12TH CANADIAN CONFERENCE ON EARTHQUAKE ENGINEERING
12ÈME CONFÉRENCE CANADIENNE DU GÉNIE PARASISMIQUE
JUNE 17-20, 2019 CHÂTEAU FRONTENAC QUÉBEC, QC.
17-20 JUIN 2019
CCEE 2019

Repair of Earthquake-Damaged Concrete Buildings

Kai Marder[1], Mehdi Sarrafzadeh[2], Kenneth J. Elwood[3]

[1] Engineer, COWI UK, London, United Kingdom.
[2] Ph.D. Student, Department of Civil and Environmental Engineering, University of Auckland - Auckland, New Zealand.
[3] Professor, Department of Civil and Environmental Engineering, University of Auckland - Auckland, New Zealand.

ABSTRACT

A prominent challenge following the 2010-11 Canterbury earthquakes was the insurance decision making process for earthquake-damaged buildings and lack of robust engineering guidelines for future risk assessment. Similarly, in the aftermath of the 2016 Kaikoura earthquake, these issues were again highlighted leaving engineers and building owners with limited guidance on the reparability of moderately damaged reinforced concrete (RC) structures in Wellington. Research is ongoing at the University of Auckland with experimental investigations into the post-earthquake residual capacity and impact of epoxy injection on the behaviour of RC beam elements following earthquake damage. Dynamic and pseudo-static testing of cantilever beam elements and beam-column assemblies extracted from buildings damaged in the Kaikoura earthquake, have shown the potential for recovery of strength and energy dissipation capacity of components repaired via epoxy injection. Damage sustained by these specimens included significant cracking, delamination of cover concrete and beam elongation. This paper outlines the results of the experimental work undertaken thus far, as well as how this research contributes to the development of a proposed framework for post-earthquake building assessment.

Keywords: Reinforced Concrete, Repair, Epoxy, Beam

INTRODUCTION

Following the 2010/2011 Canterbury the resilience of New Zealand's built infrastructure has been a topic of interest. Although the majority of structures achieved life-safety performance objectives in line with current code provisions, over 65% of the concrete buildings in the Christchurch CBD were demolished [1]. Another prominent issue was the prolonged period that buildings were left unoccupied during the decision-making process. The limited availability of post-earthquake reparability guidance for damaged RC structures in New Zealand was undoubtedly a factor in the high number of demolitions and the drawn-out recovery period following these major seismic events. Following the 2016 Kaikoura earthquake in 2016, the ductile behaviour of RC frame buildings in Wellington also led to damage to precast floors [2]. In addition to external factors, the damage to precast flooring systems in these structures and limited information on assessing the residual capacity and reparability of these components made the decision to repair such structures difficult, leading to demolition in several cases.

Similar to Canada, RC buildings in New Zealand are typically designed to behave in a ductile manner during earthquakes. Structural systems are designed and detailed to withstanding high ductility demands for a design-level earthquake (500 year return period), allowing them to dissipate energy and achieve their life safety performance objectives. During significant ground motions these ductile RC buildings are expected to form plastic hinges which is in line with the observations of damage from the Christchurch and Kaikoura earthquakes. But the question remains as to the residual capacity of such ductile plastic hinges and the repairability of damaged buildings. In 2016 a draft framework for the assessment of residual capacity of earthquake-damaged concrete buildings was developed in New Zealand [3] which builds on the model of FEMA 306 [4]. The framework, illustrated in Figure 1, provides the key steps for a detailed assessment of an earthquake-damaged concrete building. This is intended to be used for buildings which exhibit damage in the seismic force resisting system consistent with good energy dissipation and flexural hinges. Challenges remain for the implementation of this framework as discussed in [3], most notably the limited experimental data on repaired concrete components. This paper discusses the results of experimental programmes on the repair of ductile plastic hinges in RC beam elements via epoxy injection and discusses the viability of such repair techniques during post-earthquake recovery.

