EXPERIMENTAL STUDY OF DAMAGED REINFORCED CONCRETE WALLS REPAIRED USING SIMPLE TECHNIQUES

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Abstract

Recent damaging earthquakes such as Maule 2010 (Chile), Canterbury 2010-2011 (New Zealand), and Tohoku 2011 (Japan) tested seismic design procedures globally. Despite satisfactory results in terms of life safety performance, a lack of knowledge was revealed regarding the post-earthquake condition of damaged reinforced concrete (RC) buildings. The assessment of residual capacity and the effectiveness of repair techniques for earthquake damaged RC buildings is not well understood, and existing guidance documents are incomplete and difficult to implement. Recent tests of moderately damaged RC beams repaired using epoxy injection and repair mortar showed no significant reduction in strength and displacement capacity and partial stiffness recovery of 80%. Similar results were found for heavily damaged RC walls repaired by complete replacement of concrete and reinforcement in the plastic hinge region, but with a recovered stiffness of only 50% of the undamaged wall stiffness. The lower recovered stiffness for the repaired walls when compared to the beams was attributed to the axial load and light reinforcement ratios that allowed cracks to close, preventing epoxy repair.

It is common practice to test RC components up to failure, which limits possible repair tests to focusing on heavy damage states and complex repair techniques. The proposed research will instead study moderately damaged RC walls repaired using epoxy injection and epoxy mortar. Such walls represent a common post-earthquake scenario that have been infrequently investigated by past research. Four identical ductile RC walls with a 3.5 m height, 2 m length, and 0.175 m thickness will be tested. The first test wall will be subjected to a standard cyclic loading protocol, and a drift value between concrete spalling and bar buckling will be used as the starting point in the cyclic loading in the other walls. The second wall (undamaged wall) will be tested only with cycles after the defined "moderate damage" drift. The third wall (damaged wall) will be tested with a simulated earthquake loading history, followed by cycles after the defined "moderate damage" drift. The fourth wall (repaired wall) will be tested under the same loading protocol as the third, but after the earthquake demand epoxy injection and epoxy mortar will be applied as a repair technique. The global response of the test walls, such as stiffness, strength, and displacement capacity, will be compared to quantify the effect of the prior damage and the repair technique on the expected seismic response of the prototype wall. It is expected that the repaired wall will likely maintain strength and displacement capacity, but that the stiffness may not be fully recovered. However, a significant improvement in stiffness could be found between the damaged and the repaired test walls.

Keywords: reinforced concrete; walls; repair; experimental; earthquake.

1. Introduction

Modern seismic design standards require structures to be designed to resist large earthquake demands with a low probability of collapse and protection of life safety. In general, the observed performance of structures in Chile, New Zealand, and Japan after the Maule (2010), Canterbury (2010-2011), and Tohoku (2011) earthquakes achieved these performance requirements. However, there was substantial uncertainty regarding the post-earthquake condition of damaged structures and potential behavior in future earthquakes. In the Central Business District of Christchurch, New Zealand, 60% of the Reinforced Concrete (RC) buildings greater than two stories were demolished, even when only moderate damage was observed [1]. One of the reasons attributed to that outcome was the uncertainty of the seismic performance of the damaged building, and even further, the performance of the repaired building. A different scenario was observed in Chile, where heavily damaged RC walls were repaired and strengthened after the Maule earthquake [2]. The damage in RC walls included concrete crushing, bar buckling, bar fracture, and evident residual displacement in the entire buildings, which was corrected using hydraulic jacks during the repair process. There are two existing international guidelines to assess the performance of damaged and repaired RC structures ([3], [4], and [5]), but they do not rely on a robust basis of repaired test specimens across all of the damage states defined in the guidelines.

The popularity of RC walls in the seismic design of buildings worldwide makes them a high priority to study the residual capacity and the effectiveness of post-earthquake repair techniques. The following section describes previous experimental research of RC walls, mostly focused on heavily damaged walls. The tests were generally conducted using an increasing cyclic loading protocol up to the failure of the specimens, where heavy damage was observed. This heavy damage state requires complex methodologies in the repair process, which is not a common case in a post-earthquake scenario of seismically designed RC buildings. To investigate moderate damage states (as concrete cracking, evident bar yielding, and concrete spalling), the test needs to be stopped prior to failure. Also, a benchmark wall is required to be able to compare and draw conclusions on the effects of the earthquake damage or repair on various response parameters.

Repair of earthquake damaged RC components could consist of several options depending on the observed damage. Concrete and/or steel reinforcement replacement are generally conducted on heavily damaged components, where a strengthening or enhancement to the design is often targeted as well. The strengthening can consist in increasing the concrete area, the steel reinforcement ratio, or both. The addition of new materials in the cross-section as Fiber Reinforced Polymer (FRP) is also another alternative. For slight or moderate damage, where bar buckling and concrete crushing is not evident, injecting the cracks with epoxy resin and replacing the spalled concrete with repair mortar are common options these damage states. These options generate several combinations to repair and strengthening of earthquake damaged RC walls.

The objective of this study is to summarize and analyze previous experimental work on repaired RC walls and identify gaps in their results. Backbones curves extracted from the load-displacement responses are generated to compare the effectiveness of different repair techniques in the overall behavior of the specimens. Finally, an ongoing experimental program of earthquake damaged and repaired RC walls conducted at the University of Auckland is presented.

