MECHANICAL BEHAVIOR OF CFRP STRENGTHENED SHORT RC WALL SUBJECTED TO MONOTONIC AND CYCLIC LOAD TEST

S. Qazi[1*], E. Ferrier[2], L. Michel[2]

[1] *LGCIE, Site Bohr, INSA Lyon / 82 boulevard Niels Bohr - DOUA - 69622 VILLEURBANNE Cedex*
[2] *LGCIE, Site Bohr, UCBL 1 / 82 boulevard Niels Bohr - DOUA - 69622 VILLEURBANNE Cedex*
**engrsami@gmail.com*

Keywords: RC wall, CFRP Strengthening, load sustained, and energy dissipation.

Abstract

Recent earthquake surveys have revealed the significance of RC walls as an integral part of structures. It reduces the structure damage to some extent. Researchers have concluded that the RC wall buildings sustained damage, mainly due to design and construction work flaws. In this article experimental result of five RC short shear walls is discussed. They were designed under-reinforced to fail in shear. Three out of these five specimens were strengthened externally with CFRP strips bonded to wall panel and mesh anchors installed at wall foundation joint. Two specimens, one RC and one CFRP retrofitted, were subjected to static load test and three specimens, one RC and two CFRP retrofitted, were subjected to cyclic load tests. The test result analysis discussion includes cracking pattern, stiffness, ultimate load capacity, ductility, and energy dissipation.

1 Introduction

In recent years, reinforced concrete walls are considered as an integral part of building structures. Post earthquake surveys have highlighted their significance in confining the earthquake induced damage. The dissipation capacity of RC wall depends directly on its transverse dimension [1] i.e. doubling RC wall thickness doubles its energy absorption capacity. Though reinforced concrete walls are used in building to dissipate seismic induced energy they too are vulnerable to seismic damage. The main causes of damage are: occurrence of unpredictable high seismic activity, improper designing and construction flaws [2],[3], & [4]. RC wall load response behavior depends to a great extent on its height to length ratio. The RC Wall which has H/L ratio lesser than 2 is considered as short and the wall that has H/L ratio greater than 2 is considered as slender/long wall [5]. Short wall endure high shear stress as compared to slender wall. The failure modes of short wall are sliding apart of wall from it supporting foundation, diagonal shear cracks development with in wall and concrete crushing at wall toe [5],[7],[8],[9],[10],&[11]. This study investigates the influence of external CFRP reinforcement on load response behavior of short RC. In total five RC walls, designed under-reinforced to fail in shear, were fabricated. Three out of these five specimens were afterward strengthened with CFRP. Test results discussion includes load displacement curve, failure modes, stiffness and dissipated energy.

2 Experimental Program

1.1 Specimen Detail

The basic idea of short RC wall geometry was derived from Greifenhagen [11] research work. The specimen represents at a 1:3 scale the lower part of an existing building. The RC wall details are shown in Figure 1.

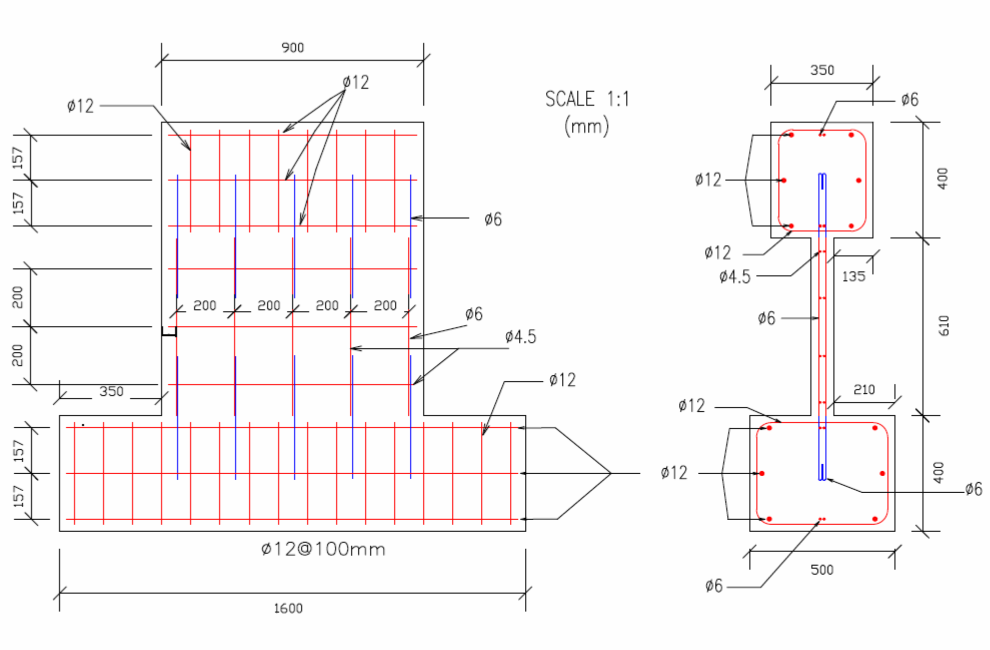

Figure 1. RC Short wall geometric and reinforcement details.

