Behavior of Curved Double Steel-Concrete Composite Shear Walls Under Cyclic Loading

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Abstract. In this study, effects of different curved structures on mechanical behavior of curved double steel-concrete composite shear (SC) walls under cyclic loading were investigated numerically. Firstly, the experimental results of a straight SC wall under cyclic loading were used to validate numerical result. After then, curved SC walls were designed in ABAQUS according to different radius-thickness (R/t) ratios (5,10,15,20,25,30,40). In numerical studies, in-plane monotonic response, cycling response, and out-of-plane monotonic response were investigated by changing the loading. The results showed that among all parameters, the R/t ratio: 5 has better seismic performance by 11.87% than straight SC wall under cyclic loading. In addition, it was found that when the ratio of the radius of curvature to the section thickness values were greater than 10, curved SC walls behaved as straight SC walls. When the R/t ratio was bigger than 10, the ultimate load and damage of wall load dropped consistently. According to in-plane monotonic response results, R/t ratio: 5 has a 13.6% better peak force than straight wall when it is compared with other R/t ratio values. Calculated results showed that although there is no effect in bearing capacity when R/t ≥ 10, it increased by 35.4% when R/t is 5.

1. Introduction
Double steel-concrete composite shear walls (SC wall) are structural members in nuclear facilities and mega structures. In general, they compose of concrete, steel-plates, and studs. Concrete covered by steel plates and steel-plates connected each other with the help of the studs. Studs welded to steel-plates, which can help to resist internal and external forces. To optimize the design and service performance of SC walls, several studies were performed to investigate bending [1–4], in-plane [5], out of plane [6], and cyclic [7,8] behaviors of SC walls. Besides, several researchers were studied the effect of steel thickness of SC walls [8,9] on compression and tension behaviors. The studies showed that when the thickness of steel-faceplates increased, there would be a significant effect on compression and tension behavior. Therefore, SC walls were studied on the rise-to-span ratio [10], and the optimal ratio was found as 0.17. The optimal thickness and tie rod spacing ratio were studied [11] and found the slightest effect on the wall’s initial stiffness and strength. In another study, the optimization of connector spacing was investigated [12]. The obtained results showed that a rise in the number of intermediate fasteners increased the shear load capacity. Cyclic behavior of SC walls before and after fire exposure [13] was studied. After fire exposure, SC walls peak load reduced more than 16%.

In literature, to predict accurate and reliable numerical analysis results, several simulations with different assumptions were performed on SC walls. Firstly, a 2D model was used in VecTor2 [14]. The
rapture of steel studs could not calculate due to the assumption’s limitation. To solve this contact problem, the interaction between steel-plate and studs were simulated as spring elements [15–17] and beam elements [18–20]. Material models, such as the smeared cracking concrete model in MSC.Marc(2005r2) [15], concrete plastic model in ABAQUS [10], brittle cracking material model in ABAQUS [15,16], the Winfirth concrete model in Ls-Dyna [21–23] were used.

Varma [24] introduced a noncomplex mechanistic model of complex in-plane shear behavior of SC composite walls. A design equation that can predict composite walls’ in-plane shear and strength was found. Sener [25] claimed that in-plane shear strength and ductility are the essential issues for seismic actions and SC walls’ design. Epackachi et al. [18] modeled SC walls under cyclic loading in Ls-Dyna to simulate the nonlinear cyclic response of the flexure-critical of SC walls. Ls-Dyna models had a good agreement with experimental results. Booth [16] demonstrated a systematic estimation of the seismic act and design of a unique premier protection structure containing SC walls designed for an ordinary pressurized water reactor. 3D nonlinear inelastic finite element performed, which could be used to predict design force requirements. Polat [23] simulated the plastic cycle test of a double-SC wall and measured the wall's initial stiffness and bearing capacity for each cycle. A good agreement between experimental and numerical results was obtained in terms of the wall's shear force and the joints, the cumulative plastic strain, and the wall's plasticity. Elmatzoglou [17] was developed FEA for the SC wall. The study showed that the shear strength of the SC wall increases when the aspect ratio reduces. Nguyen [20] simulated a straight SC wall in ABAQUS (explicit). The results are very reliable with the experimental results. The study showed that if the faceplate slenderness ratio is less than 32, there is no significant impact on the peak shearing force.