2. Previous experimental research on repaired RC walls

2.1 Overall description of test walls.

A total of 70 previously tested repaired RC walls were found from 27 different studies, dating back to 1975. Fig. 1 shows the main characteristics of the walls, where the most common cross-section is rectangular, representing 49% of the total tests. The number of barbell shape and "H" shape walls correlates well with the number of walls with high shear stress (more than 4 [MPa]). Wall thickness equal to or greater than 150 [mm] represents just 31% of the total, which aligns with the scale factors lesser than 50% in most of the cases. The axial force ratio, defined as the ratio of the vertical axial load over the compressive strength of the section, are typically 10% of below, which represents low to medium axial forces demands for New Zealand buildings.

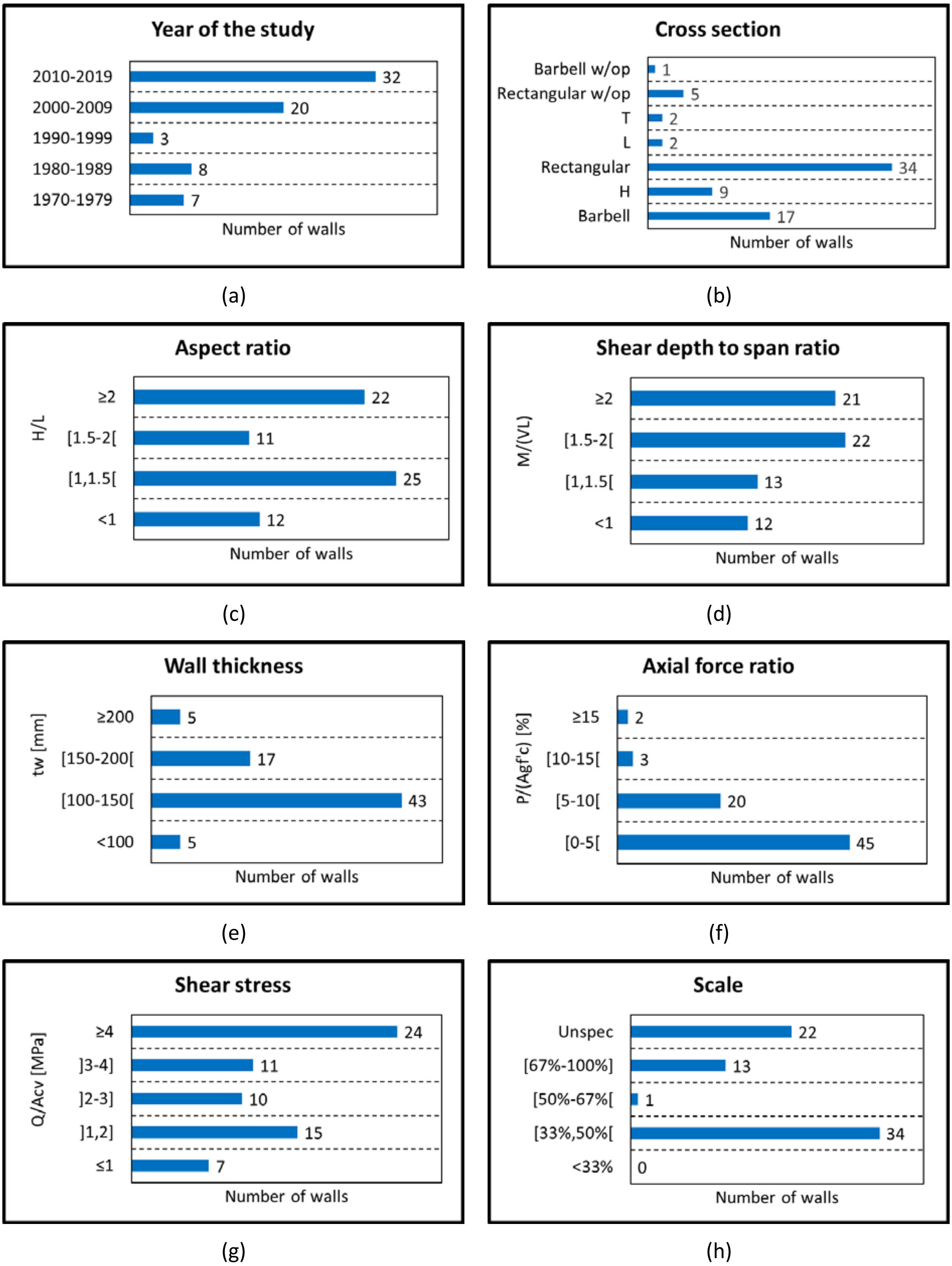


Figure 1: Characteristics of the repaired walls found in literature.

The maximum damage present in the test walls that was categorized as “Heavy” (including bar buckling, bar fracture, boundary buckling, boundary crushing, concrete crushing, web crushing, and lap splice slip failure), representing a 93% of the total (see Fig. 2). Generally, this damage state is associated with the failure of the element, triggered by a drop in lateral strength of 20% or more.

