Experimental evaluation of reinforced concrete wall repaired with increased thickness

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Abstract

The limited experimental information regarding the seismic performance of repaired reinforced concrete (RC) structures has been a technical challenge to assess the performance of repaired buildings after strong earthquakes. The main objective of this research is to evaluate the seismic performance of RC walls repaired by increasing the thickness and shear strength. Two RC walls with unconfined boundaries and aspect ratios of 1.75 and 2.5 were built. The walls were subjected to an axial load ratio of 0.1 and a pseudo-static lateral displacement protocol until failure. The damaged walls were repaired by adding additional web reinforcement and increasing the thickness. The bars that suffered buckling or fracture in the first tests were replaced with the new bars that were spliced with mechanical couplers. The repaired walls were tested with the same loading protocols like the one used in the first tests. The seismic capacity of the repaired walls is analyzed in terms of the maximum strength, deformation capacity, dissipated energy, and ductility. The test results showed that the repaired walls presented enhancement in the maximum strength and dissipated energy. Additionally, the failure mode of the repaired walls was different from that of the original walls. The obtained results from the reported tests are expected to be used to improve analytical methods to predict the seismic behavior of repaired walls.

Keywords: shear wall, repaired wall, damage, rocking

1. Introduction

Post-earthquake decisions regarding the repair or demolition of buildings that have suffered significant damage during severe earthquakes are in general governed by multiple factors including technical, legal and financial aspects. However, recent extremely contrasting experiences in Chile and New Zealand suggest that a robust technical and experimental framework must be developed to support this decision-making process which seriously affects the resilience of the communities. On the one hand, in Chile after the 2010 Maule earthquake, buildings with similar damage levels were repaired, and only 25% of the approximately 40 RC wall buildings (taller than 9 stories) with moderate to severe damage were demolished [1], [2]. On the other hand, most of the buildings located in Christchurch and damaged during 2011 earthquake were demolished, causing the closure of the central business district for more than 2 years [3], [4].

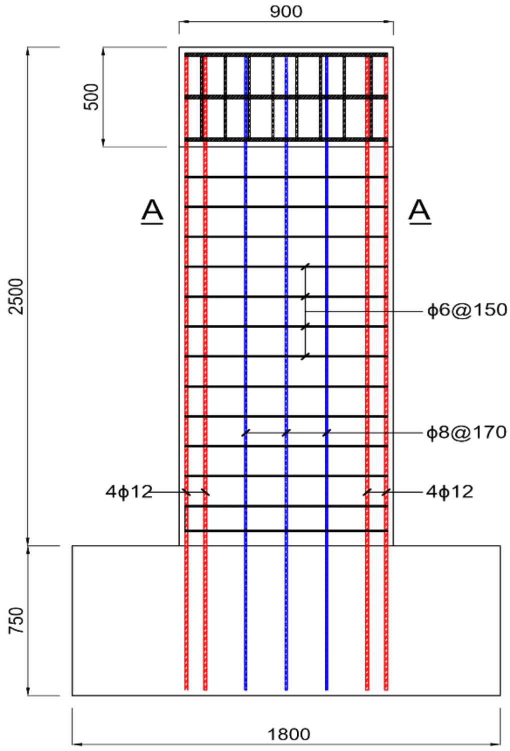
In (RC) buildings the standard seismic rehabilitation can be done with different techniques. The main techniques consider the addition of wall boundary members, addition of confinement jackets at wall boundaries, reduction of flexural strength, Increased shear strength of wall, confinement jackets to improve deformation capacity of coupling beams and columns supporting discontinuous shear walls and infilling between columns supporting discontinuous shear walls [5], [6]. These rehabilitation measures may or may not improve the lateral response of RC walls. The efficiency of the rehabilitation will depend on the previous level of damage, on the technique selected for rehabilitation and on the quality of rehabilitation.