According to Eurocode ENV 1992-1-1, Ref. 5.4.7.2.1 the minimum reinforcement ratio is 0.4%. However in this research work specimens were fabricated under-reinforced. Specimens' vertical and horizontal reinforcement ratio was kept equal to 0.3% and 0.2% respectively and a clear cover equivalent to 2 cm was kept. The test specimen head and foundation blocks were first fabricated from concrete with compressive strength of 40.6 ± 0.4 MPa. These were then cured for 28 days and later on aligned on floor with wall panel mold to fabricate it. The wall panels were then cured for 28 days. Afterward both sides of each wall were made smooth by sandblasting and CFRP strengthening was applied to four specimens.

1.2 External CFRP reinforcement

The choice of a reinforcement pattern is always difficult due to some antagonistic parameters (e.g., maximum load and energy dissipation). The RC walls with no external CFRP reinforcement arrangements are labeled as S1 and S3 and other three re-strengthened with CFRP are labeled as SR2, SR4, and SR6. The epoxy resin used had a tensile strength, modulus, ultimate strain and glass transition temperature of 55 MPa, 3200 MPa, 2.3% and 56°C, respectively. The CFRP fabric used had a tensile strength of 825 MPa, tensile modulus equivalent to 70.5 MPa and an ultimate strain of 0.85%. The CFRP reinforcement arrangement applied on wall both faces are shown in Figure 2. The mesh anchor installed at wall foundation joint had cross sectional area equivalent to 80 mm². The mesh anchors were made up of CFRP fiber tow, by winding it around two nails that were fixed apart a distance equals to 80 cm. The winded fiber tow was afterward released from nails and folded in the middle. At folded end a CFRP rod or a steel wire was attached to ease mesh anchor insertion in hole. At the non folded end the fiber tows' looped portion was cut to splay this end. The tensile strength of a mesh anchor of 26 fiber tows was 22 kN. Each anchor was embedded in drilled hole made in foundation block up to a length of 150 mm (5.90 in.) and its remaining length was splayed over the vertically bonded FRP strips. This was then over-bonded by a CFRP strip splayed in direction perpendicular to wall vertical axis. Table 1 summarizes the CFRP reinforcement arrangement made on test specimen. In case of specimen SR6, to limit CFRP strips debonding with in wall panel, mesh anchors were placed in holes, which were drilled within wall panel at CFRP strips intersection point.