Considering the geometry of SC walls, it was observed that most of the studies were carried out using straight walls. Therefore, curved SC wall standardization is still in progress, and it is not possible to directly apply the design method of the straight wall to curved SC walls. In conclusion, to expand the use of curved SC walls that can provide higher bearing capacity than straight design, it is necessary to analyze the mechanical properties of the curved SC wall and determine the effect of the radius-thickness ratio. AISC Committee on Specifications [26] also recommends performing experimental and numerical investigations on curved SC walls. It introduces an assumption about the behavior of curved SC walls. According to this assumption, if the radius-thickness ratio is greater than 20, the effect of curvature is negligible, and another ratio rates should be investigated. This is only available information in the literature about the effect of curvature on SC walls' performance. Therefore, this paper aims to contribute to determining the effects of different curvature values under static and cyclic loading of SC walls. For that purpose, several numerical tests were conducted under cyclic loading. Then, monotonic in-plane and out-of-plane responses were calculated. Numerical results of seven types of curved SC walls with different radius thickness ratios were discussed in detail.

2. Numerical modeling and model validation

2.1. Brief introduction of the test
In this study, experimental results and material properties were obtained from the studies of Tian et al. [27] and Peiyao et al. [28]. Tests were conducted in a quasi-static regime according to “building seismic test method procedures” (JGJ101-96) [31]. SCW1-6 specimen was designed and processed for in-plane mechanical performance testing. The design strength of concrete (C35), the steel plate (Q345), and the steel bar (HRB335). The design axial compression ratio was determined as 0.4 [28]. The elevation views and the parameters of SCW1-6 are shown in Figure 1 and Table 1, respectively.
Table 1. Parameters of SCW1-6

| Specimen | Length (mm) | Height (mm) | Wall Thickness (mm) | Steel-plate thickness (mm) | Stud spacing (mm) | Shear span ratio |
|----------|-------------|-------------|---------------------|---------------------------|------------------|-----------------|
| SCW1-6   | 1000        | 1000        | 150                 | 3                         | 80               | 1               |

2.2. Numerical modeling of the tested specimen

2.2.1. Concrete material model.
Concrete plastic damage model was used in SC wall, foundation, and loading beam. The loading beam and foundation were modeled using the same material properties. The tensile and compression stress-strain relations were identified using equation (1) and equation (2), respectively. Tensile stress-strain relation of the concrete in each part of the specimen adopts the same tensile constitutive relation, and the stress-strain relation is given as follows [27]:

\[
\sigma = E \varepsilon \quad \varepsilon \leq \varepsilon_t
\]

\[
\sigma = \left[1 - \frac{0.9(\varepsilon - \varepsilon_t)}{0.001 - \varepsilon_t}\right] f_t \quad \varepsilon < 0.001
\]

\[
\sigma = 0.1 f_t \quad \varepsilon > 0.001
\]

On the other hand, compression stress-strain relation of the concrete in each part of the specimen adopts the same tensile constitutive structure. For the concrete at main wall and the hidden column, constrained constitutive structure is adopted [30], and the stress-strain relation is shown in equation (2).

\[
y = 2x - x^2 \quad (x \leq 1)
\]

\[
y = \frac{x}{\beta(x - 1)^\gamma + x} \quad (x > 1)
\]

