PERFORMANCE OF EARTHQUAKE DAMAGED BEAMS AND EFFECTIVENESS OF REPAIR VIA EPOXY INJECTION

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SUMMARY
A significant challenge for engineers following the 2010-11 Canterbury earthquakes was the decision making and assessment process surrounding moderately damaged reinforced concrete (RC) structures. Similarly, in the aftermath of the 2016 Kaikoura earthquakes, these issues were highlighted, leaving engineers and building owners with limited guidance on the reparability of these buildings, particularly in the Wellington region. Four beam-column assemblies, exhibiting low to moderate damage levels after the 2016 Kaikoura earthquake, were extracted from a RC building in Wellington. Specimens were tested at the University of Auckland to assess their residual stiffness, strength and deformation capacity. One specimen was repaired using epoxy injection and repair mortar by experienced contractors. Results from the tests indicate that the damaged specimens maintained their strength and deformation capacity and demonstrate the impact of epoxy injection on recovery of stiffness in the elements.

INTRODUCTION
The Kaikoura Earthquake
On 14 November 2016, the Mw 7.8 Kaikoura Earthquake, struck the North-Eastern region of the South Island of New Zealand. With an epicentre approximately 100km north of Christchurch, the earthquake resulted in fault rupture propagating north, with surface expression extending approximately 150km. The ground shaking was felt in many regions of New Zealand and even resulted in accelerations exceeding design levels for certain periods in the Wellington region on the North Island (Figure 1).
As discussed by Henry et al. (2017), many reinforced concrete (RC) moment frame structures between 5-15 storeys were impacted, particularly those in regions of deep soil deposits in the Wellington CBD. These regions saw ground motion amplification due to basin-edge effects not previously accounted for in the loading standards (Bradley et al., 2017). Most ductile RC moment frame structures performed largely as intended, with the formation of beam plastic hinges ranging from minor to moderate damage states being observed. In some instances, beam elongation and frame dilation in these structures hindered the performance of widely-used precast flooring systems, even causing collapse of floor units in the Statistics House (MBIE Report, 2017). This brought scrutiny to the performance of precast floors in high ductility RC structures; however, this is not the focus of this report. Another prominent challenge for engineers, highlighted by the Kaikoura earthquake was assessing the residual capacity and repairability of plastic hinges in low to moderately damaged RC structures. The lack of guidelines left engineers with little technical backing to assess the residual capacity of RC structures damaged in the earthquake. Following the earthquake an opportunity was provided to extract and test directly the residual capacity and the effectiveness of simple repair techniques on the performance of damaged RC beams. The tests investigate the use of epoxy crack injection, a relatively quick and simple to apply repair technique which, if effective, would allow moderately damaged RC structures to return to normal function in a short period of time.

The Structure

Due to confidentiality requirements the Building shall be referred to as “Building A” throughout this report. “Building A” was a RC perimeter moment frame structure. A precast hollowcore flooring system with RC topping was used. The structure was designed using a ductility of 6, the highest allowable design ductility by NZ Standards. Simply speaking, a ductility of 6 means that the building is expected to experience inelastic response for any earthquake exceeding 1/6 of the elastic design spectrum. Figure 1 clearly indicates inelastic response would be expected for this building during the Kaikoura earthquake. Visual inspection following the earthquake noted clear signs of damage indicating ductile frame behaviour. Cracking in the plastic hinge regions was noted as well as cracks in the floor diaphragm adjacent to the perimeter frame, indicating frame dilation.

Extraction of Specimens

Two internal and two external beam-column joints were removed from the fourth level above ground of the structure. Figure 2 shows the typical cross-section for the extracted beam.
elements from the building. All steel reinforcement is grade 300E steel with a specified concrete strength of 25MPa. The longitudinal reinforcement ratio, $\rho_t$, of all specimens was 0.6%. The beams span ~4.8m between the column faces in the frame and satisfy all detailing requirements of NZS 3101:2006-A3.

**Specimen Damage State**

Of the extracted units, the external beam-column joints were more extensively damaged in comparison to the internal joints, likely due to lower levels of restraint against elongation than their internal counterparts. The exterior joints consisted of diagonal cracking in the plastic hinge unit with a maximum crack width of 4-5mm. The specimen aligning with gridline A (Figure 3) also consisted of spalling of cover concrete in one corner, exposing a longitudinal reinforcing bar and stirrup. The internal units showed a lower damage state with only one significant crack either side of the joint region between 1-2mm in width, with the remaining cracks in the beam region being hairline. Figure 3 and Figure 4 showcase the damage survey from all four extracted specimens, carried out by engineers following the earthquake.