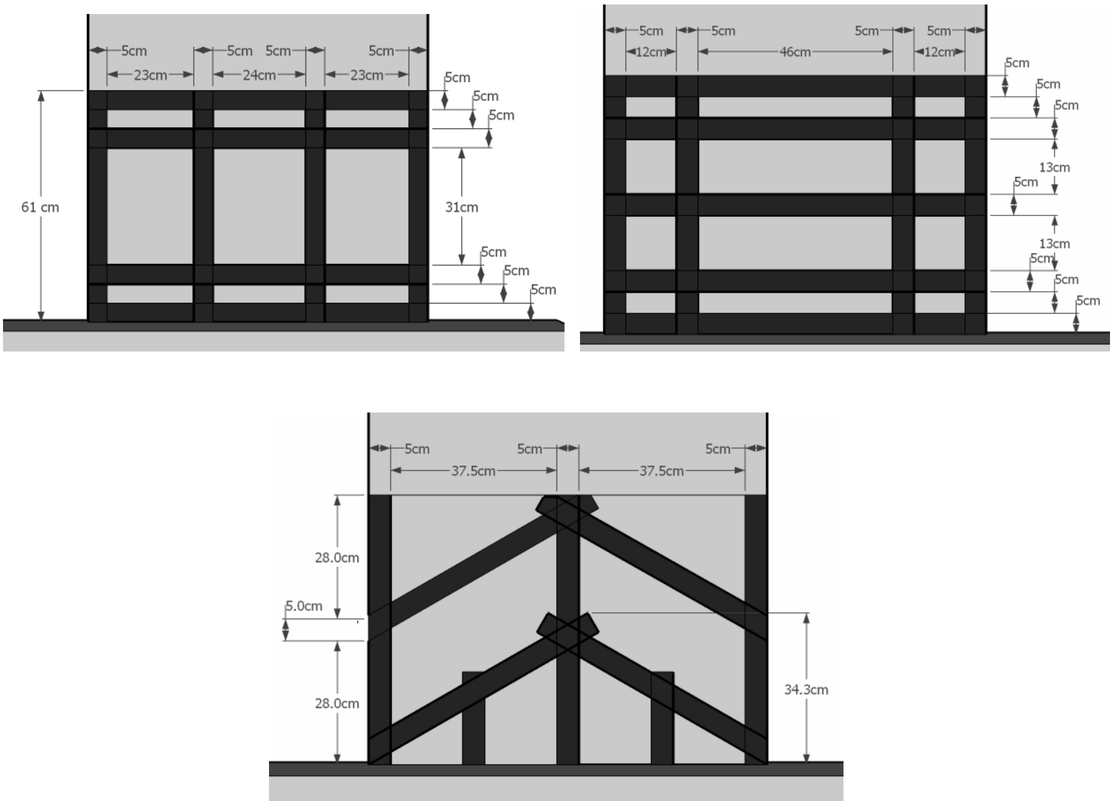

Figure 2. CFRP reinforcement schematic detail.

Specimen	Type	CFRP Strip Width (mm)	Number of fibre tows in each anchor: Wall Foundation	Wall Panel
S1	-	-	-	-
SR2	Bidirectional	50	26	-
S3	-	-	-	-
SR4	Bidirectional	50	44	-
SR6	Unidirectional	50	44	12

Table 1. External CFRP reinforcement detail

1.3 Test setup

The RC walls test setup is depicted in Figure 3. Test specimens were subjected to displacement control lateral loading, with the wall acting as cantilevers. In all specimens a constant axial compression load was applied over head beam. The axial load ratio, applied axial load to axial load capacity at concrete section, has a significant influence on shear wall performance, deformability and failure modes [12],[13]. The axial load ratio used in earlier research work for wall test based on specimen model and material properties ranged from

0.03 to 0.85 [13],[14],[15],&[16]. All specimens were subjected to a constant axial compression load equivalent to 110 kN. In regard of lateral loading, the first two specimens of each type (S1 and SR2) were subjected to quasi static loading to find out specimen performance. In this case the lateral displacement was provided at a speed of 0.01 mm/sec. Four LVDT's were placed along wall height at its free end to check its deflection pattern. One LVDT was positioned at the center of head beam, second at wall panel top, third in vicinity of wall panel bottom and fourth one at center of foundation block to measure the wall foundation slipping. The foundation block slipping value recorded was deducted from displacement value measured at the wall head to calculate the actual value of induced displacement (horizontal).

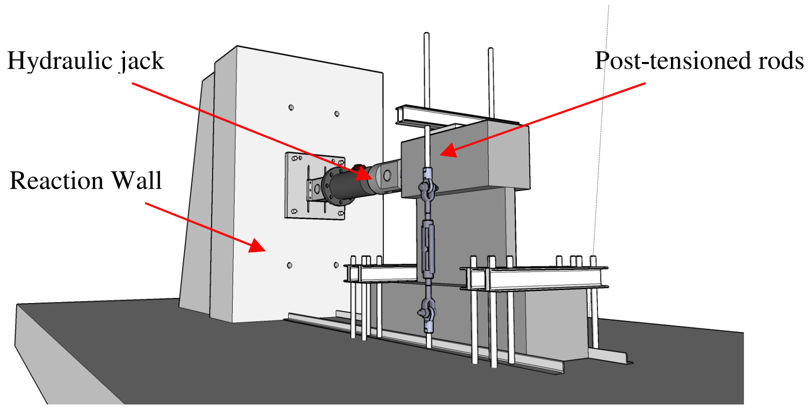

Figure 3. Test setup.

To simulate seismic actions specimens S3, SR4, and SR6 were subjected to reverse static cyclic load test. According to recommendation of ACI (ACI T1.101, 2001), specimens were subjected to three full cycles at each level. Due to variation made in external CFRP reinforcement configurations of all specimens, the lateral displacement load levels were based on drift instead of ductility to allow easy comparison. The drifts were 0.1%, 0.2%… 0.8%, 1%… 1.8%. In this case too a constant axial compression load of 110 kN was sustained at head beam.