Where;
\[ x = \frac{\varepsilon}{\varepsilon_0} \quad \text{and} \quad y = \frac{\sigma}{\sigma_0} \]

\[ \sigma_0 = [1 + (-0.0135 \cdot \xi^2 + 0.1 \cdot \xi) \cdot \left( \frac{f'_c}{f_c} \right)^{0.45}] \cdot f'_c \]

\[ \varepsilon_0 = \varepsilon_{cc} + [1330 + 760 \cdot \left( \frac{f'_c}{f_c} - 1 \right)] \cdot \xi^{0.2} \]

\[ \varepsilon_{cc} = 1300 + 12.5 \cdot f'_c \]

\[ \eta = (1.6 + 1.5) x^{-1} \]

\[ \beta = \begin{cases} (f'_c)^{0.1} & (\xi \leq 3.0) \\ \\ \\ \\ 1.35(1 + \xi)^{1/2} & (\xi > 3.0) \end{cases} \]

\[ \xi = \frac{A_f}{A_c} \frac{f'_c}{f_c} \]

Where, \( f_{cu} \) is the compressive strength of concrete cube \( f'_c \) is the compressive strength of the concrete cylinder axis \( f_{cu} \) is the compressive strength of concrete prism \( \xi \) is the constraint effect coefficient.

\[ y = \alpha_x x + (3 - 2\alpha_x)x^2 + (\alpha_x - 2)x^3 \quad (x \leq 1) \]

\[ y = \frac{x}{\alpha_d (x - 1)^2 + x} \quad (x > 1) \]

Type in the:

\[ x = \frac{\varepsilon}{\varepsilon_c} \quad \text{and} \quad y = \frac{\sigma}{f_{ck}} \]

Where, \( \alpha_x \) and \( \alpha_d \) are the ascending segment. Damage factor \( d_x, d_c \) reflects the tension of concrete, stiffness degradation due to damage during compression. The damage factor is calculated by the energy equivalence principle in equation (4) and (5) [27].

\[ d_x = 1 - \left( \frac{\sigma}{E_c \varepsilon} \right)^{1/2} = 1 - \left( \frac{f_c}{E_c \varepsilon_x} \frac{y}{x} \right)^{1/2} \]

\[ d_c = 1 - \left( \frac{\sigma}{E_c \varepsilon} \right)^{1/2} = 1 - \left( \frac{f_{ck}}{E_c \varepsilon_c} \frac{y}{x} \right)^{1/2} \]

### 2.2.2. Steel material model.

The steel was modeled as a two-fold ideal elastoplastic model, the stress-strain relationship is shown in equation (6). According to the modeling assumption, only the peak load of SCW1-6 wall was
determined. Fracture displacement cannot be found due to the steel material model employed in this study [27].

\[
\sigma_s = E_s \varepsilon_s \quad 0 \leq \varepsilon_s \leq \varepsilon_{sy} \\
\sigma_s = f_y \quad \varepsilon_s > \varepsilon_{sy}
\]  

(6)

2.2.3. Studs.
The stud’s shear and tensile resistance were simulated by a set of spring elements in three directions, namely a normal spring to simulate the stud’s axial resistance and two tangential springs in two perpendicular directions to simulate the shear resistance of the stud. The tangential spring is nonlinear spring. Its stiffness calculated from the shear displacement relation of the stud in equation (7) [28]. The selection of spring stiffness is as follows.

\[
V = V_u \left( 1 - e^{-\frac{u}{2}} \right)^{0.425} \\
V_u = \min \left\{ 0.8 f_u A, 0.29 \alpha d^2 \left( f_{ad} E_v \right)^{1/2} \right\}
\]  

(7)

in which, \( V_u \) is the ultimate shear bearing capacity of studs. \( S \) is slip (mm), \( f_u \) is the final tensile strength of the stud, \( A \) is the sectional area of the stud, \( d \) is the diameter of the stud, and \( h \) is the length of the stud. The normal spring is linear spring, whose stiffness is calculated by equation (8) [28].

\[
K = \frac{E_{stud} A}{h}
\]  

(8)