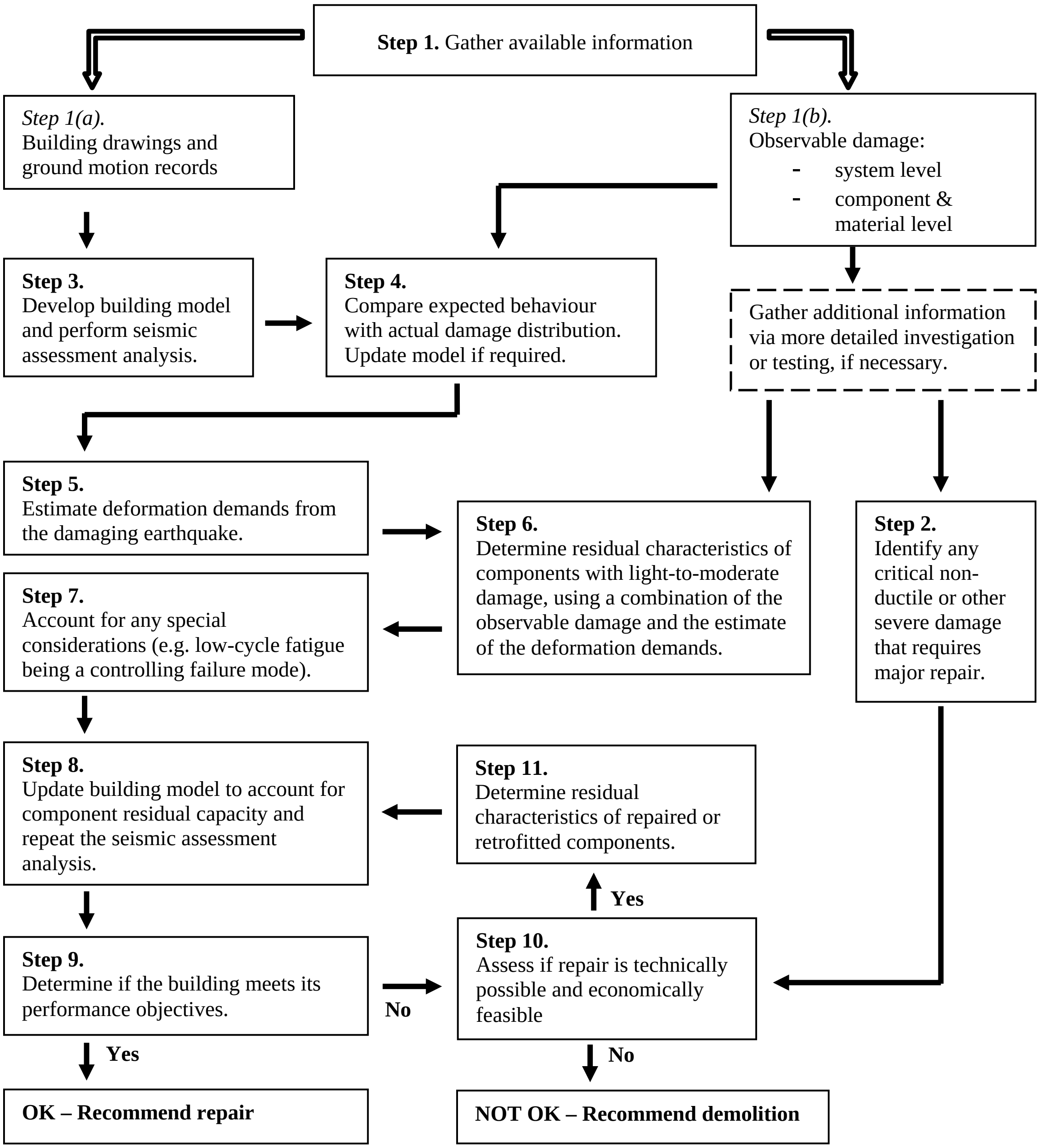


Figure 1. Proposed methodology for the detailed assessment of earthquake-damaged reinforced concrete buildings [3]