2 Test results

2.1 Load displacement curve

Figure 4 shows the load displacement curve of specimens S1 and SR2. The initial stiffness of the two specimens is almost identical because the CFRP reinforcement initiates load contribution with concrete cracking or bar yields. Therefore the two specimens exhibited alike behavior till the load level of 105 kN and 0.8 mm lateral displacement. Beyond this the CFRP reinforcement contribution is evident in load distribution curve of the two specimens. Specimen S1 exhibit plastic deformation as it exceeds the 3 mm limit while on the other hand specimen SR2 exhibit non linear elastic behavior till failure. At the point of plastic yielding in specimen S1, the load sustained by specimens S1 and SR2 were 150 kN and 174 kN, respectively. This gain in strength of 24 kN is owed to CFRP external reinforcement. The specimen S1 demonstrated an ultimate strength of 158 kN at induced displacement level of 4.2 mm and specimen SR2 established an overall improvement in strength and ductility by exhibiting an ultimate strength of 219 kN at a displacement load level of 7.43 mm. Therefore, CFRP external reinforcement improved RC short wall specimen strength and ultimate displacement by 38% and 55%, respectively.

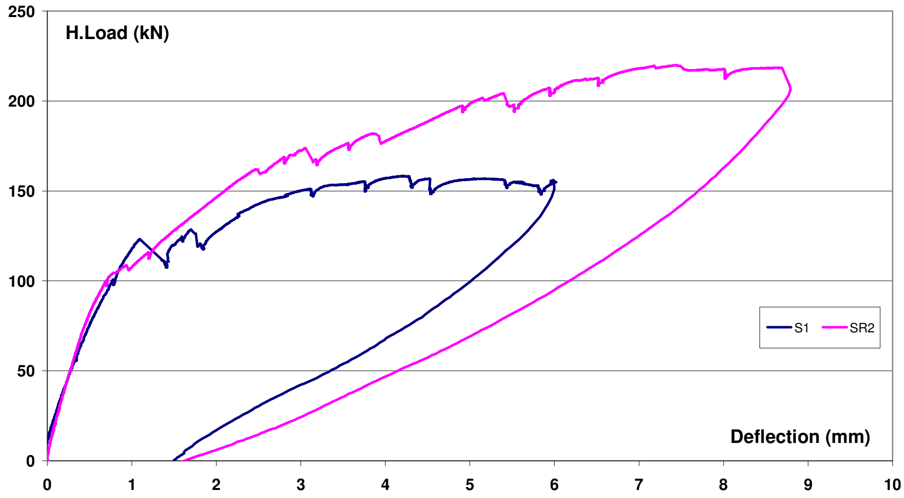

Figure 4. Load Displacement curve of specimen S1 & SR2.

2.2 Failure mode

Figure 5 show photos of failed specimen S1 and SR2 subjected to monotonic load test. Specimen S1 showed a typical under-reinforced RC short wall failure mode. The specimen failure occurred due to development of diagonal cracks with in wall panel, wall foundation relative slip and extreme rebar yield. It is important to note that sliding failure occurred at construction joint. It is important to note that these entire cracks were not superficial as they developed on wall both faces simultaneously. The specimen SR2 retrofitted with CFRP, not only exhibited an increase in strength and ductility, but also limited crack propagation with in wall panel. At wall foundation joint the bonded CFRP strips hindered the visualization of crack propagation.

Figure 5. Specimen S1 & SR2 crack pattern at test end.

2.3 Specimen load response behavior under cyclic load

The hysteresis envelopes of specimens S3, SR4 and SR6 are shown in Figure 6. The ultimate displacements observed in these specimens were 7.43, 9.81 and 12.1 mm, respectively. The relative increase in initial stiffness due to introduction of CFRP reinforcement in case of SR3 and after ward additional improvement owed to transverse mesh anchor with in SR6 wall panel is well depicted by envelope curves relative incline in initial slope.