2.3. Model validation

When the FEA and experimental results were compared it was observed that there is a slight error (8.4%) in simulation result in terms of the peak load of SCW1-6. According to FEA, the peak load of SCW1-6 was 1694 kN. On the other hand, the test result of SCW1-6 was 1552 kN. The main reason for this error was the modeling assumption. In the ABAQUS model, the foundation beam was assumed as fixed and defined as ENCASTRE, thus foundation response was neglected. Therefore, the obtained error can be considered as acceptable. The comparison between the load-displacement curve calculated by FEA and the test’s hysteresis curve is shown in Figure 3 (a). It can be said that the calculated curve and the test curve are in good agreement.

![Figure 2. a. FEA of SCW1-6, b. SCW1-6 test](image-url)
3. Finite element analysis model of curved SC wall

3.1. Design of curved SC wall

Figure 4 gives the variations of the dimensions of curved walls. To compare the performance of the curved walls with the straight wall, the height (1000 mm) and the chord length (1000 mm) values were determined the same with straight one. On the other hand, seven curved SC walls designed with consideration of ratio:5,10,15,20,25,30 and 40. These values are determined according to limit value of 20 which is given in AISC Committee on Specifications [26]. There is an inverse proportion between the R/t ratio and angle of curved SC walls (Figure 4 (b)).

Table 2 shows the geometric properties of curved SC walls used in FEA analyses. CSC-5 to CSC-
40 all models created in ABAQUS/Standard. Foundation beam and loading beam were modeled the same as SCW1-6.

### Table 2. Parameters of curved SC walls

| Model | Length (mm) | Height (mm) | Wall Thickness (mm) | Steel-plate thickness (mm) | Radius (mm) | R/t ratio |
|-------|-------------|-------------|---------------------|---------------------------|-------------|-----------|
| CSC-5 | 1000        | 1000        | 150                 | 3                         | 750         | 5         |
| CSC-10| 1000        | 1000        | 150                 | 3                         | 1500        | 10        |
| CSC-15| 1000        | 1000        | 150                 | 3                         | 2250        | 15        |
| CSC-20| 1000        | 1000        | 150                 | 3                         | 3000        | 20        |
| CSC-25| 1000        | 1000        | 150                 | 3                         | 3750        | 25        |
| CSC-30| 1000        | 1000        | 150                 | 3                         | 4500        | 30        |
| CSC-40| 1000        | 1000        | 150                 | 3                         | 6000        | 40        |

Studs are defined as 90 degrees to concrete and steel plate. Each curved wall stud’s location changed due to the total angle of the curved SC wall. All models have different angles, and it causes differences for modeling of studs. Studs are defined as the same distance as a straight wall. The ratio of the stud’s location angle was calculated symmetrically. Therefore, studs should be in the same direction with partition degrees. The intersection points of studs are assumed considering the curvature angles. Vertical spacing of the studs was designed as 82.5mm, equal to degree partition of the curved wall. Horizontal spacing was designed up to 90mm.

#### 3.2. Numerical modeling of curved SC wall

For curved walls, the material constitutive relation selection, element type, contact relation, and other aspects are the same as those of straight walls. Curved SC walls model’s geometry was created in ABAQUS 6.13. Loading beam, foundation, and the curved wall are defined as C3D8R (8-linear brick, reduced integration) elements. These solid elements merged at the assembly option. T3D2 (A2-node 3D Truss) and S4R elements are used for Rebars and Steel Plate [27,28]. All three structural members, curved walls, loading beam, and foundation beam merged in the ASSEMBLY option. Thus, the concrete part of curved SC walls acts as together. Steel-plate was modeled with the same parameters as the concrete wall. Partition webs are designed as a constant ratio proportion according to the straight wall.

