Testing of heavily damaged reinforced concrete walls repaired using concrete and steel replacement

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ABSTRACT

Post-earthquake damage assessments are essential to decisions on the need for and extent of structural repairs. Despite this, New Zealand currently has no official guidelines to evaluate the residual capacity of earthquake-damaged structures and the effectiveness of different repair techniques. This project evaluated the effectiveness of repair techniques to restore the seismic capacity of RC buildings by conducting a series of tests on large-scale RC walls designed according to the current New Zealand Concrete Structures Standard NZS 3101:2006 A3. Two identical wall specimens that were previously tested and damaged were repaired and retested using the same loading conditions. The damage prior to repairs was consistent with well-detailed flexure dominant walls, including concrete crushing and buckling of the reinforcing bars. Both walls were repaired by replacing the concrete and the reinforcing bars in the plastic hinge region. The new reinforcing bars were connected to the existing bars using welded connections. While one wall kept its original foundation and reinforcement, the other wall had a new foundation constructed to avoid discontinuities at the critical section. Both repaired walls exhibited similar performance during testing compared to the reference wall in terms of damage progression, stiffness, strength, and displacement capacity. The welded reinforcement connections did not fail throughout the tests, confirming that properly designed welded connections for G500E reinforcement can resist earthquake demands at the critical region of the wall.

1 INTRODUCTION

1.1 Background

As part of the consequences of the 2010-2011 Canterbury earthquakes, 60% of the multi-storey reinforced concrete (RC) buildings in the Central Business District in Christchurch were demolished (Kim et al., 2017). In several cases, the decision was taken even when just moderate damage was observed (Kim et al., 2017). Several factors influenced the decision to demolish the buildings instead of repairing them, and the uncertainty of the seismic performance of repaired RC buildings was one of them. The lack of national guidelines and more experimental data generates a lack of consistency among engineers in the after-earthquake assessment of damaged RC buildings. A similar but different outcome occurred in Chile after the 2010 Maule earthquake, where heavily damaged RC buildings were repaired and strengthened. Concrete crushing, bar buckling, and residual displacement were repaired in an 18-storey RC building in Santiago (Sherstobitoff et al., 2012). The total cost of repairing the building was close to 25% of the estimated cost of demolishing and rebuilding it. Examples like this show that an effective repair procedure can significantly increase society's resilience in a post-earthquake scenario reducing costs, operation time, and environmental impacts of the city reconstruction.

1.2 Previous research

The study of the assessment of the post-earthquake scenario of RC buildings is extensive. Combining different RC components, damage states, and repair techniques creates a vast, elaborate matrix of possibilities to explore. On top of this, in some instances, a benchmark specimen is needed, increasing the complexity of the experimental programs. Some of the past research on the repairability of RC components conducted in New Zealand is described below.

Moderately damaged RC beams have been studied by Marder (Marder et al., 2018). Different beam specimens were damaged under simulated earthquake demands, showing concrete cracking and concrete spalling. Epoxy injection and repair mortar were used to repair the beams, and later a cyclic loading was applied to evaluate their seismic performance. Results indicate that under that level of damage, the damaging load affects neither the strength nor the displacement capacity of the beams.

Motter (Motter et al., 2017) repaired and tested two previously damaged RC walls. Concrete crushing, bar buckling, and bar fracture were observed in the specimens, as shown in Figure 1 (a). The damaged concrete was removed, and new bars were used to replace the damaged ones. A welded connection was used to connect the new bars with the existing ones. Figure 1 (b) shows the repaired wall before attaching the formwork and pouring the concrete. A similar procedure was conducted by Cortes-Puentes (Cortes-Puentes et al., 2018) in Canada, as shown in Figure 1 (c) and Figure 1 (d). Results indicate a good strength recovery and a poor stiffness recovery. Despite a complete replacement of the plastic hinge area, the walls could not fully restore their displacement capacity.

(a) (b) (c) (d)

Figure 1: Repair procedures. (a) Damage before repair in wall M5-R (Motter et al., 2017) (b) Steel replacement in wall M5-R (Motter et al., 2017), (c) Damage before repair in wall RW1-SR (Cortes-Puentes et al., 2018), and (d) Steel replacement in wall RW1-SR (Cortes-Puentes et al., 2018).