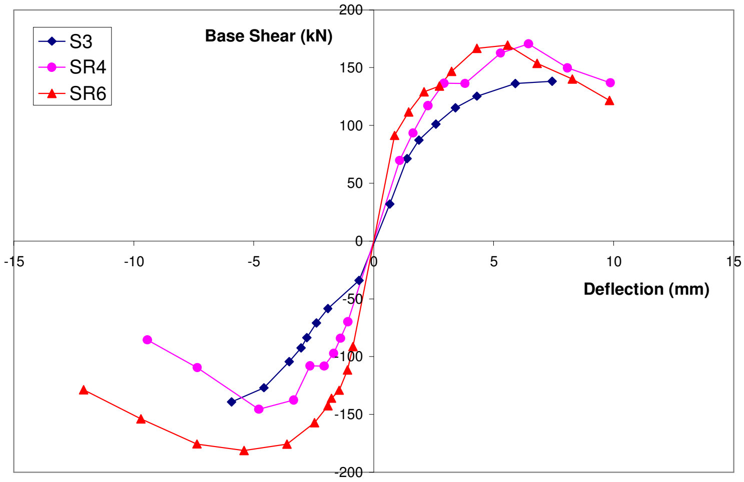

Figure 6. Hysteresis envelope curves.

Figure 7 and 8 shows the EE and DE curves of these three specimens, respectively. The curves values are obtained by summation of EE and DE for three complete cycles at each load level. The curves shows that the CFRP reinforcement arrangement made in case of SR4 did improved its elastic energy compared to specimen S3 but on the other hand reduced its dissipation capacity. This development is attributed to CFRP material elastic nature. In case of specimen SR6, the additional transverse anchor arrangement made within wall panel improved both its elasticity and dissipation capacity as compared to SR4 and S3. At induced drift level of 0.8 %, the cumulated E.E within specimen S3, SR4 and SR6 was 1.4, 1.7 and 2.06 kJ, respectively. Thus SR4 and SR6 CFRP reinforcement arrangement improved specimen elasticity up to 21.4% and 47% at 0.8% drift. The cumulated D.E within specimen S3, SR4 and SR6 at 0.8 % drift was 1.59, 1.23 and 1.68 kJ, respectively. Specimen SR4 depicted a decline in DE of 22.6 % and SR6 exhibited an improvement of 5.6 % in comparison to control specimen S3. Specimen SR6 exhibited superior performance in comparison to SR4. At drift level of 1.2 %, specimen SR6 showed a relative increase in E.E and D.E to that of specimen SR4. These were 23% and 45 %, respectively. This improvement in behavior was attributed to two modifications, (a) transverse mesh anchor: kept the CFRP strip intact with concrete surface and therefore improved wall elasticity (b) diagonally bonded CFRP strips: tended to bridge the cracks oriented in direction transverse to it and limited their widening apart. These arrangements kept the cracked surfaces in contact and dissipated induced energy by friction due to relative slipping.

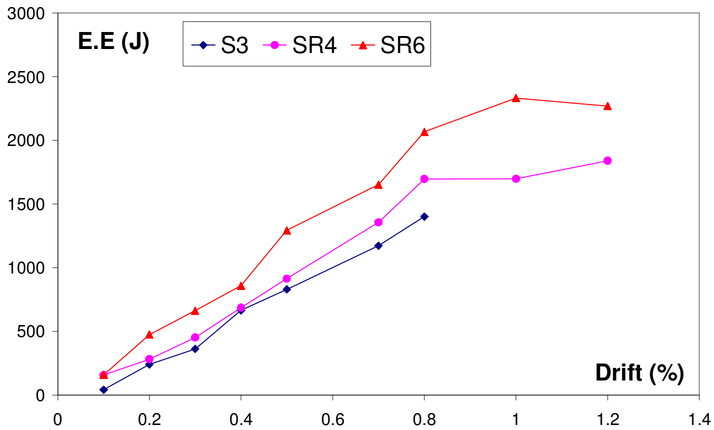

Figure 7. S3, SR4 and SR6: E.E curves.

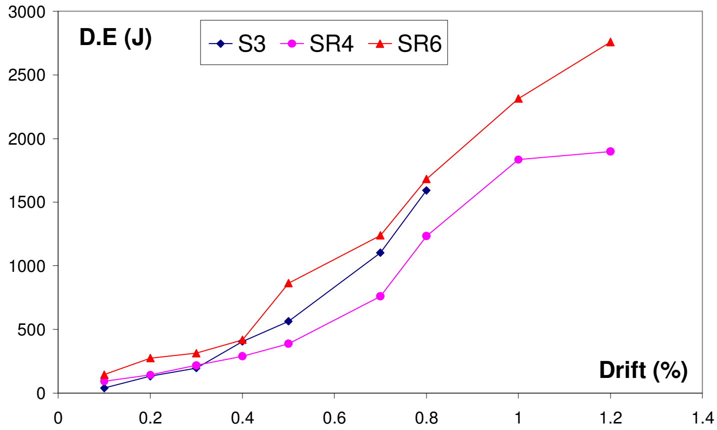

Figure 8. S3, SR4 and SR6: D.E curves.