![Figure 5](image)

**Figure 5.** a. Concrete, b. Steel-plate, c. Stud spacing, d. Loading

To validate straight SC wall modeling assumption, Surface to Surface Contact (Standard) with excluding shell/membrane elements thickness option defined. The friction constant between concrete and steel plate are assumed as 0.25. Friction formulation and overclose pressure options are respectively Penalty and ‘Hard’ Contact. However, in case of curved walls, friction between concrete and steel plate was neglected due to there is no significant effect on in-plane response [20]. Studs were simulated as three-way zero-length spring elements that are chosen according to design [28]. Rebars and Steel-Plate were embedded in concrete. The partition option was used for meshing since of the spring locations.
Stiffener separated according to degrees in the angles divided by straight walls spacing ratio, considering as total angle as 1000 mm. FEM simulation has two steps. In the first step, uniformly distributed concentrated force (1664 kN) was applied through the direction of the center of gravity, which is represented as the point of N (Figure 5(d)). In the second step, a cyclic lateral displacement of 25mm was determined through the center of the shear point on the curved wall. For in-plane monotonic response, 25mm one-way horizontal displacement is defined through P1. For out-of-plane monotonic response, 40mm lateral displacement is defined in the P2 direction.

4. Analysis results and discussion of the curved SC wall

4.1. Global force-displacement response
The hysteresis curve is an important indicator to judge the energy dissipation ability of the specimen. The fuller the shape means the stronger the energy dissipation ability and the better the seismic performance. The comparison between the load-displacement curve calculated by FEA and the tests' hysteresis curve is shown in Figure 6. It is clear from Figure 6 that when considering cyclic loading conditions, the dynamic behavior of calculated CSC walls and SCW1-6 wall curves are in good agreement.

![Figure 6. Comparison of CSC-5 to CSC40 and SCW1-6-FEA global force-displacement response](image)

4.2. Backbone curves
The backbone curve of each curved SC wall is shown in Figure 7. Figure 7 shows that the curved SC wall with an R/t:5 ratio has better shear deformation resistance. The peak shearing forces calculated by the FEA models are (1895/1790/1804/1775/1790/1786/1768 kN) for CSC-5 through CSC-40, respectively. CSC-5 has 11.87% better seismic performance than SCW1-6 FEA.
Figure 7. Backbone curve comparison between SCW1-6 FEA and CSC-5 to CSC-40 a. CSC-5, b. CSC-10, c. CSC-15, d. CSC-20, e. CSC-25, f. CSC-30, g. CSC-40

Figure 9 shows the average lateral load values of the push and pull directions. While R/t ratio gets smaller, the lateral resistance and displacement angle gets bigger which provides relatively strong deformation capacity (Figure 8) [31].

Figure 8. Lateral load and displacement results of all the models
4.3. Damage to SC walls
In the initial loading stage, the displacement-load of the CSC walls is linear, and there is no obvious sign of failure in the appearance of CSC walls. When the horizontal deformation gradually increases, the corner steel plate buckles, the concrete on the compression side of the basic specimen is compressed, the steel plate buckles outward, and weld tearing occurs at the root the steel plates on the tension side are pulled out to varying degrees, the base concrete is destroyed. The failure modes are all typical bending failures. The largest stresses are mainly concentrated on the root of the wall's compression side, which is consistent with the bending failure mode of the specimen in the test. Yield occurs on both the compression side and the tension side of the steel plate.

When the peak loads, damages of wall and steel yields are checked, it can see that there is a consistent decrease in peak loads (Figure 9). While the R/t ratio increased, peak loads and damage of wall loads are slowly decreased. Although there is a consistent decrease in peak and damage loads, steel yield yielding behavior is unpredictable.