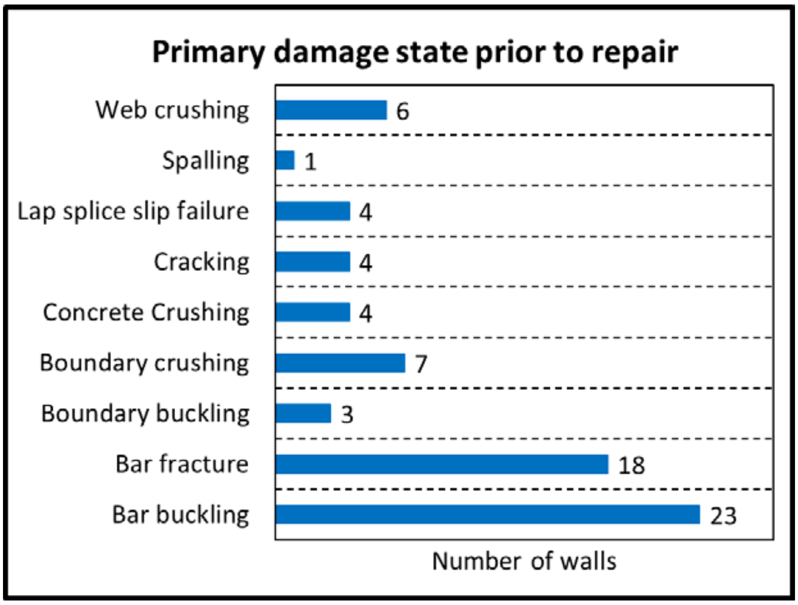


Figure 2: Primary damage state after the first test.

2.2 Overall description of repair techniques

The different repair techniques used on the test walls are summarized in Fig. 3. Replacement (Repl.) includes replacing cover concrete (C), steel (S), and both (C+S). Any use of fiber-reinforced polymer is represented by “FRP”. The strengthening of the section by adding additional reinforcement, increasing the cross-section of the wall, or adding others elements such as steel plates, is cover by “Stre.”.

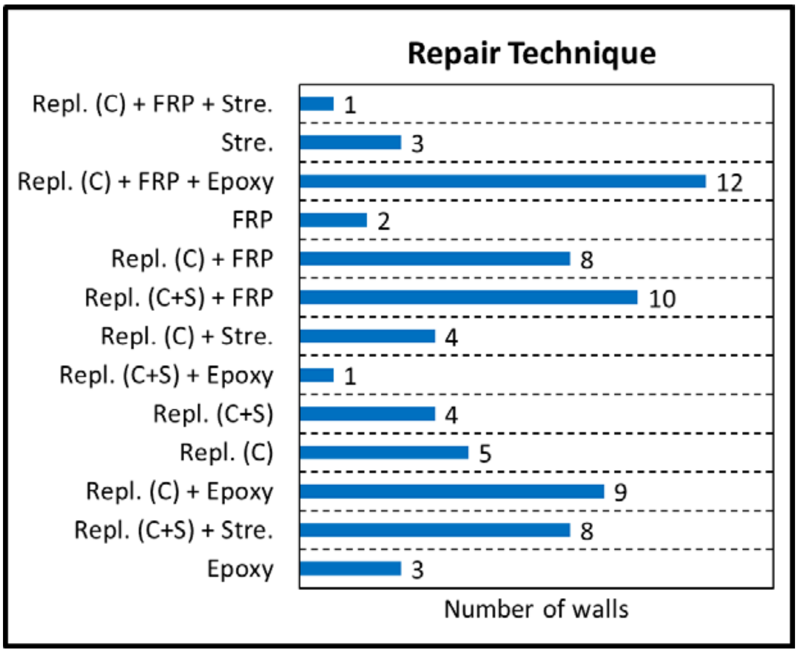


Figure 3: Repair techniques used in the repaired walls.

Categorizing the different repair techniques with respect to their intended improvement to the wall, or the complexity of the procedure is not straightforward due to the wide range of techniques and combinations used. However, they can be grouped into three main easily identifiable categories: 1) simple repair technique, 2) complex repair techniques, and 3) strengthening repair techniques. The first definition covers crack injection using epoxy resin and repair mortar used for minor spalling. Complex repair techniques refer to procedures intended to achieve similar performance to the original specimen (strain hardening or aging not considered), replacing the elements “like for like”. Finally, strengthening repair techniques are intended to achieve a higher level of performance than the original walls. Generally, the effectiveness and influence of the repair technique is expressed in terms of strength, stiffness, and displacement capacity. Depending on the repair technique, some or all these response parameters can be affected. Table 1 presents the different repair techniques sorted by these three categories, where it can be seen that most of the repair techniques were intended to achieve higher performance. The use of FRP sheets is categorized in the last column because the design generally aims to strengthen the member rather than achieve the same response.

Table 1: Repair categories

Simple repair technique (Category I)	**Achieve similar capacity (Category II)**	**Achieve higher capacity (Category III)**
Epoxy injection	Repl. C	Repl. C + FRP + Stre.
Epoxy mortar	Repl. C + Epoxy	Stre.
	Repl. C + S	Repl. C + FRP + Epoxy
	Repl. C + S + Epoxy	FRP
		Repl. C + FRP
		Repl. C + S + FRP
		Repl. C + Stre.
		Repl. C + S + Stre

The distribution of the different repair technique for all 70 previously tested walls is showed in Table 1, where 48 specimens fall into category III, 19 walls are in category II, and only 3 walls are in category I. The majority of previous tests are focused on heavy damage prior to repair and complex repair techniques. However, the residual capacity and the use of simple repair technique on flexural dominant rectangular RC walls are not adequately addressed by the existing tests. On top of this, none of the previous tests used a benchmark wall, and so the displacement capacity of walls damaged prior to failure have never been studied directly.

