SUMMARY

In the aftermath of the Canterbury earthquakes, the reparability of ductile reinforced concrete moment resisting frames has been a topic of much discussion. This paper presents results from laboratory testing of reinforced concrete beam-column subassemblies extracted from Clarendon Tower, a 19 storey building that was damaged by the Canterbury earthquake sequence. Subassemblies were repaired and/or retrofitted before being subjected to quasi-static cyclic loading. By comparison of the results with results from previous laboratory testing of similar specimens it is shown that the performance of the repaired specimens equalled or exceeded the performance that would have been expected from undamaged specimens. No effects related to low cycle fatigue of reinforcement were observed during the testing.

INTRODUCTION

In the aftermath of the Canterbury earthquake sequence, and particularly the 22 February 2011 Christchurch earthquake, the ability of repaired ductile structures to perform satisfactorily during future earthquakes has been questioned repeatedly. These questions arise from concerns that yielded reinforcing bars may be affected by low cycle fatigue and thus suffer premature failure during future cyclic yielding.

To date, testing related to low cycle fatigue in structures damaged by the Canterbury earthquakes has been limited to testing of individual reinforcing bars using unproven indirect techniques. Notwithstanding accuracy, such tests are of questionable value, owing to the performance of structures rarely being limited by the behaviour of individual reinforcing bars. This paper presents results from testing of repaired large scale specimens extracted from a damaged ductile structure, and thus presents a more holistic view of the reparability of damaged ductile structures.

BACKGROUND

Clarendon Tower was a nineteen storey reinforced concrete building completed circa 1988 and located at the west side of the Christchurch CBD as shown in Figure 1. The structural system for the building consisted of perimeter moment resisting frames, two secondary (gravity) frames running north-south parallel to the longer axis of the building, and double tee floors spanning east-west parallel to the shorter axis of the building. The layout of the structure is shown in Figure 1. The structure was designed to behave in a ductile manner, with design based on the capacity design philosophy as encompassed by then current New Zealand Standards (NZS 3101 1982; NZS 4203 1984).

The design of the moment resisting frames differed between the long and the short axes of the building. As shown in Figure 1 the columns along the north and south sides of the building were unusually closely spaced, while those on the east and west sides were conventionally spaced. Owing to the close column spacing, the north and south side frames used an unusual diagonal bar detail that was intended to position the plastic hinge in the
middle portion of the beam rather than adjacent to the column faces (see Figure 2 and Figure 10 later). Similar details have been described previously in the literature (Bull 1999; Restrepo et al. 1995). The detailing of the east and west frames was that of a conventional moment resisting frame intended to develop plastic hinges at the column faces.

![Figure 1: Clarendon Tower viewed from the south west (left, courtesy of Google) and arrangement of typical floor (right)](image)

**Effect of earthquakes on Clarendon Tower**

Clarendon Tower was moderately damaged by the Canterbury earthquake sequence. Based on a combination of observation and analysis, the key aspects of the damage can be summarised as:

- Light damage to the east and west moment resisting frames, indicative of ductility demands of approximately \( \mu = 2.0 \) or less;
- Moderate damage to the north and south moment resisting frames, with damage being most significant at the mid levels and at the north end. Maximum ductility demands were of the order of \( \mu = 4.0 \);
- Localised tearing of diaphragms were these connected to the frames at the north and south ends of the structure, most likely due to frame elongation;
- Partial unseating of some double tee beams due to frame elongation;
- Collapse of four stair flights due to displacement incompatibility.

Due to the ductile behaviour of the moment resisting frames it was alleged that the performance of Clarendon Tower during future earthquakes would be reduced by low cycle fatigue. The testing discussed in this paper was undertaken to show that this was not the case.

**TEST PROGRAM**

From the outset, the aim of the testing described was to investigate the effect of past yielding on future performance at a member level. This aim was selected as it was felt to be more pertinent to the performance of complete structures than consideration of individual reinforcing bars. Specifically, the intent of the testing undertaken was to investigate the behaviour of the plastic hinges. In order to adequately test the performance of the plastic hinges, the testing needed to apply forces and displacements to the hinges that were equivalent to those that would be experienced by the hinges when they were in the building during an earthquake. This was achieved by extracting beam-column sub-assemblies for testing from the damaged building.
Significance of testing

The testing described in this paper presented numerous challenges, but overcoming these was considered worthwhile due not only to its relevance to Clarendon Tower, but also several other reasons that made the testing noteworthy. These included:

- The test specimens are amongst the largest beam-column joint sub-assemblies that have ever been tested in New Zealand;
- The test specimens are amongst the largest components extracted from actual buildings that have been tested worldwide;
- The test specimens are amongst the only ones of this type to have been extracted from an earthquake damaged building worldwide (Brook 2013).

Description of specimens

Due to the differing plastic hinge locations, the type of test specimen required differed between the north and south frames and the east and west frames. Both specimen types represented a single bay, single storey unit of the respective frames. Due to transport requirements the length of columns cut from the building was limited to 2400 mm. Steel extensions were attached during testing so that the total column length was equal to the storey height of Clarendon Tower.