1.3 Objectives

This study aims to repair heavily damaged reinforced concrete walls and reinstate their original strength and displacement capacity. A secondary objective is to assess the difference in repairs based on access to foundation for bar replacement. The methodology and test matrix were selected to best achieve this.

2 METHODOLOGY

2.1 Wall specimens

Two identical heavily damaged and repaired flexural RC walls are discussed in the present study. The original walls were part of a previous experimental program described by Muñoz (Muñoz et al. 2022). Walls W2D and W1C were repaired and subsequently renamed W5D-R and W6C-R. The concrete and the reinforcing bars were replaced "like for like", aiming to achieve a cross-section identical to the original one. The difference between wall W5D-R and wall W6C-R is the use of the existing foundation in the new wall specimens.

2.2 Damage in previously tested walls and repair plan

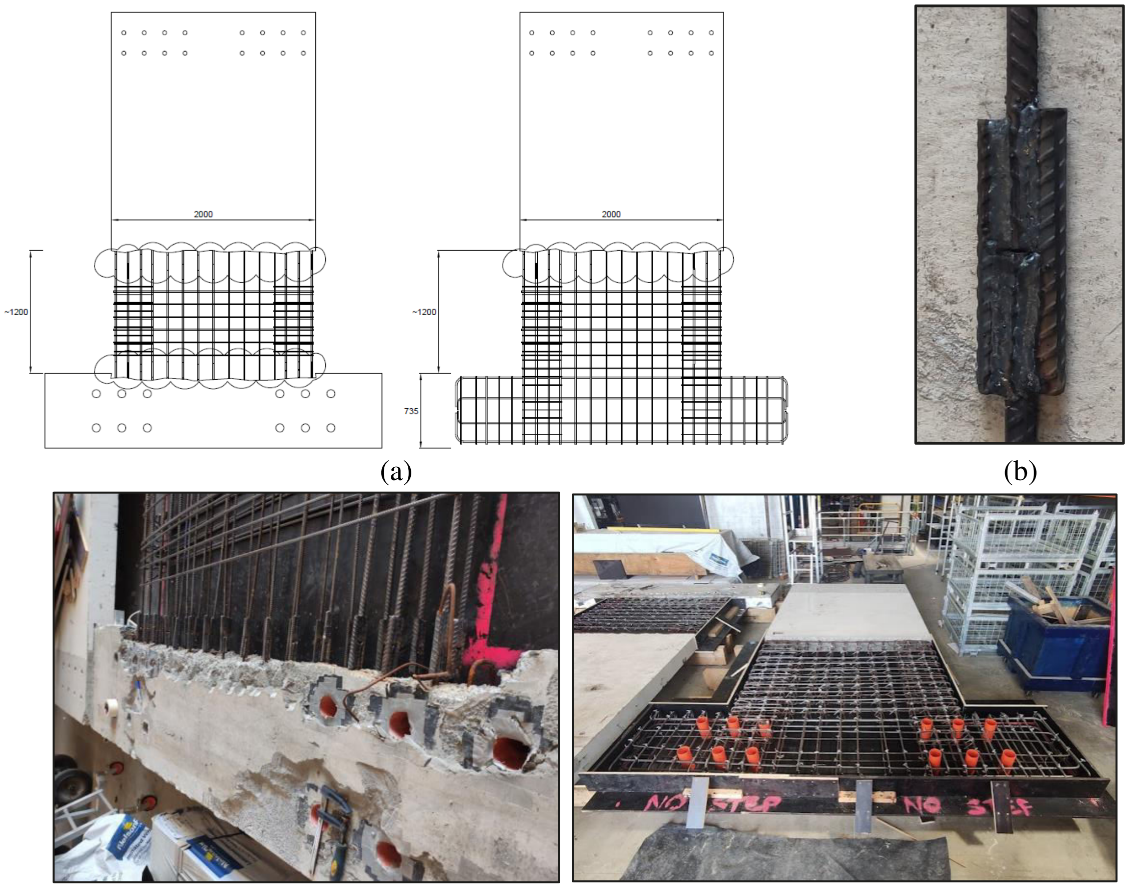
At the end of the test of walls W1C and W2D, concrete crushing, bar buckling, and bar fracture were observed at the base. The damage started in the boundary element and then propagated towards the centre of the walls. Distributed flexural and shear cracking was observed almost up to the top of the wall. Figure 2 shows walls W1C and W2D at the end of testing.