Existing Literature on Epoxy Injection of RC Elements

Epoxy injection and other simple repair techniques, such as fibre reinforced polymer (FRP) sheeting, can be less costly and labour intensive to apply than more comprehensive repairs, such as hydro demolition and reconstitution of concrete elements and replacement of reinforcement. The scope of this research is hence limited to damage which can be classified as low to moderate where simple techniques are feasible. In this study moderate damage is considered as a state in which reinforcement in the plastic hinge region has not undergone buckling or rupture and no crushing of the core concrete is evident, comparable to the moderate damage state defined in FEMA 306 guidelines for the repair of ductile walls and coupling beams [4]. Review of past literature identified seven experimental investigations which fit within the scope of the study, applying simple repair techniques on ductile RC elements following moderate damage. These studies, summarized in Table 1, investigated the effectiveness of repair via epoxy injection and FRP sheeting on the cyclic behaviour of various RC elements. While this paper will be focusing on the use of epoxy injections in beam elements, the repair of walls via FRP sheeting is included here to demonstrate the viability of such repair techniques for various types of elements in damaged buildings. Based on this limited data set, repair via epoxy injection was found to be effective in restoring, and in fact increasing, the strength of the components in comparison to their undamaged state. Stiffness was found to be largely recovered with a lower bound of ~85% for RC beams. The test involving a bridge column however was not able to adequately recover the stiffness of the element with a stiffness recovery ratio of 50% compared to original stiffness. This could potentially be attributed to the reduced penetration of epoxy injection due to closing of cracks by the axial load in the column. The deformation capacity and cycle-cycle energy dissipation capacity of the elements was also able to be recovered in most cases. Further discussion of the epoxy-injection tests can be found in [5].

Table 1. Summary of Experimental Results from Existing Literature

	Epoxy Injection					FRP Sheeting	
	Interior B/C Joints			Exterior B/C Joints	Columns	Walls	
	[6]	[7]	[8]	[9]	[10]	[11]	[12]
Yield Secant Stiffness Recovery Ratio	1.1	0.85-0.89	0.9-1	0.93-1.03	0.5	0.9	0.87
Strength Recovery Ratio	1.1-1.2	1.01-1.05	1.05-1.1	1.24-1.28	1	1.29	1.55
Deformation Capacity Recovered?	Reduced*	Unclear	Unclear	Unclear	Yes	Yes	Yes
Energy Dissipation Capacity Recovered?	Yes	Reduced**	Yes	Yes	Yes	Yes	Yes
Number of Specimens	4	2	2	2	1	1	2

Notes:
* Low-cycle fatigue may have influenced this outcome with a high number of post-yield cycles having been applied during the testing.
** Reduction in energy dissipation capacity was attributed to loss of reinforcement anchorage in the joint region due to lack of penetration of epoxy during the repair of the joint cracks.

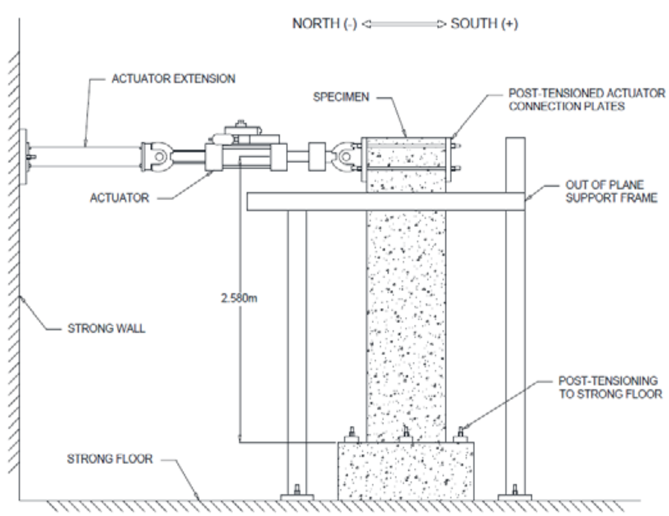


Figure 2. General Test Setup

EXPERIMENTAL PROCEDURE

A set of tests was carried out at the University of Auckland on seventeen large-scale, nominally identical cantilevered RC beam specimens, investigating a variety of parameters relating to post-earthquake residual capacity and reparability. As part of these tests, three specimens were subjected to a damaging long-duration earthquake input followed by repair via epoxy injection and further testing to failure. This section outlines the general test setup and the procedure used to simulate earthquake damage and assess effectiveness of repair. More details on the test specimens and loading protocol can be found in Marder et al. [13].