2.2 Backbones curves and performance modification factors

The performance of the original and the repaired walls were defined using global response parameters including secant stiffness, lateral strength, and lateral displacement capacity, similar to the methodology of FEMA 306 and the companion documents, FEMA 307 and FEMA 308 ([3], [6], and [7]). The backbone curves of both the original and the repaired walls were digitized from the lateral load-displacement response provided in the articles (Fig. 4 (a) and Fig. 4 (b)). The maximum force (Q), the secant stiffness at 80% of the maximum force (K), and the displacement at 20% drop in strength (D) were extracted using the backbone curve (Fig. 4 (c)). The ratio of the response parameter in the repaired wall to the parameter in the original wall was defined as the performance modification factor (PMF) of that test (Fig. 4 (d)). The PMF’s for stiffness, strength, and displacement capacity are labelled, λ_K^*, λ_Q^*, and λ_D^*, respectively. Special displacement capacity PMFs were also calculated when one wall failed (original or repaired) and the other does not

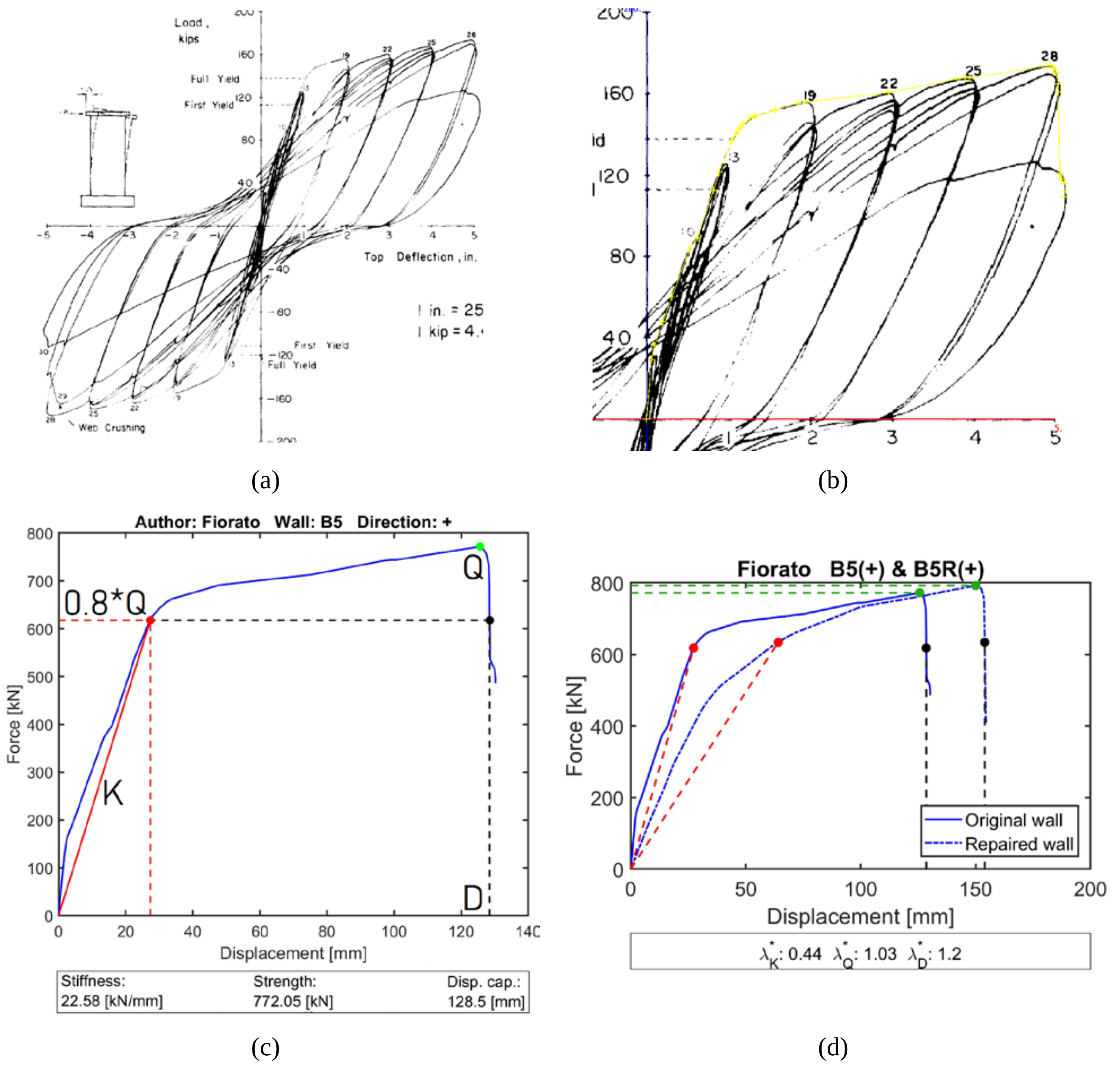
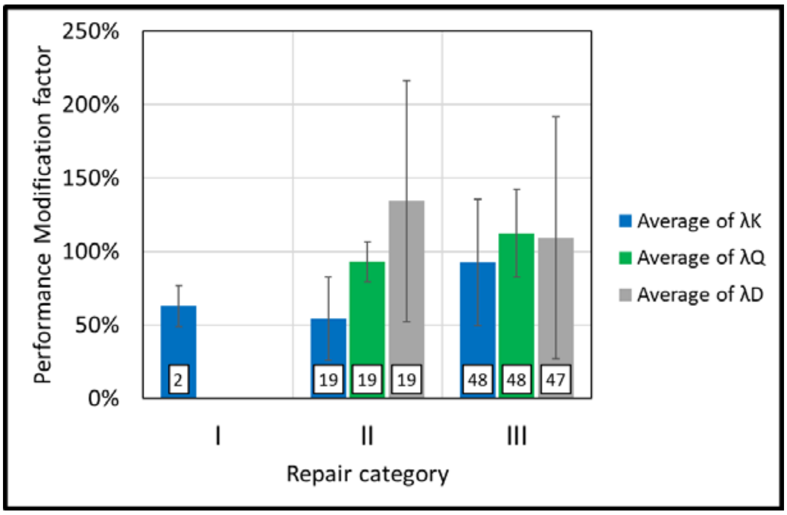
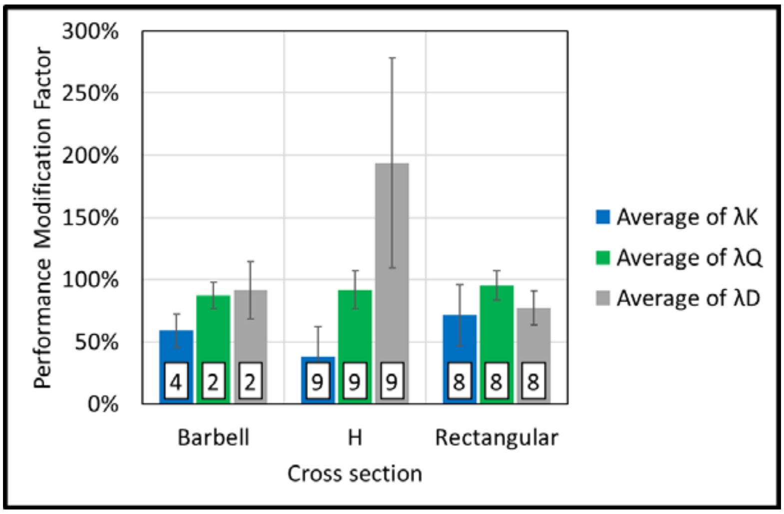