(a) (b)

Figure 2: Damage in walls after the test, (a) W1C, and (b) W2D.

The repair of the wall started with the hydro-demolition of the lower section of the walls. The height to hydro demolish was defined to avoid the need to epoxy inject wide cracks. The walls were hydro-demolished up to the top crack with a 0.2 mm width, which was 1.2 m above the wall base. While the foundation of wall W1C was disposed of, the foundation of wall W2D was retained in the repaired wall W5D-R. Close to 80 mm of top concrete in the foundation was hydro-demolished to achieve a welded connection at the interface. All the longitudinal bars were cut, leaving close to 100 mm free for the welded connection. As described earlier, wall W5D-R used the original wall foundation, while a new foundation was cast in wall W6C-R. Figure 3 (a) shows a scheme of the walls, showing the new reinforcement and clouds around the cross-sections with a discontinuity created by the bar connections (horizontal and confinement reinforcement removed for clarity). The accessibility to reach the wall foundation in actual buildings defined the differences between these foundations. In a post-earthquake scenario, the wall ideally should be repaired considering works in the foundation, either total or partial replacement. These works would help with the strain penetration, bond slip and avoiding a discontinuity in the critical cross-section. However, in multiple cases, accessing the wall foundation is challenging and can considerably increase repair costs. A repair procedure without work in the wall foundation is captured by wall W6C-R., leading to a discontinuity created by the rebar connection at the critical cross-section.

In all cases, the rebars were connected with an indirect butt-welded connection, as shown in Figure 3 (b). The detail of the welded connection meets the requirements of a connection type BI-1d according to the New Zealand Standard of Structural steel welding Part 3: Welding of reinforcing steel, AS/NZS 1554.3:2014 (Standards New Zealand, 2014). New horizontal and transverse reinforcement was provided in the repaired area. Figure 3 (c) shows the discontinuity created by the welded connection at the base of wall W6C-R, and Figure 3 (d) shows wall W5D-R before placing the new concrete.



(a) (b)

(c) (d)

Figure 3: Repair procedure, (a) walls W5D-R and W6C-R, (b) indirect butt welded connection used for the repair, (c) Welded connection at the base of wall W6C-R, and (d) wall W5D-R before pouring the new concrete.

2.3 Test matrix and loading protocol

The three walls presented in this study show the results of two heavily damaged repaired RC walls with and without a discontinuity created by a welded connection at the base and a benchmark wall used for comparison analysis. Apart from the repair works, all three walls are nominally identical.

The loading protocol was also identical between the walls. Constant axial force equivalent to 5% of the nominal compression capacity of the cross-section of the wall was applied while increasing cyclic displacement demands were applied at the top of the wall, at 4 m above the wall base. With a wall length of 2 m, the shear depth to span ratio of the walls was 2. The first cycles were force-controlled and aimed to achieve 25%, 50%, and 75% of the expected yielding moment of the inner HD16 bar. After this, two cycles per drift level were applied until a failure occurred, defined as a 20% drop in lateral strength. An extra cycle after that was conducted when possible. The lateral drift levels were 0.3%, 0.4%, 0.5%, 0.75%, 1%, 1.5%, 2%, 3%, and 4% of the height of the lateral force.

2.4 Test setup and instrumentation

The walls were anchored to the strong floor of the lab using a post-tensioned grouted foundation. A loading beam was attached to the top of the wall to connect the three actuators and simulate the earthquake demands. Two vertical actuators applied the constant axial force while the horizontal actuator applied the horizontal displacement demands. A photo of the test setup is shown in Figure 4.

The walls were heavily instrumented with different sensors. An arrangement of portal gauges and linear potentiometers was used to capture the wall deformations. Strain gauges were glued to the longitudinal reinforcing bars to capture strains at different heights. Draw wires and LVDTs were used to measure global and out-of-plane displacements. On the wall face shown in Figure 4, a Digital Analysis Correlation (DIC) analysis was conducted to obtain strain and displacement fields.