Test Specimen and Loading

All seventeen specimens were 0.8 scale replicas from the second story beams on the perimeter frame of the prototype frame building from the Bull and Brunsdon [14] guideline for RC buildings in New Zealand. The building was designed in accordance to the New Zealand concrete standard, NZS 3101:2006 [15] and the New Zealand seismic loading code, NZS 1170.5:2004 [16] to a ductility $\mu = 4$, resulting in a design base shear equal to 3.2% of the weight of the structure. The length of the beam specimen corresponds to one half of the length from the face of the column to midspan of the frame. All beams were detailed in accordance with ductile detailing requirements of NZS 3101:2006. Beams had a design longitudinal reinforcement ratio of 0.6% (~1.25 times minimum requirement) with cross sectional dimensions of 320x720mm and a shear span ratio, M/V of 2.58.

The test setup is shown in Figure 2. In order to approximate the restraint to beam elongation that exists in a frame structure, axial restraint was also provided for specimen LD-2-LER and LD-2-LER-R through the use of a spreader beam placed on top of the specimen distributing elongation-induced axial load to two high strength post-tensioning rods connected to the laboratory strong floor via pin connections. Bellville springs were added to the rod-spreader beam interface for these two specimens to limit the restraint level. Other specimens had no restraint to beam elongation

The test matrix shown in Table 2, outlines the test parameters on eight of the seventeen tested specimens considered in this paper. These include three beams subjected to damage and repair, three equivalent beams which were subjected to the same damaging excitation followed by testing to failure **without** repair, and two control specimens (*MONO* and *CYC*) with monotonic and cyclic loading protocol. The cyclic loading protocol was a modified version of that proposed by Park [17]. One cycle was applied at 0.75 and 1.25 times the yield displacement, followed by single cycles starting from displacement ductility $\mu = 2$ to $\mu = 10$ at increments of $\mu = 1$. This was followed by cycles at increments of $\mu = 2$ to failure.