In this research, the seismic capacity of repair RC shear walls, originally designed and constructed without boundary element, are evaluated in terms of the maximum strength, deformation capacity, dissipated energy, and ductility. The increase of the section wall [5] is the type of repair that is evaluated, due to this, measured was used in Chilean RC buildings repaired after 2010 earthquake Fig. 1. Two specimens are tested and evaluated, each specimen represent the typical shear RC walls built in Chile before the 2010 [1], and the results showed the efficiency or not of this type of repair on damaged walls after earthquakes.

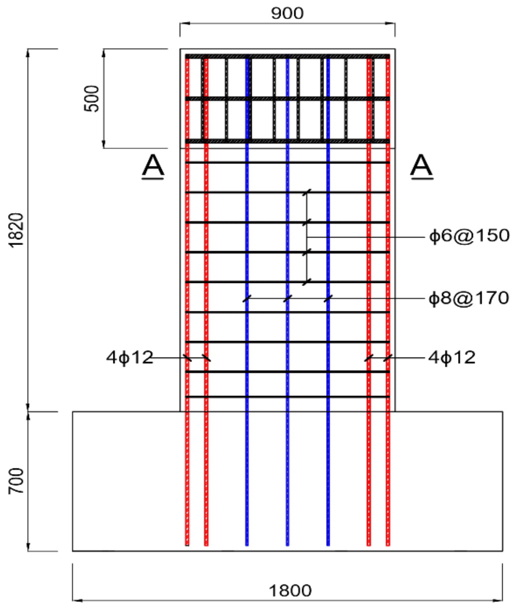
Fig. 1 – Wall repaired in Chile after the 2010 earthquake (EMB Construction Magazine).

2. Test Specimens

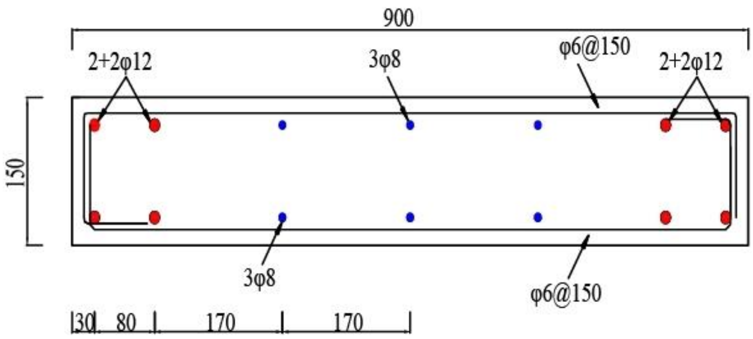
The two test specimens that were repaired and re-tested in this study were increasing the shear strength and confining the boundaries of the RC walls that were tested previously as part of a separate study [7]. The two initial specimens have identical wall cross-sections but different height and called W1 and W2, as shown in Fig. 2, the vertical boundary reinforcement is $4\,\phi\,12\,mm$ (12 mm diameter) bars ($\rho_b = 0.34\%$) and the distributed vertical reinforcement is ϕ 8 mm bars spaced at 170 mm in two layers ($\rho_l = 0.40\%$). The horizontal distributed reinforcement is ϕ 6 mm bars spaced at 150 mm($\rho_t = 0.38\%$). The steel selected for the horizontal reinforcement was different because of the available bar diameters, this give an s/db ratio of 12.5, where s is the spacing of the horizontal reinforcement and db is the diameter of the vertical bars.



Reinforcement in elevation of the W1



Reinforcement in elevation of the W2



Section A-A

Fig. 2 – Specimen details.

3. Materials

For the specimens without damage, six cylinder were tested, three to 28 days and three in specimen test day. The average of compressive strengths at the day of the specimen test were 34.7 MPa for W1. While for W2 average of compressive strengths the day of the specimens test were 23.3 MPa. The average yield strength of 6 mm reinforcement bars was 493 MPa (maximum strength was measured at 699 MPa), the average yield strength of 8 mm reinforcement bars was 513 MPa (maximum strength was measured at 775 MPa), the average yield strength of 10 mm reinforcement bars was 485 MPa (maximum strength was measured at 686 MPa), while the corresponding values for the 12 mm bars were 483 MPa (maximum strength was measured at 692 MPa).