Figure 4: Test setup.

3 EXPERIMENTAL RESULTS

3.1 Damage progression and observed failures modes

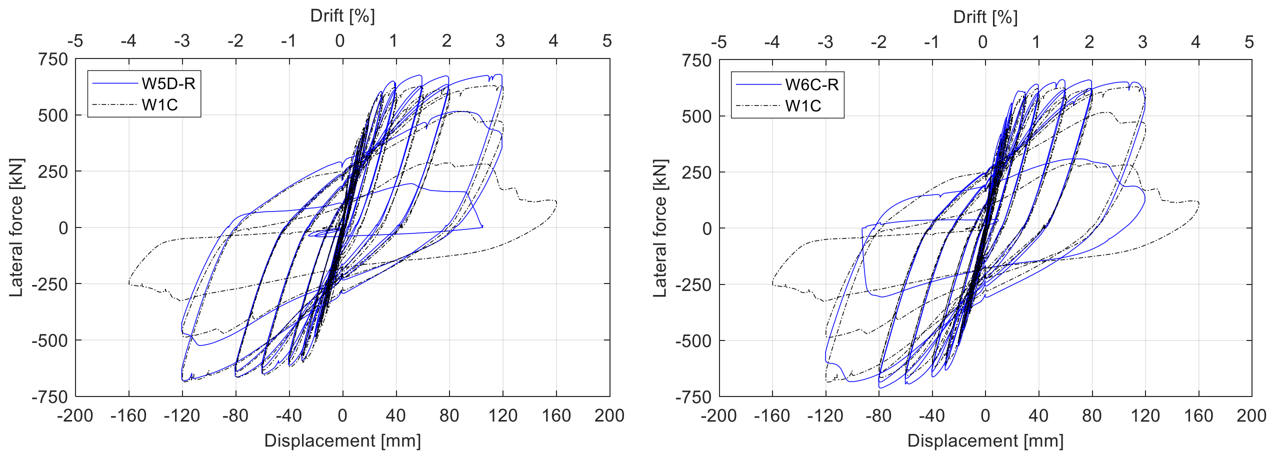
The overall response of repaired walls was similar to that of the benchmark wall W1C. A dominant flexural response combined with a moderate shear crack pattern was observed during the tests. Vertical cracks were observed at the ends of the wall at 0.75% drift, which led to concrete spalling at 1.5% and 2%. The crack pattern was not affected by the presence of the cross-sections with welded connections. Concrete crushing and bar buckling were observed in the boundary element at 2% drift, and the failure of the walls occurred at the second cycle of 3% drift. The damage propagated from the boundary elements towards the centre of the walls, crushing concrete and buckling longitudinal bars. Figure 5 shows walls W5D-R and W6C-R at the end of the test, showing a similar failure between them and compared to walls W1C and W2D (shown in Figure 2). The heavily damaged area in wall W6C-R was slightly above the damaged area in the other three walls. This shift could be explained by the welded connection at the base, which moved the critical section slightly above the original location. However, no significant changes were observed in the overall wall response nor the load-displacement curves, which will be discussed below.

(a) (b)

Figure 5: Damage at the end of the tests, (a) wall W5D-R and (b) wall W6C-R.

3.2 Force displacement response

Figure 6 shows the lateral load-displacement responses of the repaired walls compared to the response of the benchmark wall W1C. As observed from the figure, the wall responses are similar to wall W1C, showing a stable behaviour of up to 3% lateral drift. Wall W6C-R had a slight drop in strength at the end of the first cycle at 3% drift, but the response was still almost identical to W1C. The fact that the severe damage was shifted above the section of the other walls can explain the negligible reduction in the displacement capacity. The fact that wall W1C sustained cycles to 4% is not considered relevant considering the significant reduction in lateral strength on those cycles. Different sources can explain the increment in the strength of the repaired walls. In wall W5D-R, the increment occurred only in the positive direction, indicating that the difference may have occurred by slight offsets in the reinforcement or the test setup. The symmetrical increment observed in wall W6C-R may be explained by the shift of the critical section closer to the top of the wall. A reduction in the effective height will increase the wall lateral strength to achieve the same moment capacity. However, the range of these differences is small and should not be critical in analysing the repaired walls' response.