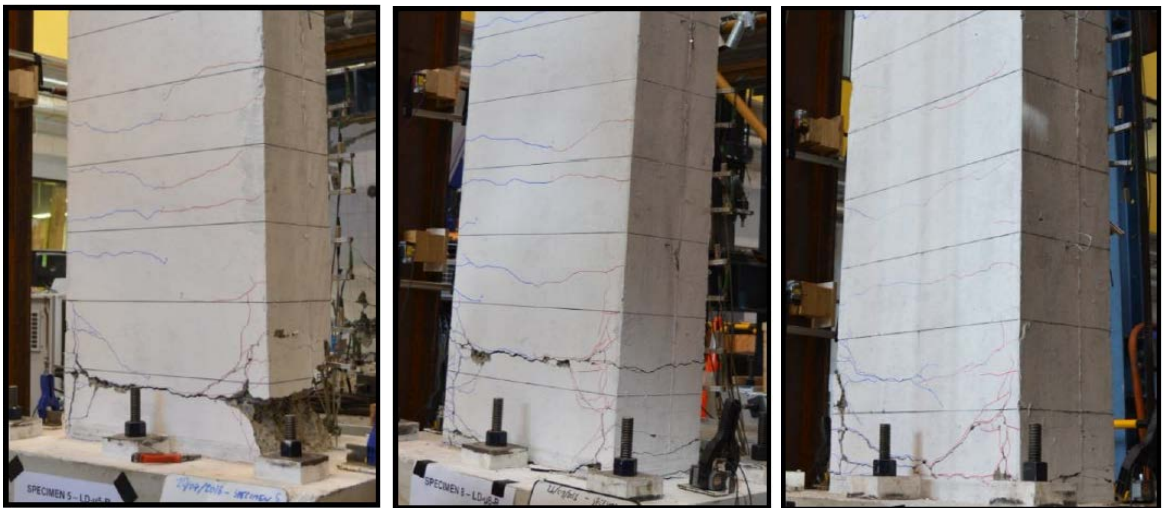
Table 2. Test Matrix

Specimen Name	Initial Damaging Earthquake Loading and peak drift	Failure Loading	Repair	Elongation Restraint
MONO	-	Static Monotonic	No	No
CYC	-	Static Cyclic	No	No
LD-1	Long duration, 1.36% drift	Static cyclic (cycles above 1.36%)	No	No
LD-2	Long duration, 2.17% drift	Static cyclic (cycles above 2.17%)	No	No
LD-2-LER	Long duration, 2.17% drift	Static cyclic (cycles above 2.17%)	No	~15kN/mm
LD-1-R	Long duration, 1.36% drift	Static cyclic (cycles above 1.36%)	Yes	No
LD-2-R	Long duration, 2.17% drift	Static cyclic (cycles above 2.17%)	Yes	No
LD-2-LER-R	Long duration, 2.17% drift	Static cyclic (cycles above 2.17%)	Yes	~15kN/mm

The loading protocol for the non-control specimens was run in two phases. Initially, a damaging displacement history would be dynamically applied to the beam followed by a second loading phase of standard quasi-static cyclic loading to failure. Repair was undertaken between the two loading phases to applicable specimens. The displacement histories applied to the beams were based on output from a non-linear time-history analysis of the prototype building in OpenSees. Two long duration displacement histories were applied to the beams, LD1 (Peak drift 1.36%) and LD-2 (2.17%) corresponding to the specimen names. For the failure loading phase, cyclic loading was only applied at cycles above the peak drift to which the beams were subjected during initial dynamic loading.

Damage State Prior to Repair

Figure 3 shows the damage state of the three repaired beams prior to application of epoxy repair. Specimen LD-2-R and LD-2-LER-R both exhibited flexural cracking, longitudinal cracks along the longitudinal reinforcement, as well as delamination of cover concrete. Specimen LD-1-R exhibited more concentrated damage in the form of a sliding plane crack near the base of the beam. LD-1-R also exhibited delamination of cover concrete despite being subjected to a lower peak drift of 1.36% compared to 2.17% for the other two specimens. It is worth noting that the non-repaired specimen for LD-2-LER exhibited similar damage to its repaired counterpart following dynamic loading. Specimen LD-1-R exhibited more severe damage to its un-repaired counterpart which did not see a sliding plane crack or delamination of cover concrete. Specimen LD-2-R exhibited a less severe damage state to its un-repaired counterpart which saw concentrated damage in the form of a sliding plane crack. This variability in damage for the same drift demands should be taken into account when assessing the effectiveness of the epoxy injection. The maximum residual crack widths in the three repaired specimens ranged from 2.5-3.5mm prior to repair.



(a) (b) (c)

Figure 3. Pre-repair damage sate for specimen (a) LD-1-R (b) LD-2-R (c) LD-2-LER-R

Repair Methodology

Repair was undertaken under the supervision of an experienced specialist contractor. The procedure involved the removal of all loose or delaminated cover concrete and reinstatement using a self-compacting high-early strength repair mortar. Epoxy injection ports were then installed at all cracks above 0.2mm wide and surface of cracks sealed using an epoxy resin putty. Low viscosity epoxy resin was then hand pumped into cracks until no longer possible. Resin was allowed to set, followed by re-profiling of the surface of the beam. It should be noted that residual displacements were not corrected prior to repair.

RESULTS

Post-Repair Damage State

The damage state of the three beams following repair showed an increase in the distribution of cracks in the plastic hinge region. Measurable cracks (>0.2mm) were distributed over a length up to 530mm from the beam-foundation interface, in comparison to an average length of 430mm seen in the non-repaired specimens. While this indicates a slight shift or lengthening of the plastic hinge, a complete relocation of the plastic hinge region was not observed as was reported in some of the previous literature [6-8]. Crack length distribution was measured in all specimens following both cycles to 2.44% drift, which corresponded to the lowest post-repair drift increment applied to specimens LD-2-R and LD-2-LER-R. Longitudinal cracks re-appeared at the location of prior delamination where repair mortar was applied. Epoxy resin was largely effective in keeping injected cracks from re-opening. At higher drifts and increased damage progression, some epoxied cracks did re-open, particularly at locations where horizontal sliding plane cracks were repaired; however, specimen LD-1-R did exhibit a wider distribution of cracks, despite the formation of a sliding plane crack prior to repair (as discussed previously and shown in Figure 3a).