Figure 4: Methodology to define the PMF's. (a) Hysteresis response provided from the study [8], (b) Digitalization of the backbone curve, (c) Extraction of K, Q, and D, and (d) Comparison between original and repaired wall.

The averaged PMFs for stiffness, strength, and displacement capacity are presented in Fig. 5 (a). The number at the base of each column represent the number of walls considered in that average, and the error bars represent one standard deviation in each direction of the sample. In all categories, the average stiffness recovery was more than 50% (63%, 54%, and 92% in categories, I, II, and III, respectively), but generally less than a full recovery. Only 19 repaired walls achieved a stiffness higher than their benchmark walls, all grouped in category III. For repair category I, only λ_K^* was calculated since the walls tested were not suitable to calculate λ_Q^*, and λ_D^* (the third wall is not presented because there are inconsistencies in the load-displacement curve provided). The 93% and 112% average strength recovery for categories II and III, respectively, show that full strength recovery is not hard to achieve. In categories II and III, the displacement capacity was greater than 100% (134% for categories II, and 109% for category III), but the calculation of these values are influenced by cases where only one of the walls in the set failed. If these cases are removed from the analysis, the average displacement capacity recovery was 113% for category II and 115% for category III. Both cases still have values higher than 100%, but the difference between them was reduced. The unexpected high displacement capacity and low stiffness of the repaired walls in category II compared with those in category III was attributed to a large amount of tests with "H" cross-section. Fig. 5 (b) present the PMF's for all walls in categories I and II according to their cross-section. Wall with flanges generally behaved in a more flexible manner after the repair, with reduced stiffness but increased displacement capacity. Minor repair were completed on the flanges because the damage was typically concentrated on the web region of the wall. However, the flange response is crucial to obtain a full recovery of the stiffness.



(a)



(b)

Figure 5: (a) Average of λ_K^*, λ_Q^*, and λ_D^* using the entire database versus repair category, (b) Average of λ_K^*, λ_Q^*, and λ_D^* for categories I and II versus cross section.

3. Experimental program on moderate damaged RC walls

The lack of previous repaired wall tests that fall into category I in Table 1 motivated the ongoing experimental work conducted at the University of Auckland. In addition, independent test walls are required to achieve a full comparison of the walls performance and were often missing from past studies. A repaired wall and an unrepaired wall are required to assess the effectiveness of the repair technique, and an undamaged wall is required to obtain the residual capacity compared to the damaged wall. Due to the lack of data on such repaired walls, additional testing on moderately damaged RC walls and simple repaired was proposed.

