based on ACI 318-08. The ACI 318 code is intended for structural concrete in general and not specifically for nuclear applications. Both ACI 318 and ACI 349 provisions for special reinforced concrete structural walls generally apply to slender walls, which are common in conventional buildings, while shear walls are almost always squat in nuclear safety-related concrete structures. Therefore, the current ACI 349 provisions may create complex problems in the design process of shear walls in NPP structures due to inapplicability and/or lack of guidance. Particular provisions of ACI 349 regarding boundary elements and in-plane shear limit requirements are reviewed to highlight the potential areas where further studies and guidance may be necessary to 1) improve the clarity of the relevant ACI 349 provisions and 2) minimize the delays and overly conservative reinforcement demand for shear walls in the design stage of NPP structures. The key issues are identified, and specific recommendations are given based on experience to address the particular issues of the respective code provisions and to remedy the lack of guidance in the existing design codes. As demonstrated, the design of structural walls for NPPs can be a complicated and slow process, and the design engineers responsible for development of the detailed designs should exercise caution in interpreting relevant ACI code sections.

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