During reinstatement of concrete for each specimen, cylinders of the repair mortar were cast. Compression testing of these cylinders was conducted on the days that the repaired specimens were tested to failure. The average concrete compressive strength was 58.1 MPa (average of six cylinders with strengths of 58.5, 56.1, 57.2, 56.1, 59.7 and 61.1 MPa) for RW1, tested 109 days after casting, and 28.1 MPa (average of six cylinders with strengths of 28.0, 28.5, 27.7, 29.1, 27.4 and 27.9 MPa) for RW2, tested 29 days after casting.

4. Initial Damage

The original specimens W1 and W2 showed a similar extent of damage previous testing to the repair and the repair techniques used was the same. During testing of the original specimens W1 and W2 was tested to failure under a quasi-static, reversed cyclic protocol developed in accordance with ACI 374.2R-13[8]. The testing results in similar levels of damage at the completion of testing of the original specimens, the damage included concrete crushing, vertical reinforcement buckling and horizontal reinforcement opening at specimen boundaries, as shown in Fig. 3.

a) b) c)

Fig. 3 – Initial damage in specimens.

5. Repair procedures

Fig. 4 shows the target of the final repair to RW1 and RW2. The purpose of the repairs was increase the strength, addition of the boundary elements and increase the stiffness in shear wall. These measures seek increase the strength and capacity of deformation of the shear walls.

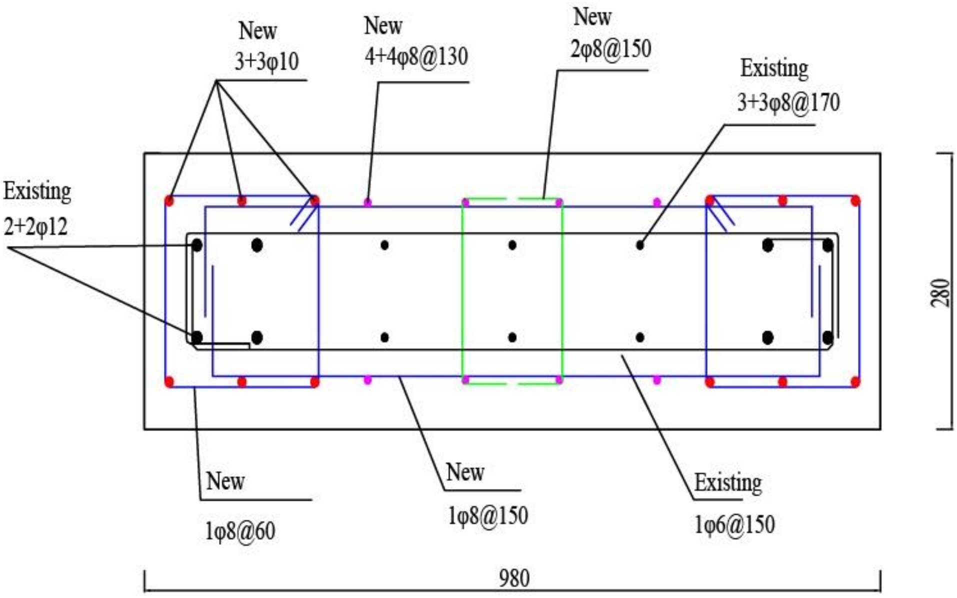


Fig. 4 – Repaired section detailed.