Stiffness

Comparisons of the relative stiffness increase between repaired and damaged beams saw an increase of up to 250% in repaired beams. Specimens LD-2-R and LD-2-LER-R saw higher relative increases than specimen LD-1-R, due to the higher peak drift imposed on the beams during the dynamic loading phase (2.17% vs 1.36%). Comparisons between the stiffness of specimens LD-1-R and LD-2-R with undamaged specimens subjected to cyclic loading, showed a secant stiffness to yield between 79-88% of the undamaged specimens. Similarly, for Specimen LD-2-LER-R, comparison to equivalent cyclic restrained test showed a secant stiffness to yield, approximately 85% of the undamaged specimen. The reduction in recovered stiffness is likely a result of the loss of the tension stiffening effect due to the inability of epoxy resin to penetrate finer cracks below 0.2mm in width which were not injected. Figure 4 is a plot of the recovered stiffness vs displacement ductility prior to repair for the three repair tests as well as the test identified in previous literature (Table 1). The plot shows no clear relationship between prior peak ductility displacement and recovered stiffness. Based on these results, with the exception of the column specimen (test 4 in Figure 4), a lower bound secant stiffness to yield of 80% of an undamaged equivalent can be recommended for beams with little or no axial load. This recommendation is in line with FEMA 306 recommendations for walls [3].

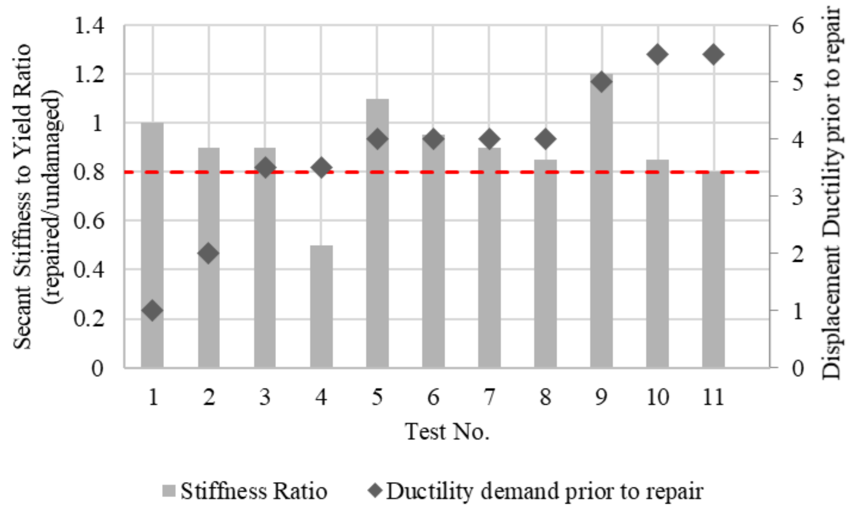


Figure 4. Post-repair secant stiffness to yield ratio vs. maximum displacement ductility prior to repair.

