EFFECTIVENESS OF REPAIR VIA EPOXY INJECTION OF EARTHQUAKE DAMAGED REINFORCED CONCRETE BEAM ELEMENTS

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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 earthquakes, 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 further experimental and analytical work currently underway.

Introduction

Following both the 2010/2011 Canterbury and 2016 Kaikoura earthquakes, 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, many low to moderately damaged RC structures were demolished following these earthquakes (Marquis et al. 2015). 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 which occurred and the drawn-out recovery period following these major seismic events. 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, allowing them to dissipate the seismic 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. Following the Kaikoura earthquake in 2016, the ductile behaviour of RC frame buildings was of particular concern for engineers with a significant number of mid-rise RC frame structures exhibiting formation of ductile beam plastic hinges with associated beam elongation (Henry et al. 2017). In addition to external factors, and the performance of precast flooring systems in these structures, 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.

To aid in the recovery and resilient performance of our built environment, the use of timely and simple repair methods requires further investigation. 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.

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 (FEMA 1998). 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 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 was included here to demonstrate the viability of such repair techniques for various types of elements in damaged buildings. A summary of the results of these experiments is provided in Table 1. Based on this limited data set, repair via epoxy injection was found to be effective in restoring and 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%. The test involving a bridge column however was not able to adequately recover the stiffness of the element with a 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 Marder (2018).

Table 1. Summary of Experimental Results from Existing Literature

<table>
<tr><th rowspan="3"></th><th colspan="5">Epoxy Injection</th><th colspan="2">FRP Sheeting</th></tr>
<tr><th colspan="3">Interior B/C Joint</th><th>Exterior B/C Joint</th><th>Columns</th><th colspan="2">Walls</th></tr>
<tr><th>(Celebi and Penzien 1973)</th><th>(French, Thorp, and Tsai 1990)</th><th>(Cuevas and Pampanin 2017)</th><th>(Lee 1976)</th><th>(Lehman et al. 2001)</th><th>(Lombard, Lau, and Humar 2000)</th><th>(Cruz-Noguez et al. 2015)</th></tr>
<tr><td>Yield Secant Stiffness Recovery Ratio</td><td>1.1</td><td>0.85-0.89</td><td>0.9-1</td><td>0.93-1.03</td><td>0.5</td><td>0.9</td><td>0.87</td></tr>
<tr><td>Strength Recovery Ratio</td><td>1.1-1.2</td><td>1.01-1.05</td><td>1.05-1.1</td><td>1.24-1.28</td><td>1</td><td>1.29</td><td>1.55</td></tr>
<tr><td>Deformation Capacity Recovered?</td><td>Reduced*</td><td>Unclear</td><td>Unclear</td><td>Unclear</td><td>Yes</td><td>Yes</td><td>Yes</td></tr>
<tr><td>Energy Dissipation Capacity Recovered?</td><td>Yes</td><td>Reduced**</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td><td>Yes</td></tr>
<tr><td>Number of Specimens</td><td>4</td><td>2</td><td>2</td><td>2</td><td>1</td><td>1</td><td>2</td></tr>
</table>

Notes: * Potential impact of low-cycle fatigue could be attributed to this 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.

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 loading 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. (2018).

Test Specimen. 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 (2008) guideline for RC buildings in New Zealand. The building was designed to a ductility $\mu = 4$, structural performance factor, $S_p = 0.7$, with a design base shear equal to 3.2% of the weight of the structure. The building was designed in accordance to the New Zealand concrete standard, NZS 3101:2006 (Standards New Zealand 2006) and the New Zealand seismic loading code, NZS 1170.5:2004 (Standards New Zealand 2004). The length of the beam specimen corresponds to one half of the length from the face of the column to the expected point of inflection in the bay 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.

Test Matrix. The test matrix shown in Table 2, outlines the test parameters on eight of the tested specimens. These included 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 with monotonic and cyclic loading protocol.

Table 2. Test Matrix

Specimen Name	*Initial Damaging Earthquake Loading*	*Failure Loading*	*Repair*	*Elongation Restraint*
MONO	-	Static Monotonic	No	No
CYC	-	Static Cyclic	No	No
LD-1	Dynamic long duration displacement history to 1.36% drift	Static cyclic (cycles above 1.36% drift only)	No	No
LD-2	Dynamic long duration displacement history to 2.17% drift	Static cyclic (cycles above 2.17% drift only)	No	No
LD-2-LER	Dynamic long duration displacement history to 2.17% drift	Static cyclic (cycles above 2.17% drift only)	No	Yes ~15kN/mm
LD-1-R	Dynamic long duration displacement history to 1.36% drift	Static cyclic (cycles above 1.36% drift only)	Yes	No
LD-2-R	Dynamic long duration displacement history to 2.17% drift	Static cyclic (cycles above 2.17% drift only)	Yes	No
LD-2-LER-R	Dynamic long duration displacement history to 2.17% drift	Static cyclic (cycles above 2.17% drift only)	Yes	Yes ~15kN/mm