(a) (b)

Figure 6: Load displacement responses, (a) wall W5D-R, and (b) wall W6C-R.

3.3 Backbone response comparison

To better understand the repaired walls' seismic performance, backbone curves were extracted from the load-displacement responses and are plotted in Figure 7. Immediately it can be observed that the curves are similar to W1C. The figure also shows values of performance modification factor, similar to the ones defined by FEMA 306, Evaluation of earthquake damaged concrete and masonry wall buildings (ATC, 1998).

Stiffness, strength, and displacement capacity are defined for each curve, and then the ratio of repaired over original is the performance modification factor, λ. Stiffness, K, is defined as the secant stiffness obtained at 70% of the wall strength. Strength, Q, is defined as the maximum lateral force resisted by the walls. Displacement capacity, D, is defined as the displacement value at a 20% drop in lateral force. Values shown in Figure 8 indicate a successful repair procedure for both walls. In terms of stiffness, the range of ±15% variation does not significantly impact the building behaviour under seismic conditions. Both repaired walls had a minor overstrength, which was also considered negligible to the wall response. A capacity design recheck could be done to ensure that sufficient shear strength is maintained. The most important outcome is the essentially identical displacement capacity. These results show that a complex repair procedure replacing concrete and reinforcing steel can achieve a similar response compared to the original wall when adequately conducted.

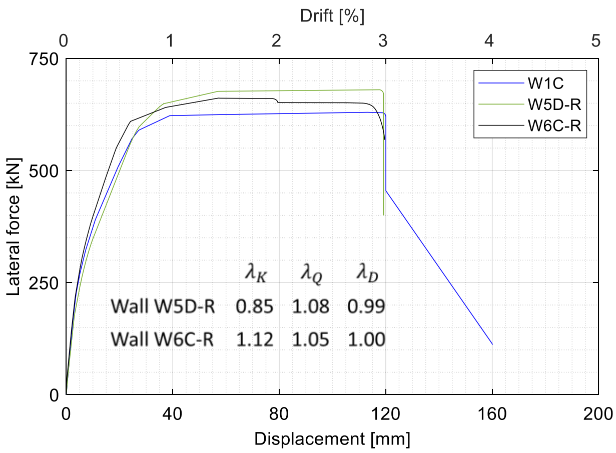


Figure 7: Backbone curves and performance modification factors.

4 DISCUSSION AND CONCLUSIONS

The results of two heavily damaged and repaired RC walls were presented. Differences in the use of the existing wall foundation provide experimental data for different scenarios when reaching the wall foundation is unfeasible. The steel replacement was conducted using an indirect butt-welded connection meeting the requirements of AS/NZS 1554.3:2014. The repair methodology aimed for a "like for like" replacement for concrete and steel.

Crack pattern and failure modes were similar to the benchmark wall. The cross-sections with the discontinuities generated by the welded connections did not affect the overall wall response. In wall W6C-R, there was a slight shift in the location of primary damage, but it did not appear to affect the wall response. The repaired walls achieved flexural strength and a displacement capacity nominally identical to the benchmark wall. These results show that when properly conducted, a total replacement of the plastic hinge region in a ductile flexural wall can successfully restore its seismic performance. However, previous experimental programs have shown a reduction in the displacement capacity of the repaired walls. Uncertainties in the repair process, quality of the materials, welding procedure, qualifications of the welder and weather conditions, among others, can alter the outcome of the wall repair for the worse. All these points suggest that a reinforcement connection should be avoided in the plastic hinge area, connecting the bars in areas with less inelastic demands and going into the foundation for proper bar anchoring. If this is not possible, stricter requirements should be met for a reinforcement replacement in the plastic hinge area.

Finally, this study did not consider practical aspects of the repair methodology in real buildings. In particular, the presence of the axial force is a relevant aspect during the repair process. It will be a case-by-case decision if the wall needs to be jacked to redistribute the axial demands temporarily or alternatively use of consecutive partial replacements along the wall length.

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