An overview of the repair procedures for RW1 and RW2, that was similar, is shown in Fig. 5. The repair process began removing all of the concrete of the boundary and a thickness of 4 cm of the concrete, on each side, on the center of the specimen. This removing was doing over a height of 1.75 m for W1 and 1.0 m for W2 above the base of the specimen; this distance was selected based on the extent of the observed damage and in order not to generate damage in unrepaired areas of the specimen. Concrete was removed using jackhammer equipment. After completion of demolition, the boundary longitudinal reinforcement were cut at the height of 300 mm above the base of the specimen, the new segments of longitudinal reinforcement were installed using anchors mechanic. The additional vertical reinforcement were placed anchoring the new bars at the base of the specimen. The anchoring was done with a drilling hammer, the new bars of the boundary (6 of 10mm) were anchored 150 mm and the new vertical distributed reinforcement (8 of 8mm) was anchored 12 cm. Furthermore, the epoxy was used to anchor the new reinforcement at the base. Once placed the new vertical reinforcement, was placed the transverse reinforcement in the boundary (stirrups with seismic hooks) and the horizontal distributed reinforcement. The spacing of the horizontal reinforcement was design according to AC318-14 (Chapter 18) [9]. Additional to vertical and horizontal reinforcement, mechanical interlock (two horizontal bars crossing the cross-section) was used, to resist shrinkage between new and old concrete. Reinstatement of repair concrete completed the repair process.

Fig. 5 – Repair Procedures for RW1 and RW2.

6. Test procedure and load protocol

The test setup is shown in Fig. 6a. An axial load of approximately $10\%A_g f_c'$ was applied at the bottom of the specimen by hydraulic jacks mounted on steel beam that load transfer by post-tensioning rod to top of specimen. Cyclic lateral displacements were applied to the specimens by a hydraulic actuator mounted horizontally to a reaction specimen. An out-of-plane restraint system was used to restrict out-of-plane movement at the top beam of the specimens. The loading protocol used was developed by [10] in accordance with ACI 374.2R-13 [8]. Fig. 6b shows the loading protocol. This protocol was similar for two specimens with three displacement-controlled cycles at each drift level.

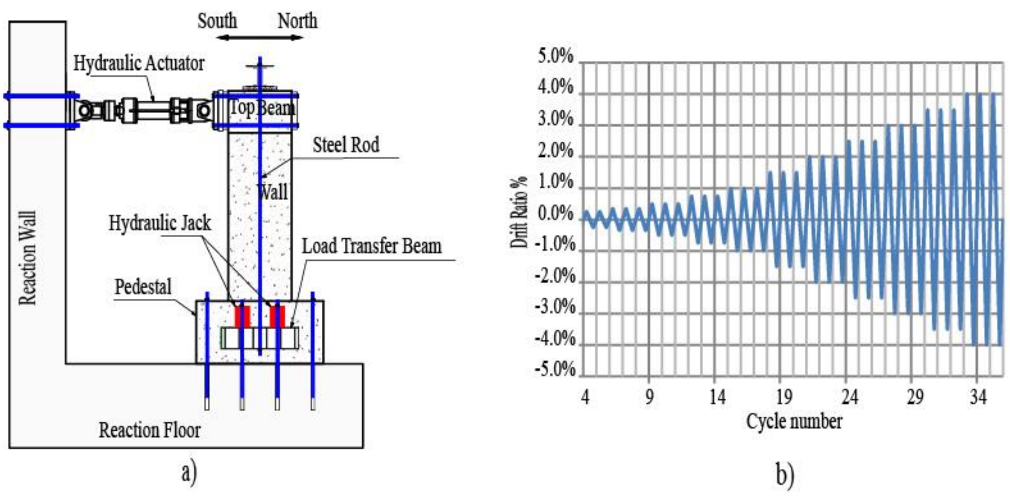


Fig. 6 – a) Test setup, b) Loading protocol.

7. Test results

Observed Damage

The principal damage in repaired specimens after the test is shown in Fig. 7. The observed damage for RW1 and RW2 was similar through the test. Cracking was first observed to occur in the repaired zone for the first cycle at 0.5% drift for RW1, while that in RW2 in the third cycle at 0.35% drift. At the peaks of pre-yield cycles, the widths of existing cracks above the repaired region were observed to open. Yielding occurred at roughly for the third cycle at 0.5% drift for RW1 and the first cycle at 0.5% drift for RW2 (strain gages were placed on the new reinforcing bars). The tensile cracks width increased until the third loading cycle at 1.5% drift in both specimens. Starting the 2.0% drift the specimens showed a considerable drop of strength, due to the rocking effect observed in the specimens during the test. The rocking effect was due to possible sliding in new reinforcing bars. In subsequent cycles the strength decreased faster, this effect was more visible in the RW2 (Fig. 7). The failure mode present in the two specimens showed crushing in the concrete in the zone between the wall and pedestal (Fig. 7). The tests were stopped in 2.5% drift for RW1 and 4.0% drift for RW2.