Test Setup. The testing arrangement for the specimens is shown in Figure 1. Beams were tested vertically and post tensioned to the laboratory strong floor using six post-tensioning rods passing through precast ducts. The floor-specimen interface was grouted as well as 300kN tensions force applied to each post tensioning rod. A 300kN capacity displacement-controlled actuator was used to apply the loading, connected at the beam face to steel plates post-tensioned onto the beam. Out of plane movement was inhibited using a steel support frame on either side of the specimen as well as the use of Ultra-high molecular weight polyethylene (UHMWPE) strips to minimise friction at the beam-frame interface. 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 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.

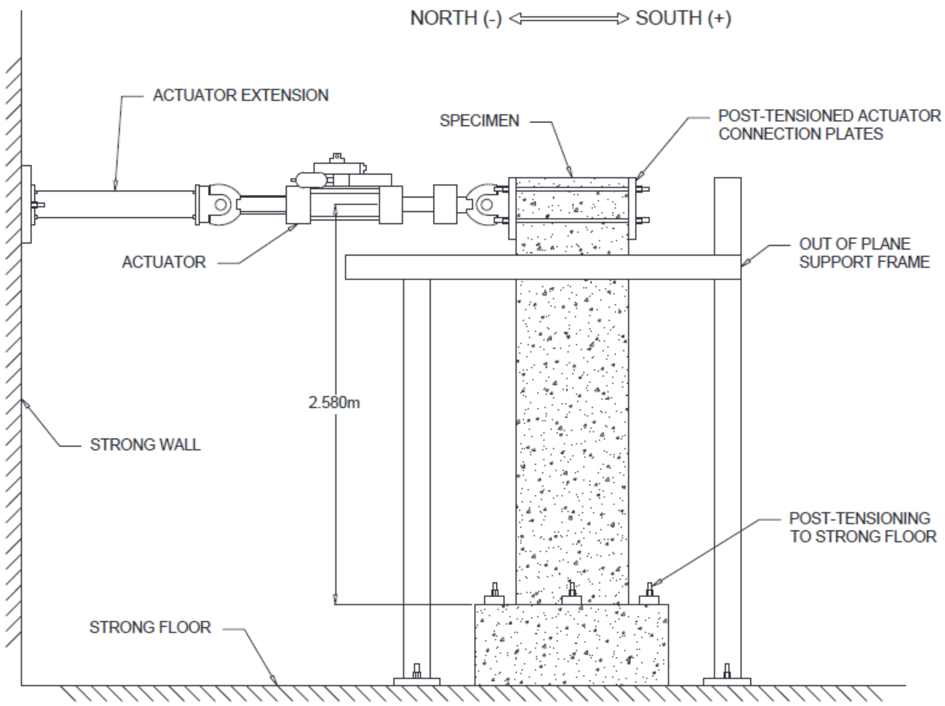

Figure 1. General test setup.

Loading Protocol. 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. The cyclic loading protocol was a modified version of that proposed by Park (1989). 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. Cyclic loading was only applied at cycles above the peak drift to which the beams were subjected during dynamic loading.

Damage State Prior to Repair. Figure 2 shows the damage state of the three repaired beams prior to application of the 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 should be taken into context 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.