An archetype building was proposed as a basis of a rectangular wall design in order to ensure that the test walls were representative of realistic designs. The archetype building was evaluated using the New Zealand Standard Structural design actions – Part 5: Earthquake actions [9]. The nominal concrete compressive strength was 40 MPa, and the reinforcing steel selected was Grade 500 (except for R6 bars). The wall was designed using the ductile detailing requirements of the New Zealand Concrete Structures Standard – Amendment 3 [10]. A 50% scale factor must be used with the final cross-section consisting of a 2 m length and 0.175 m thickness, as shown in Fig. 6. The test wall height was obtained from the effective height defined as the ratio between moment and shear at the wall base. For the scaled wall this resulted in a wall height of 4 m. The longitudinal boundary reinforcement consisted of 6 HD16 bars and 2 HD10 bars, confined by D6 stirrups and cross ties spaced at 80 mm (s/d = 5). The web longitudinal reinforcement consisted of HD10 bars spaced at 300 mm centers and restricted with D6 crossties spaced at 125 mm. Tthe horizontal reinforcement consisted of HD10 bars spaced at 125 mm centers.

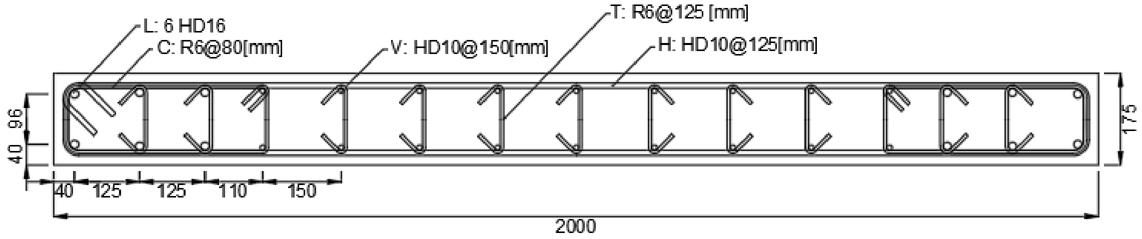


Figure 6: Cross section of the wall specimens.

Four identical test walls were constructed to study the influence of the loading protocols and the repair technique on the wall response, as summarised in Table 2. The first wall (CYC) will be subjected to a standard reverse cyclic loading protocol. In order to achieve the target level of moderate damage before the repair, a drift value between the drift at which spalling and buckling of the longitudinal boundary reinforcement steel occurred will be used as the target peak drift for the earthquake demand in the other specimens. After failure, the wall will be repaired using a technique from category II of Table 1 and retested. The second wall (UNDAM) will be subjected to a similar loading protocol as CYC, but the drifts below the target damage drift removed. The third wall (DAM) will be subjected to a simulated earthquake displacement demand extracted from a numerical model and scaled to achieve the target damage drift. After this, the wall will be tested cyclically until failure. Then the wall will be repaired using a repair technique in category III in Table 1 and retested. The fourth wall (REP) will be subjected to the same loading protocol of the third wall, but epoxy injection and epoxy mortar will be used as a repair technique between the earthquake demand and the cyclic displacement. Fig. 7 shows a scheme of the loading protocol of each specimen.

Table 2: Test matrix

Wall	**Testing sequence**				
	1st Test			**Repair Category II or III**	**2nd Test**
	1st part	**Repair Category I**	**2nd part**		
CYC	Small cycles	No	Big cycles	Yes (II)	Same as 1st Test
UNDAM	-	No	Big cycles	Yes (III)	
DAM	Earthquake	No	Big cycles	Yes (III)	
REP	Earthquake	Yes	Big cycles	No	-

In all cases, the test setup will consist of a cantilever wall anchored at the base to the laboratory strong floor through a foundation block. A capping beam will be attached to the top of the wall to connect with a loading beam. Two vertical actuators and one horizontal actuator will be connected to the loading beam to apply constant axial force (4% axial force ratio) and controlled pseudo-static horizontal displacements. A drawing of the proposed test setup is shown in Fig. 8