Table 3. Numerical result of FEA models

| Model  | R/t ratio | Peak Load (kN) | Steel Yield (kN) | Damage of Wall (kN) | The increase in peak load |
|--------|-----------|----------------|------------------|---------------------|--------------------------|
| CSC-5  | 5         | 1895           | 1229             | 1610                | 11.87%                   |
| CSC-10 | 10        | 1790           | 1069             | 1521                | 5.67%                    |
| CSC-15 | 15        | 1804           | 1058             | 1533                | 6.49%                    |
| CSC-20 | 20        | 1775           | 1022             | 1508                | 4.78%                    |
| CSC-25 | 25        | 1790           | 975              | 1521                | 5.67%                    |
| CSC-30 | 30        | 1786           | 1044             | 1518                | 5.43%                    |
| CSC-40 | 40        | 1768           | 1051             | 1502                | 4.37%                    |
| SCW1-6 FEM | -       | 1694           | 936              | 1439                | 0.00%                    |

Figure 9. Load and R/t ratio relationship

4.4. In-plane and out-of-plane monotonic response
Considering the convergence difficulty of low cyclic loading in FEA, to better reveal the working mechanism of curved walls under in-plane and out-plane loads, FEA under monotone loads were carried out, respectively. In the process of horizontal reciprocating loading, once the specimen becomes plastic, as the load continues to increase, the rigidity of the SCW1-6 and all the CSC walls continue to decrease. The load-displacement curve can express the attenuation process of the specimen stiffness. See Figure 10.a. SCW1-6 FEA of in-plane monotonic response is calculated 1535 kN. All the curved walls’ ultimate
lateral load responses are bigger than SCW1-6 FEA. The peak in-plane monotonic response calculated by the FEA models are (1744/1674/1660/1656/1666/1665 kN) for CSC-5 through CSC-40, respectively. As a result, it can seem that even a small curvature effect has a positive impact on in-plane response. CSC-5 is a 13.6% higher monotonic response than SCW1-6 FEA. Although CSC-40 behaves like a straight wall under cyclic loading, the CSC-40 model’s behavior changes like curved walls under in-plane monotonic response.

Figure 10. a. In-plane monotonic response, b. Out-of-plane monotonic response

The out-of-plane monotonic response was investigated for predicting the bearing capacity of curved SC walls. SCW1-6 FEA of out-of-plane monotonic response is calculated as 336 kN. The peak out-of-plane monotonic response calculated by the FEA models are (455/347/334/325/322/321/317 kN) for CSC-5 through CSC-40, respectively. All these results show that if the CSC wall’s R/t ratio is smaller than 10, CSC walls bearing capacity increases (Figure 10.b.). CSC-5 has a higher bearing capacity than SCW1-6 FEA by 35.4%.

5. Conclusion
In this study, the curvature design on the mechanical behavior of SC walls was investigated numerically. The determination of the dynamic behavior of curved SC walls is the novel aspect of the study. According to obtained global force-displacement graphs, the R/t ratio:5 has the biggest peak load and steel yield load. In cyclic loading, the CSC-5 model has the best seismic performance and the stronger the energy dissipation ability. The radius-thickness ratio greater than 10, the effects of curvature are negligible. R/t ratio:5 model is the only model which is observed different behavior than other models. If the R/t ratio greater than 20, there is no curvature effect. Monotonic response behavior shows that if the R/t ratio greater than 20, there is no effect in the monotonic response. Numerical analysis shows that radius thickness ratio:5 is the optimal curved SC wall when it is compared with CSC-10, CSC-15, CSC-20, CSC-25, CSC-30, and CSC-40. Investigation of bearing capacity proves that if R/t ≥ 10, there is no effect in out of plane response. According to FEA, R/t ratio:5 is the optimal curved SC wall compared with CSC-10, CSC-15, CSC-20, CSC-25, CSC-30, and CSC-40 in out of plane response. The calculated results show that the modeling assumption of SCW1-6 has a good agreement with the test results. ABAQUS can calculate the in-plane seismic response of SC walls. In addition, interaction between studs and steel-plates were simulated as three-way zero-length spring elements. It was proved that this method has good performance to predict contact interactions. As a future study, concrete plastic damage model and two-fold ideal elastoplastic model can be used to further improve the numerical findings.

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