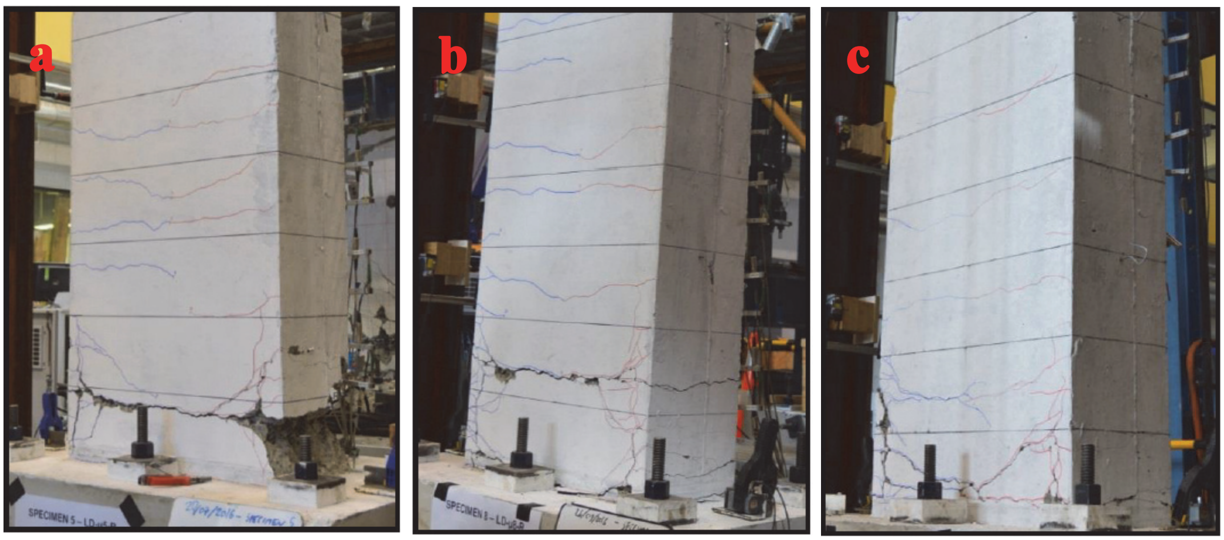

Figure 2. Pre-repair damage state for specimen (a) LD-1-R (b) LD-2-R (c) LD-2-LER-R

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 note 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 as well as the re-appearance of a plastic hinge in the repaired region. A complete relocation of the plastic hinge region was not observed as was reported in some of the previous literature. 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. 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. The distribution of damage, however, did improve in specimen LD-1-R, despite the formation of a sliding plane crack prior to repair.

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%). Specimen LD-1-R however exhibited 10% higher absolute stiffness in comparison to LD-2-R. Comparisons between the stiffness of specimens LD-1-R and LD-2-R 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 tensions stiffening effect due to the inability of epoxy resin to penetrate finer cracks below 0.2mm in width which were not injected. Figure 3 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 a lower bound secant stiffness to yield of 80% of an undamaged equivalent is recommended. This recommendation is in line with FEMA 306 recommendations (FEMA, 1998).

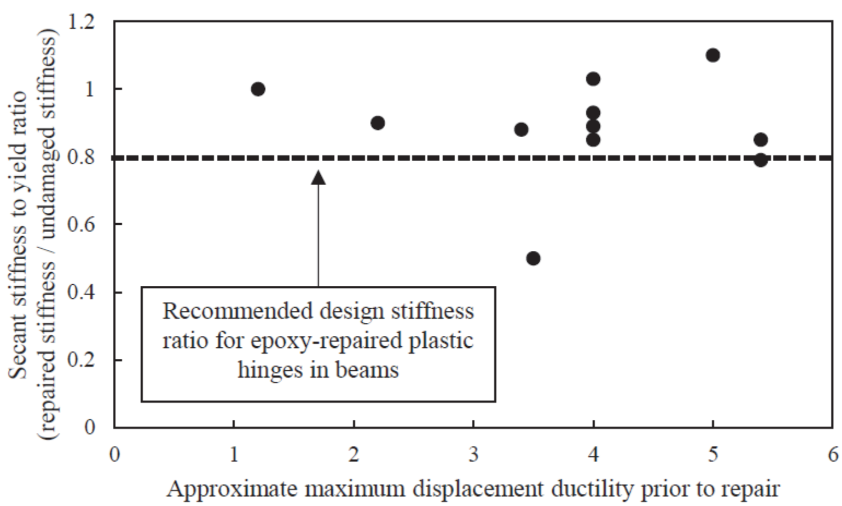

Figure 3. Post-repair secant stiffness to yield ratio vs. maximum displacement Ductility prior to repair.

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 4 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 4. The increase in flexural strength in these tests can largely be attributed to the strain aging and strain hardening of steel reinforcement. Epoxy resin is unlikely to impact the flexural strength of elements as even following repair cracks can form in 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.

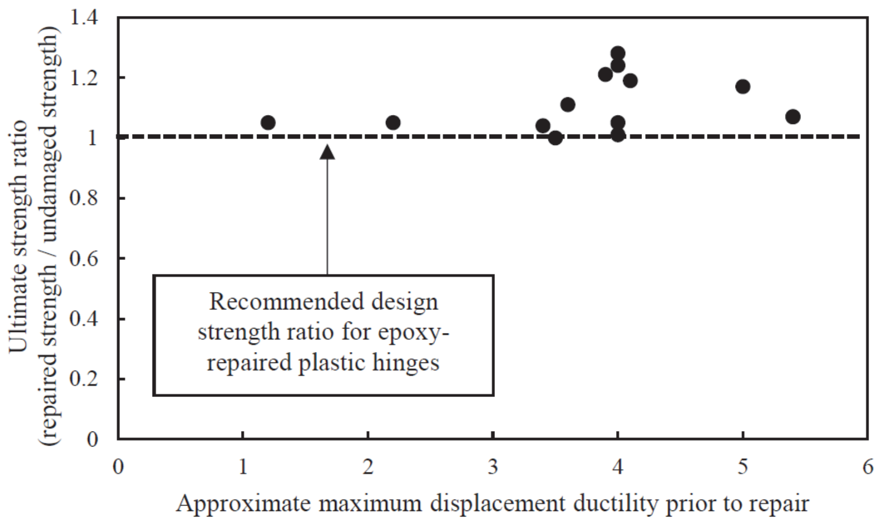

Figure 4. Post-repair ultimate strength ratio vs. maximum displacement Ductility prior to repair.