![Figure 2. Typical cross-section of extracted beam elements.](image)

![Figure 3. Crack mapping of level 5 beam column joints of perimeter moment frame.](image)
EXPERIMENTAL METHODOLOGY

To investigate the effectiveness of epoxy injection on moderate damage, one external unit (from Gridline A) was repaired, while the other (from Gridline Z) left as damaged. The lightly damaged internal units were used to investigate the residual capacity after further damaging cycles. Table 1 outlines the specimen ID’s and testing parameters for each specimen.

Table 1. Testing Matrix

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Repair (Y/N)</th>
<th>Loading Protocol</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-U (Gridline Z)</td>
<td>N</td>
<td>Figure 5a</td>
</tr>
<tr>
<td>E-R (Gridline A)</td>
<td>Y</td>
<td>Figure 5a</td>
</tr>
<tr>
<td>I-1.5-U (Gridline Y)</td>
<td>N</td>
<td>Figure 5b with 1.5% initial cycles</td>
</tr>
<tr>
<td>I-3.0-U (Gridline B)</td>
<td>N</td>
<td>Figure 5b with 3.0% initial cycles</td>
</tr>
</tbody>
</table>

Notes: The notation U and R in specimen IDs represents Unrepaired and Repaired, respectively. The notation E and I in specimen IDs represents External and Internal beams, respectively.

Loading Protocol:

Specimens were subjected to a static cyclic loading protocol to failure as shown in Figure 5a. A displacement-controlled protocol was applied to the beams using a hydraulic actuator. Two cycles were applied to the specimen at each drift level and cycles were applied until a 20%
drop in strength was observed in each specimen. Specimens E-U and E-R were used to investigate the residual capacity of the unrepaired and repaired beams following the Kaikoura Earthquake. After the good performance observed for these beams (see “Experimental Results” below), it was of interest to investigate the residual capacity of the beams when subjected to additional earthquake loading. The loading protocol shown in Figure 5b was selected for this purpose. The initial cycles were used to approximate the drift demand and cycles from a repeat of the Kaikoura Earthquake (specimen I-1.5-U) or that from an earthquake approximately twice as intense as the Kaikoura Earthquake (specimen I-3.0-U).

Experimental Setup:

All four specimens were tested in a cantilever beam configuration as shown in Figure 6. The beam column joints were rotated to be supported on the column face, with the beam protruding vertically from the laboratory strong floor. The beams were fixed to the floor using custom fabricated steel frames designed to sit on the face of the column and be post-tensioned onto the laboratory strong floor with a total force of approximately 3600kN. A 2000kN hydraulic actuator was attached to the beam face by clamping the hydraulic piston to the beam using steel plates post-tensioned to the beam face. An out of plane restraint frame was placed around the beam, with PTFE strips used to minimise friction between the specimen and restraints. The beams were tested with a shear span to depth ratio of 2.1.

Figure 6. Typical cantilever beam testing arrangement.

Testing of the external unit as cantilever specimens was readily done, with specimens E-U and E-R consisting of large 1m x 1m columns, providing a stable foundation. To test specimens, I-1.5-U and I-3.0-U in a cantilever configuration, one beam was removed from each specimen by a specialist contractor. Beam longitudinal reinforcement was then anchored at the back face of the joint by welding steel plates to the end of the cut joint reinforcement and reconstituting the concrete behind the welded plate. This allowed the specimens to be rotated onto their base and to be tested as cantilevers.
Repair Methodology:
A specialist contractor was commissioned to carry out repairs on specimen E-R via epoxy crack injection and reconstitution of cover concrete. Crack injection was carried out using Sikadur 52, a low viscosity epoxy resin. All cracks <0.2mm in width were sealed using epoxy resin with injection ports inserted at regular intervals along the cracks under repair. Epoxy resin was then pumped via the injection ports until resin was seen to pour out of outlet ports. Surface of injected cracks were then ground flush with existing concrete. Spalled concrete on specimen E-R was replaced using repair mortar (Sika Monotop 412 N). Loose concrete was first removed from the specimen using a small hammer drill. This was followed by repair of the beam surface with the Monotop mortar and smoothed to match the existing surface.