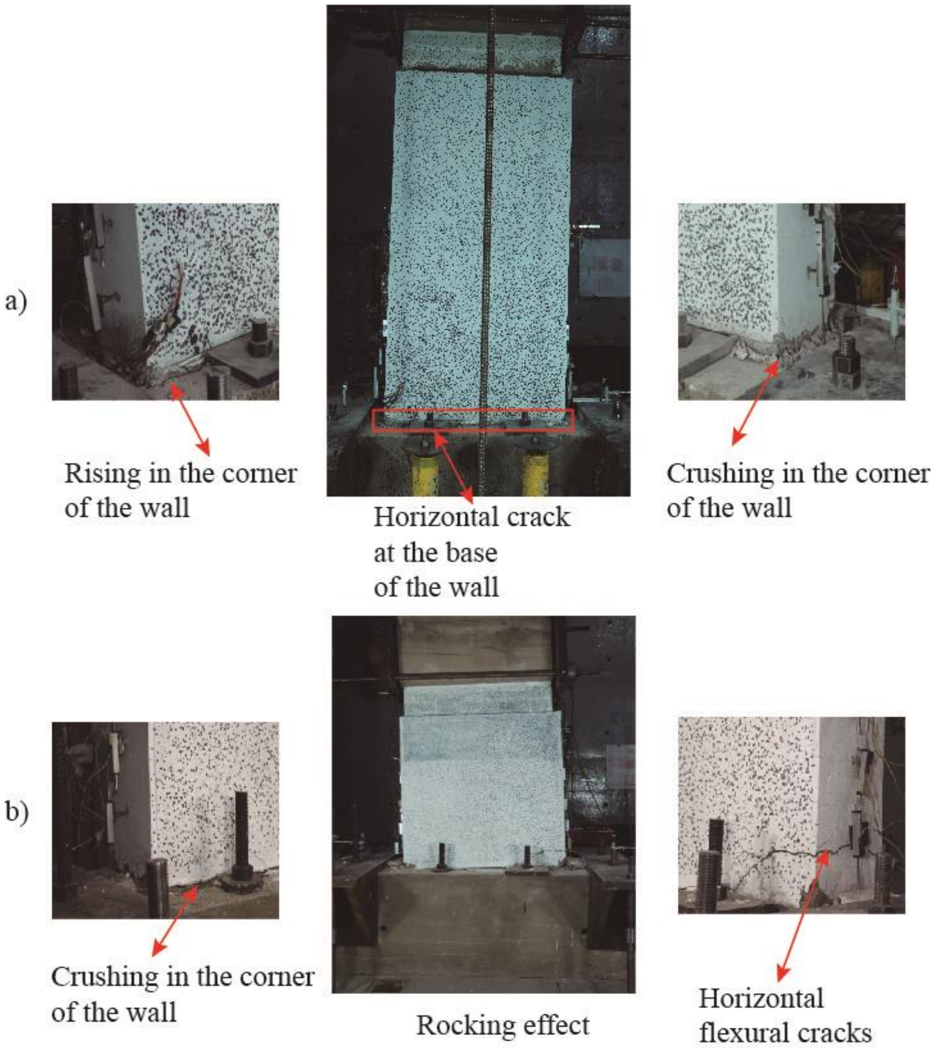


Fig. 7 – Damage photos a) RW1, b) RW2.