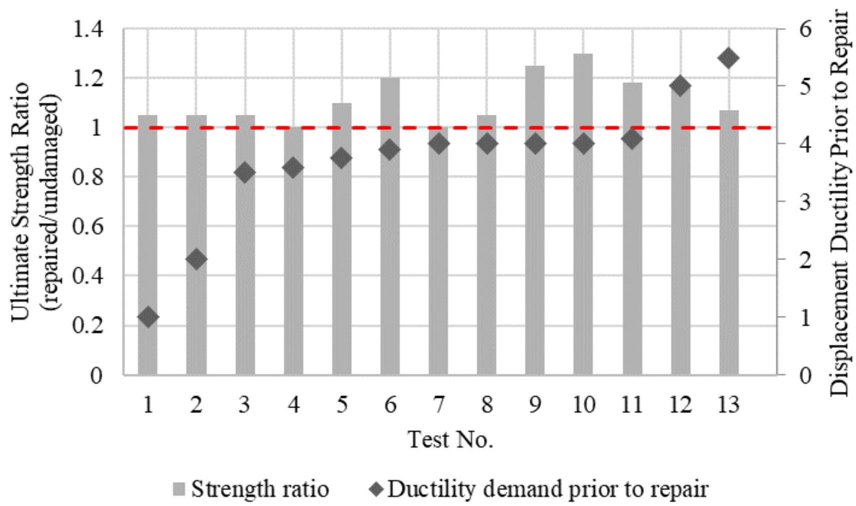


Figure 5. Post-repair ultimate strength ratio vs. maximum displacement ductility prior to repair.

Strength

All three repaired specimens exhibited ultimate strengths post repair that were similar or higher than equivalent unrepaired tests, albeit still lower than the over-strength calculated from NZS 3101:2006. Comparisons of the peak strength for specimens LD-1-R and LD-2-R to average strength of all un-restrained tests showed a post repair strength ratio of 1.04 and 1.07 respectively, while specimen LD-2-LER-R had a peak strength ratio of 1.07, in comparison to the average strength of all tests with a similar restraint level. Figure 5 shows a comparison of the ultimate strength ratio to the maximum displacement ductility prior to repair for all repair specimens from this study and previous literature (Table 1). Similar to stiffness, no strong correlation can be seen between prior displacement ductility and post repair strength. The ultimate strength ratio for all fourteen specimens subjected to epoxy injection ranges between 1-1.3 with an average of 1.1. An approximate lower bound strength ratio of 1.0 can be recommended based on the data in Figure 5. The increase in flexural strength in these tests can be attributed to both a shifting of the plastic hinge away from the joint face and the strain aging and strain hardening of steel reinforcement. Epoxy resin is unlikely to impact the flexural strength of elements since following repair cracks have been observed to form in the concrete adjacent to the epoxied cracks. Effects of strain aging and strain hardening, particularly in sections with large residual elongations (hence residual strains) should be considered when considering repair of RC beams. Significant increases in strength beyond over-strength are conceivable with a combination of such factors which may lead to brittle failure of structures during high displacement demands. Further research is required to quantify the impact of strain aging and hardening on the performance of repaired beams.

Energy Dissipation and Deformation Capacity

The cycle-cycle energy dissipation capacity was seen to increase for all three repair specimens in comparison to both the average of all cyclic specimens as well as the unrepaired specimens. Specimen LD-2-R maintained this throughout testing, while LD-1-R and LD-2-LER-R degraded to be similar to other specimens at higher drift cycles. The deformation capacity of the specimens was assessed via comparison to other specimens showing similar damage progressions. Specimens LD-2-R and LD-2-LER-R saw a deformation capacity similar to all beams exhibiting similar damage progression, regardless of repair, while specimen LD-1-R saw an increase in deformation capacity relative to its unrepaired counterparts. All three repaired beams experienced a 20% drop in resistance at the cycle to 4.34% drift, in both the positive and negative cycles. Specimens which developed sliding plane cracks, like specimen LD-1-R, prior to repair, had relatively low deformation capacities, ranging from 3.26-3.8%.