EXPERIMENTAL RESULTS
This section discusses the results of the beam tests described above. This includes a comparison of the repaired (E-R) and unrepaired (E-U) tests intended to quantify the effectiveness of the epoxy repair and a comparison of key structural characteristics (stiffness, strength and deformation capacity) for all specimens. Discussion of Specimen I-1.5-U and I-3.0-U focuses on the impact of prior loading cycles on beam performance.

Repaired vs Unrepaired - Damage Progression and Deformation Capacity
All four specimens showed consistent damage progression, with significant diagonal shear cracking in the plastic hinge region, similar to the in-situ earthquake damage observed in specimens E-U and E-R following the Kaikoura earthquake. Figure 7 illustrates the damage progression for specimens E-R, from pre-test condition to failure. Damage to E-R was observed to occur throughout the plastic hinge region, extending to the column face with no indication of a relocation of the plastic hinge region, as had been observed in some previous tests of repaired beams such as Celebi and Penzien (1973), Cuevas and Pampanin (2017) and French et al. (1990). New cracks formed at locations directly adjacent to epoxied cracks, which remained closed, suggesting that the epoxy bond to the concrete exceeded the tensile capacity of the concrete. Residual crack widths were also monitored throughout the tests with Specimen E-R showing significantly lower residual crack widths than E-U at corresponding drift levels.

All specimens saw significant spalling of cover concrete at 4% drift with signs of core crushing. Following cycles to 6% drift buckling of longitudinal reinforcement was also evident as seen in Figure 7 (d). All specimens saw a 20% or greater drop in strength on the second cycle to -6% drift coinciding with the unhooking of the stirrups in the two or three layers closest to the beam end. No fracture of longitudinal or transverse reinforcement was observed. The failure drift of 6% is significantly larger than what these components would be subjected to in a real structure during a damaging earthquake (limiting drift in NZS 1170.5 for ULS shaking is 2.5%). Furthermore, the failure drift is approximately twice the drift capacity of 2.8% calculated according to NZS 3101:2006-A3 (using $K_d = 19$).
Figure 7. Damage progression for specimen E-R (a) Prior to Test. Following cycles at (b) 1% Drift (c) 3% Drift and (d) 6% Drift. Damage states shown are indicative of all four specimens.

Stiffness

Figure 8 shows the force-displacement plot for the first ¼ cycle to yield of Specimen E-U and E-R. The secant stiffness of each specimen to yield was calculated based on the drift at 0.8Mn (Equation 1) where Mn represents the nominal yield strength of the member as calculated based on NZS 3101:2006. This metric is used as an approximation for the secant stiffness to yield of the members and has been used in prior experimental investigations by Marder et al. (2019). This point is shown on the plot in Figure 8 and corresponds to a lateral force of 536 kN.

\[
\text{Secant Stiffness to Yield} = \frac{F_{0.8Mn}}{\text{disp}_{0.8Mn} - \text{disp}_{\text{residual}}} \tag{1}
\]

Figure 8. Shear force-displacement relationship for specimen E-U and E-R during the initial cycle to yield.

The stiffness of the various specimens is summarised below in Table 2. The epoxy repair was able to increase the secant stiffness to yield of specimen E-R by 25.7% in comparison to its equivalent unrepaired specimen, E-U. Specimens I-1.5-U and I-3.0-U had higher stiffnesses
than observed in the specimens with higher damage levels prior to testing. The stiffness in these specimens was up to 55% higher than seen in specimen E-U.

Table 2. Secant stiffness to yield comparisons between repaired and unrepaired specimens.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>E-U</th>
<th>E-R</th>
<th>I-1.5-U</th>
<th>I-3.0-U</th>
<th>NZS 3101 C5 Assessment Guidelines</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Secant Stiffness to Yield (kN/mm)</td>
<td>35.68</td>
<td>44.84</td>
<td>55.43</td>
<td>47.75</td>
<td>181.5</td>
</tr>
</tbody>
</table>

*Note: Stiffness determined based on E.I_eff in accordance with NZS 3101:2006.