Load-Deformation

The load-deformation responses for the repaired specimens, RW1 and RW2, and the original specimens, W1 and W2, are shown in Fig. 8. The elastic stiffness, maximum strength, deformation and total dissipated energy of the repaired specimens was higher than that of the unrepaired specimens. For RW1 the stiffness was 48.5% higher than W1, while for RW2 the stiffness was 38.9% higher than W2. Similar trend occurred in maximum resistance where for RW1 the strength was 49.1% higher than W1, the increase was also seen for RW2 with 68.8% above W2. While, for the deformation the increase was RW1 13.2% higher than W1and in RW2 was 76.2% higher than W2. The maximum increases occurred in the total dissipated energy, where the increase was 59.8% between RW1 and W1, while for RW2 the increase was 180.1% compared to W2.

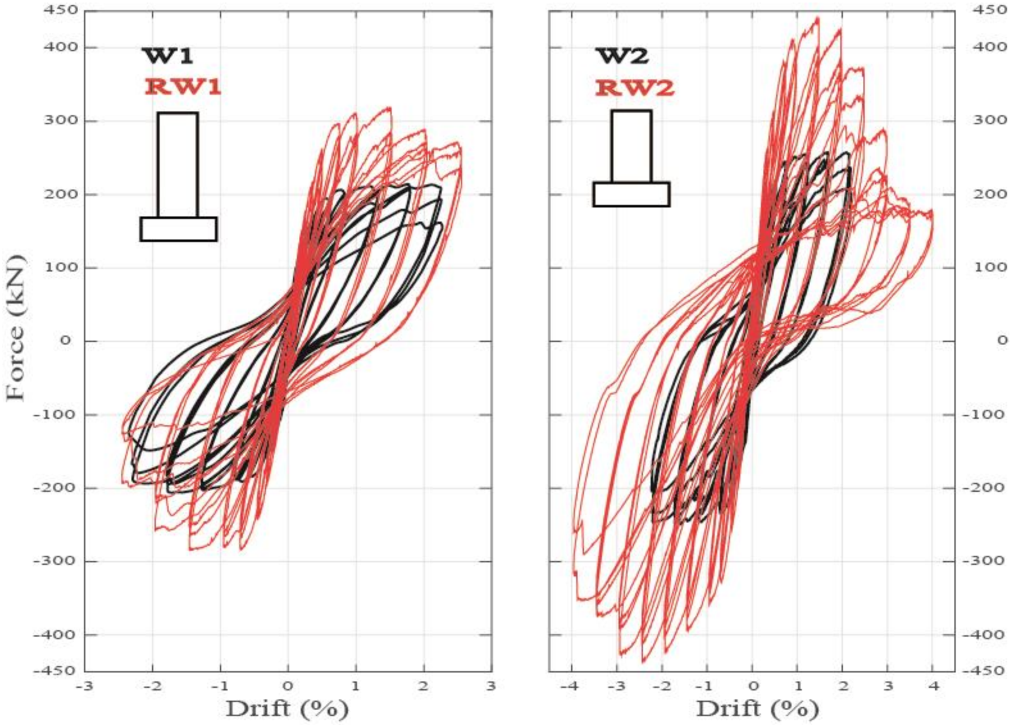


Fig. 8 – Measured load-drift responses

8. Conclusions

This paper summarizes an experimental campaign conducted to characterize the seismic behavior of repaired RC walls with increased thickness (similar to repaired walls does in Chile post-2010). The variable analyzed in the test program were the height of the specimens. The reinforcement ratio and the specimen thickness were similar for two specimens.

From the specimen tests, stiffness, maximum strength, deformation capacity, and total dissipated energy were obtained. Additionally, the failure mode of specimens was identified. The specimens presented a rocking-type failure mode, with the presence of horizontal cracks at the boundaries of the specimen and a horizontal crack at the base of the specimen (sliding type). The mode of failure present in the specimens may be due to a possible slippage of the new reinforcement bars placed on the specimen. Although despite the mode of failure present in the specimens repaired, it was possible to improve stiffness, maximum strength, deformability, and dissipated energy. Although the mode of failure present in the repaired specimens, it was possible to improve stiffness, maximum strength, capacity deformability, and dissipated energy. The RW2 specimen showed a sudden drop in resistance in its positive part, which could be interpreted as fragile behavior.

9. References

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