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 while LD-1-R and LD-2-LER-R degraded to be similar to other specimens at higher 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 recovery in deformation capacity while specimen LD-1-R saw an increase. All three beams experienced a 20% drop in strength at the cycle to 4.34% drift, in both the positive and negative cycles. Specimen which developed sliding plane cracks like specimen LD-1-R prior to repair, had displacement capacities ranging from 3.26-3.8%. Specimens exhibiting damage progressions similar to LD-2-R and LD-2-LER-R saw a drop in strength at comparable drifts.

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 available, it is recommended that post-epoxy repaired stiffness 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 and potentially cause brittle failure. 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 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 have had significant impacts on the communities.

References

Marquis, F., Kim J.J., Elwood, K.J. and Chang, S.E. 2015. "Understanding Post-Earthquake Decisions on Multi-Storey Concrete Buildings in Christchurch, New Zealand." *Bulletin of Earthquake Engineering* 15(2): 731–58.

Henry, R.S., Dizhur, D., Elwood, K.J, Hare, J. and Brunsdon D. 2017. "Damage to Concrete Buildings with Precast Floors during the 2016 Kaikōura Earthquake." *Bulletin of the New Zealand Society for Earthquake Engineering* 50(2): 174–86.

FEMA. 1998. "Evaluation of Earthquake Damaged Concrete and Masonary Wall Buildings." *ATC-43*.

Marder, K.J. 2018. "Post-Earthquake Residual Capacity of Reinforced Concrete Plastic Hinges." (Unpublished doctoroal dissertation) University of Auckland, Auckland, New Zealand.

Celebi, M. & Penzien, J. 1973. "Hysteretic Behavior of Epoxy-Repaired Reinforced Concrete Beams". Earthquake Engineering Research Center, University of California.

French,C.W., Thorp, G.A. and Tsai, W.J. 1990. "Epoxy Repair Techniques for Moderate Earthquake Damage." *ACI Structural Journal* 87(4): 416–24.

Cuevas, A., and Pampanin, S. 2017. "Post-Seismic Capacity of Damaged and Repaired Reinforced Concrete Plastic Hinges Extracted from a Real Building." *Proceedings of the New Zealand Society of Earthquake Engineering Conference, Wellington, New Zealand.*

Lee, D L N. 1976. "Original and Repaired Reinforced Concrete Beam-Column Subassemblages Subjected to Earthquake Type Loading". University of Michigan.

Lehman, D.E., Gookin, S.E., Nacamull, A.M. and Moehle, J.P. 2001. "Repair of Earthquake-Damaged Bridge Columns." *ACI Structural Journal* 98(2): 233–42.

Lombard, J., Lau, D.T., Humar J.L., Foo, S. and Cheung M.S. 2000. "Seismic Strengthening and Repair of Reinforced Concrete Shear Walls." *Proceedings of the 12th World Conferecnce on Earthquake Engineering, Auckland, New Zealand.*

Cruz-Noguez, C.A., Lau, D.T., Sherwood, E.G., Hiotakis, S., Lombard, J., Foo, S. and Cheung, M. 2015. "Seismic Behavior of RC Shear Walls Strengthened for In-Plane Bending Using Externally Bonded FRP Sheets." *Journal of Composites for Construction* 19(1).

Marder, K.J., Motter, C., Elwood, K.J. and Clifton, G.C. 2018. "Testing of Seventeen Identical Ductile Reinforced Concrete Beams with Various Loading Protocols and Boundary Conditions." *Earthquake Spectra*.

Bull, D. K. and Brunsdon, D. (2008). *Examples of Concrete Structural Design to New Zealand Standard 3101*. Cement & Concrete Association of New Zealand. Wellington, NZ.

Standards New Zealand. 2006. "NZS3101 - Concrete Structures Standard Part 1 - The Design of Concrete Structures."

Standards New Zealand. 2004. "NZS 1170.5 - Structural Design Actions Part 5: Earthquake Actions - New Zealand."

Park, R. (1989). "Evaluation of ductility of structures and structural assemblages from laboratory testing." *Bulletin of the New Zealand National Society for Earthquake Engineering, 22*(3), 155-166.