Conclusion

This research work highlights the positive influence of external CFRP reinforcement on RC short wall. The CFRP strips bonded to RC wall panel did improve their ultimate load capacity, ductility, and limited the crack propagation to a certain extent. The mesh anchor placement at wall foundation joint remedied the joint failure due to improper reinforcement arrangement in this region. They also limited CFRP strips debonding problem, a major concern in case of utilization of external FRP reinforcement technique, by transferring load effects from the bonded strips to lower foundation block. The partial FRP strengthening adopted here proved to be successful as it did not deteriorated the RC wall energy dissipation capacity. RC structural elements dissipate induced energy as a result of friction in between concrete cracks and rebar yielding. This arrangement ensured concrete cracking within wall panel to some extent which in turn resulted in energy dissipation.

Acknowledgments

Authors would like to thank the French National Research Agency (ANR) for financial support for this research program called INPERMISE.

References

[1] Zacek Milan. *Construire parasismique; risque sismique, conception parasismique des bâtiments: réglementation*. Editions Parenthèses, Marseille (1996).
[2] Inoue N., Yang K., Shibata A. Dynamic nonlinear analysis of reinforced concrete shear wall by finite element method with explicit analytical procedure. Earthquake Engineering Structural Dynamics, 26, pp. 967-986 (1997).
[3] Oh YH, Han SW, Lee LH. Effect of boundary element details on the seismic deformation capacity of structural walls. Earthquake Engineering Structural Dynamics, 31(8), pp. 1583-1602 (2002).
[4] Brun M., Reynouard J. M., Jezequel L., Ile N. Damaging potential of low-magnitude near-field earthquakes on low-rise shear walls. Soil Dynamics and Earthquake Engineering, 24(8), pp. 587-603 (2004).
[5] Brun M., Reynouard J. M., Jezequel L. A simple shear wall model taking into account stiffness degradation, Engineering Structures, 25(1), pp. 1-9 (2003).
[6] Fouré B. *Un programme d'essais des murs de contreventement* in AFPS-SECED Colloquium, Experimental Methods in Earthquake Engineering and Structural Dynamics, St Rémy-lès-Chevreuse, France (1993).

[7] Maier J, Thürlimann B. Bruchversuche an Stahlbetonscheiben. IBK Bericht 8003-1. Zürich, ETH (1985).
[8] Paulay T, Priestley MJN, Singe AJ. Ductility in earthquake resisting squat shear walls. ACI Journal, 79(4), pp. 257-269 (1982).
[9] Lefas I, Kotsovos M, Ambraseys N. Behavior of reinforced concrete structural walls: strength, deformation characteristics, and failure mechanisms. ACI Structural Journal, 87(1), pp. 23–31(1990).
[10] Nicolae ILE. Contribution a la compréhension du fonctionnement des voiles en béton arme sous sollicitation sismique : Apport de l'expérimentation et de la modélisation a la conception. Thesis INSA Lyon, France (2000).
[11] Greifenhagen C., Lestuzzi P. Static-cyclic tests on low reinforced concrete shear walls. Engineering Structures, 27/11, pp. 1703-1712 (2005).
[12] Lefas I, Kotsovos M, Ambraseys N. Behavior of reinforced concrete structural walls: strength, deformation characteristics, and failure mechanisms. ACI Structural Journal, 87(1), pp. 23–31(1990).
[13] Su R, and S Wong. Seismic behaviour of slender reinforced concrete shear walls under high axial load ratio. *Engineering Structures,* 29(8), pp. 1957-1965 (2007).
[14] Salonikios, Thomas N, Andreas J Kappos, Ioannis A Tegos, and Georgios G Penelis. Cyclic Load Behavior of Low-Slenderness Reinforced Concrete Walls: Design Basis and Test Results. *ACI Structural Journal*, 96, pp. 649-660 (2000).
[15] Young-Hun Oh, Sang Whan Han and LI-Hyung Lee. Effect of boundary element details on the seismic deformation capacity of structural walls. Earthquake Engineering Structural Dynamics, 31, pp. 1586-1602 (2002).
[16] Lestuzzi, P, and H Bachmann. "Displacement ductility and energy assessment from shaking table tests on RC structural walls." *Engineering Structures,* 29(8), pp. 1708-1721 (2007).