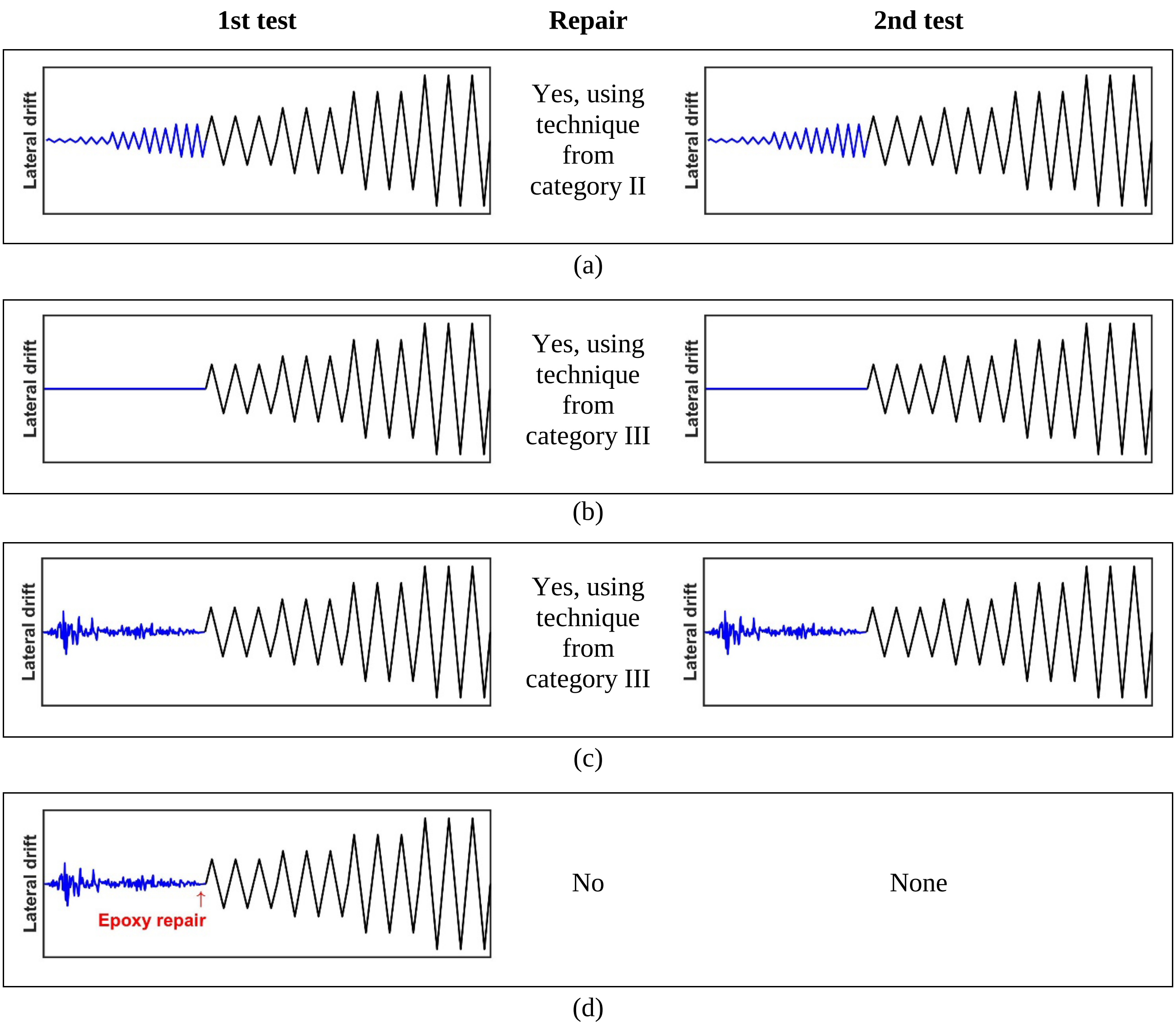


Figure 7: Scheme of the loading protocols (a) CYC, (b) UNDAM, (c) DAM and, (d) REP.

The instrumentation for the test walls is shown in Fig. 9. The global displacement measurements will consist of 4 draw wires and 1 vertical Linear Variable Differential Transformer (LVDT) at different heights, 2 vertical Linear Variable Differential Transformer (LVDT) to measure body rotation at foundation level and, 1 LVDT to measure sliding. In addition, 2 horizontal displacement transducers (called Portal Gauge, or PGs) will be used to measure strain penetration at wall-foundation interface, 1 horizontal PG to measure shear sliding at the wall base, and 2 draw wires to capture axial elongation. Local response measurements will consist of vertical sets of PGs and linear pots to capture longitudinal deformation at wall edges, and diagonal couples of PGs to capture shear distortions. The strain in the reinforcing bars will be recorded using 8 strain gauges in the outer longitudinal boundary reinforcement. Finally, a Digital Image Correlation analysis will be carried on in the other face of the wall.