Strength

Figure 9 (a) and (b) shows the force-displacement relationship for specimens E-U and E-R and specimens I-1.5-U and I-3.0-U, respectively. It is evident from the plot that both the yield and ultimate strength of specimen E-R has increased relative to E-U, particularly in the positive direction. Table 3 summarise the strength values of the specimens for comparison. Both yield and ultimate strengths of specimen E-R were ~10% higher than the average of the three unrepaired specimens. The yield strength of the unrepaired specimen was on average 14% higher than the predicted overstrength of the specimens. Given the damage state of the specimens prior to repair and crack widths suggesting prior yielding of the longitudinal reinforcement, this strength increase is a clear sign of the impacts of strain hardening and strain aging of the reinforcement. It should be noted that despite the presence of these phenomenon, which act to reduce the strain capacity of reinforcement, fracture of reinforcement was not observed during testing and all specimens were pushed to a very large drift (6%) prior to failure (due to unhooking of transverse reinforcement), as discussed in the previous section on deformation capacity.
Figure 9. Force-displacement plot comparison of (a) Specimen E-U and E-R (b) Specimen I-1.5-U and I-3.0-U.

Table 3. Strength comparisons between repaired and unrepaired specimens.

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Yield Strength (kN)</th>
<th>Ultimate Strength (kN)</th>
<th>Predicted Yield Strength (kN)</th>
<th>Predicted Overstrength Yield (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-U</td>
<td>784</td>
<td>866</td>
<td>506.5</td>
<td>670</td>
</tr>
<tr>
<td>E-R</td>
<td>830</td>
<td>968</td>
<td>506.5</td>
<td>670</td>
</tr>
<tr>
<td>I-1.5-U</td>
<td>748</td>
<td>875</td>
<td>506.5</td>
<td>670</td>
</tr>
<tr>
<td>I-3.0-U</td>
<td>758</td>
<td>876</td>
<td>506.5</td>
<td>670</td>
</tr>
</tbody>
</table>

Notes: *Predicted overstrength yield calculated based on NZS 3101:2006 and a longitudinal reinforcement yield of 1.35f_y and concrete strength of [f'c+15] MPa.

CONCLUSIONS

Four beam-column joints were extracted from a damaged RC building in Wellington following the 2016 Kaikoura Earthquake. The specimens displayed varying damage states ranging from low to moderate. The specimens were tested at the University of Auckland to investigate the residual capacity and impact of epoxy repair on the performance of the beam elements. The key results of the experimental testing are outlined below.

Stiffness:
Prior earthquake damage causes a reduction in the stiffness of reinforced concrete elements. Repair via epoxy injection can recover some stiffness, but not to its original state.

The epoxy repaired beam specimen (E-R) was 25.7% stiffer than the equivalent unrepaired specimen (E-U), but only ~20% of the stiffness assumed in structural design using NZS 3101.

The beam elements with lower damage states (I-1.5-U and I-3.0-U) exhibited stiffness up to 55% higher than specimen E-U.

Strength:

For ductile beams designed in accordance to NZS 3101 and controlled by flexure, prior earthquake damage does not reduce the observed flexural strength.

Subjecting the damaged specimens to multiple large cycles, such as in specimen I-1.5-U and I-3.0-U, also did not impact the flexural strength of the beam elements, suggesting multiple prior earthquakes would not reduce the strength of the beams.

Prior strain hardening and ageing of the reinforcement leads to a larger strength than anticipated in design, which can exceed the design overstrength. This higher strength must be accounted for in the post-earthquake assessment of the building, particularly as it may influence the hierarchy of strength in the building.

Drift Capacity:

The drift capacity of all specimens was 6%, regardless of the presence of epoxy repair. This is more than two times the limiting drift specified in NZS 1170.5 (2.5%) and the rotation capacity calculated according to NZS 3101 (2.8%).

Additional initial cycles at 1.5% subjected to the beam as in specimen I-1.5-U (approximately simulating the equivalent damage subjected to the structure in the Kaikoura Earthquake) did not reduce the drift capacity of the beam.

Additional initial cycles at 3% subjected to the beam as in specimen I-3.0-U (simulating a much larger event than the Kaikoura Earthquake) also did not result in any reduction of the deformation capacity of the beam.

Strain hardening and strain ageing of steel due to prior loading did not result in the fracture of any reinforcement or result in the reduction of deformation capacity in any of the tested specimens.

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REFERENCES