ONGOING TESTING - EPOXY INJECTION OF EXTRACTED BEAM ELEMENTS

Following the 2016 Kaikoura Earthquake the decision was made to demolish several RC structures with moderate damage in the Wellington CBD. In one such case, the opportunity was presented for the extraction of beam-column joints from the structure during demolition for further investigation. This structure was within approximately 200m of the Wellington waterfront and was impacted by basin amplification effects. Designed between 2003-2005, the building was an eight storey RC perimeter moment frame structure with an “L” shaped floor plan. A precast Hollowcore flooring system with RC topping was used and the structure was designed to a ductility of 6 (maximum permissible), expected to form plastic hinges in the perimeter frame beams in the event of a significant earthquake. As part of the investigation, two exterior and two interior beam column joints were extracted from the fifth floor of the frame. Testing is still underway on these specimens, however, preliminary results based on the two exterior joints are summarized in Table 3 below. Beams were tested in a similar cantilever configuration as the specimens discussed in the previous sections of this paper with an M/V ratio of 2.1, comparable to the aspect ratio seen in the building frame. Earthquake damage to the two exterior joints was comparable, both seeing cracking up to 5mm in width. The beam selected for repair (Specimen E-R) also exhibited spalling of cover concrete exposing longitudinal reinforcement but showed no signs of bar buckling. Specimen E-R was repaired by an experienced contractor via epoxy crack injection and reconstitution of cover concrete using epoxy mortar (similar to repaired beam tests discussed previously).

Both E-U and E-R exhibited distributed cracking along the plastic hinge length, dominated by diagonal cracking and large shear deformations. This contrasted with the tests discussed previously which saw predominantly flexural cracking and tendency for sliding shear failures. Initial test results indicate an increase in stiffness for the repaired beam (E-R) of approximately 25.5% in comparison to the damaged beam (Specimen E-U). The repaired beam also saw an increase of ~15% in yield strength and ~12% in the ultimate strength following repairs. The predicted yield and overstrength capacity of the beams are also given in Table 3. The results show up to a 25% increase in the strength of the specimens relative to the calculated overstrength. This indicates that prior strain hardening and strain aging of the longitudinal reinforcement seem to have impacted the strength of the components. The increase in strength of the beams however did not impact the deformation capacity, with both specimens failing at the end of their second cycle to 6% drift without fracture of any reinforcement. The effects of strain-aged longitudinal reinforcement on beam plastic hinge behaviour are further discussed by Marder [5].

Table 3. Summary of Test Results on Extracted Beam-Column Joint Repair

Specimen Name	Repair?	Calculated Yield* (kN)	Calculated Overstrength* (kN)	Yield Strength (kN)	Ultimate Strength (kN)	Stiffness (kN/mm)	Drift at failure (%)
E-U	No	500	680	744	866	35.7	6
E-R	Yes	500	680	853	968	44.8	6

* calculated in accordance with NZS 3101:2006 [15]

CONCLUDING REMARKS

Based on the experimental results outlined in this paper, the effectiveness of epoxy injection on restoring the behaviour of ductile RC beams has been outlined. The stiffness, strength, energy dissipation and deformation capacity of damaged RC beams were largely restored with only stiffness not being fully recovered. Based on the dataset of repaired beams available, it is recommended that stiffness of repaired beams be conservatively taken to be 80% of undamaged stiffness and the ultimate flexural strength be assumed to be 100% of undamaged strength. Larger strength increases, higher than the expected over-strength, due to strain hardening and strain aging of reinforcement, can cause relocation of plastic hinge regions away from joints which must be accounted for when assessing the repaired structure. It is therefore crucial that residual strains and steel properties be considered when making decisions on repair of damaged RC elements. Further experiments are currently underway at the University of Auckland on earthquake damaged beam elements extracted from a RC building damaged in the

2016 Kaikoura earthquake. The results of these tests will provide further data on the effectiveness of epoxy repair as well as the impacts of strain aging on the post-repair beam behaviour. An experimental and analytical program is planned using component-level testing and non-linear building models to investigate the impact of repair on global building performance. This will build on the results discussed in this paper to provide further guidance on the viability of repair via epoxy injection for moderately damaged RC buildings. Additionally, the impact of New Zealand's high ductility design philosophy on the repairability of RC structures will be investigated. The development of a "**Repairability Limit State**" (RLS) for new design would enable designers to shift the performance criteria to achieve faster re-occupancy following simple repairs, a move away from high ductility demands. The objective of this research is to target an improved resilience of New Zealand's building infrastructure, particularly in high seismic zones such as Wellington and Christchurch where prolonged post-earthquake recovery has had significant impacts on the communities.

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