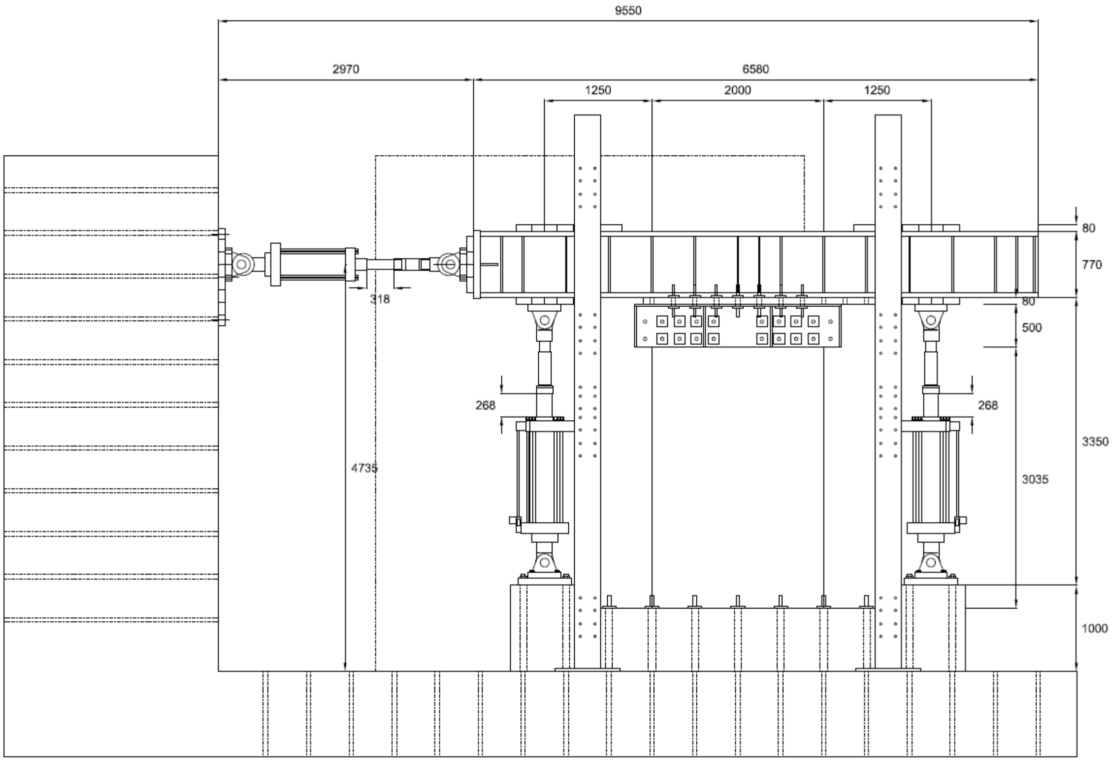


Figure 8: Test setup drawing.

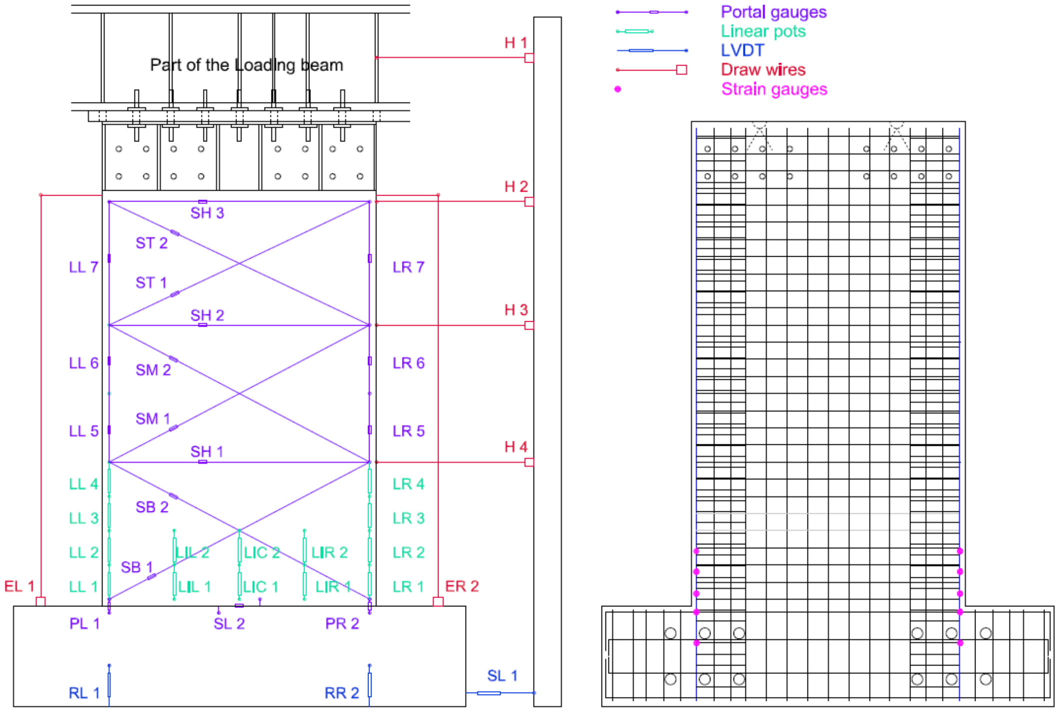


Figure 9: Instrumentation.

4. Conclusions

A literature review was conducted to identify previous tests of repaired RC walls, with a total of 70 test walls from 27 studies dating back to 1975. Given the common practice of testing RC components using increasing cyclic demands until loss of lateral strength, the observed damage in the test walls is generally more severe than the damage observed in real buildings after earthquakes. Only 3 previously tested walls had damage classified as moderate and were repaired with simple techniques, such as epoxy injection, showing a stiffness recovery close to 63% compared to the original value. However, the walls were tested without a benchmark wall, so the effect of the repair on the displacement capacity and lateral strength were not able to defined. When the repair methodology aimed to achieve the same performance of the original design, the strength recovery was 93%, and the displacement capacity recovery was 113%. When any type of strengthening was conducted on the wall during the repair process, the strength recovery was 112%, and the displacement capacity recovery was 109%.

An ongoing experimental program being conducted in the University of Auckland will test four identical ductile RC walls designed in accordance with New Zealand standards for seismic actions and RC design. The first test in the test matrix will allow for evaluation of the residual capacity and the effectiveness of epoxy injection in moderately damaged RC walls. The second test will allow the capability of more complex repair techniques applied to heavily damaged RC walls to achieve similar and increased performance in terms of their stiffness, strength, and displacement capacity. The target damage drift defined as being between the drift when spalling and longitudinal reinforcement buckling occurred is expected to be the threshold to achieve similar strength and displacement capacity between the undamaged, the damaged, and the simply repaired wall.

5. Acknowledgements

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