- The specimens from the north and south frame consisted of two columns with a beam spanning between them as shown in Figure 2. Due to their shape, these specimens are referred as H specimens;
- The specimens from the east and west frame consisted of a single column with a beam spanning to each side of the column as shown in Figure 2. Due to their shape, these specimens are referred to as cruciform specimens.

A total of three specimens were tested following extraction from Clarendon Tower. These comprised one H-specimen from each of the north and south frames and one cruciform specimen from the west frame, all extracted from the most damaged levels. Specimen designations, locations within the structure, damage level, and repair summaries for each specimen are shown in Table 1.

Figure 2: H-specimen (left) and cruciform specimen (right)
Table 1: Summary of tested specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type</th>
<th>Location</th>
<th>Column(s)</th>
<th>Repair</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-UG-1</td>
<td>H</td>
<td>L8 north</td>
<td>H2, J2</td>
<td>Retrofit and crack injection</td>
</tr>
<tr>
<td>H-AB-2</td>
<td>H</td>
<td>L8 south</td>
<td>G19, H19</td>
<td>Crack injection</td>
</tr>
<tr>
<td>C-AB-3</td>
<td>Cruciform</td>
<td>L7 west</td>
<td>B5</td>
<td>None</td>
</tr>
</tbody>
</table>

Test specimen repair procedures

One aim of the testing was to verify whether performance of Clarendon Tower after repair would have equalled or exceeded its performance before the earthquakes. Thus where repairs were proposed that would restore the capacity of the moment resisting frames, these repairs were applied to the test specimens prior to testing.

Specimen H-UG-1 was taken from the most damaged region of the north frame where it was anticipated that both repair and retrofit of the beams would be required. Thus the specimen was repaired using a procedure based on that described by Restrepo for a similar detail (Restrepo et al. 1995) and consisting of the steps listed below:

1. Concrete surrounding the diagonal bar detail was broken out using hydraulic demolition techniques to avoid damage to reinforcing steel (see Figure 3).

2. Additional reinforcement transverse to the axis of the beam was placed at the inside of each bend of the diagonal reinforcement to improve bearing strength at these locations.

3. Steel straps supporting bend of the diagonal reinforcement were altered so that bearing stresses were more evenly distributed between inner and outer bars.

4. Ties were added around the vertical legs of the hooks formed where longitudinal reinforcement terminates at commencement of diagonal detail.

5. High strength repair concrete was placed in the broken out region to complete the repair.

Figure 3: Repairs to specimen H-UG-1
The H specimen H-AB-2 was taken from the less damaged south frame where damage to the beams consisted of only minor cracking. Repair was therefore achieved by epoxy injection of cracks only.

The cruciform specimen C-AB-3 was not repaired, as the damage to the beams was not anticipated to affect their structural performance. Repair would only have been required for reasons of cosmetics and durability.

**Test setup and load history**

The intent of testing the beam-column specimens was to replicate as closely as possible during testing the forces and deformations that would occur in the plastic hinges if they were within the complete structure during an earthquake. This required development of a test setup that could cyclically apply an appropriate pattern of forces to the specimens. This development was complicated by the large size and strength of the test specimens, and also by the need to accommodate two different types of specimen with different loading and restraint requirements.

The size and strength of the specimens significantly exceeded the capacity of readily available test facilities. Thus a custom self balancing test frame had to be designed. A reconfigurable frame was developed that consisted primarily of steel universal column sections. Isometric views of this frame configured for testing of H-specimens and cruciform specimens are shown in Figure 4. The assembled test frame configured for a cruciform specimen is shown in Figure 9 towards the end of this paper.

Simulated earthquake forces were applied to the test specimens by double acting hydraulic actuators. Different arrangements of actuators and supports were required for application of forces to the H specimens and the cruciform specimen. These are described below and shown in Figure 5.

- For the H specimens, two active actuators applied forces to displace the bottom ends of the columns. A third passive actuator was placed at the top of one column in conjunction with a double pin arrangement that allowed for the possible occurrence of beam elongation during testing.
- For the cruciform specimens, a single actuator applied force to the bottom of the column, with the beam ends pinned to prevent vertical movement but free to translate horizontally and to rotate. Due to limitations on the forces that could be reliably applied by the test setup the left and right beams were tested independently. This did not affect the ability of the testing to accurately assess the performance of the plastic hinges.
Connection of the test specimens into the setups was achieved by heavy steel shoes placed at the column ends. These shoes were not physically attached to the columns; rather a grouted steel ‘skirt’ surrounded the columns to provide shear transfer, while three post-tensioned Macalloy steel rods on each side of the column were used to clamp the shoes and provide transfer of moment and axial tension. The total force applied by these rods was set at approximately 2300 kN.

The loading history used for testing took the form described by Park (1989), i.e. applying pairs of cycles to increasing ductility increments. Adoption of this load history aided comparison of the testing with past tests that were used as the performance benchmark. In particular the load history used was similar to that used by Restrepo (1995) during testing of a specimen (‘Unit 4’) of very similar design to, and used as the benchmark for, Specimens H-UG-1 and H-AB-2. Application of a load history that replicates Restrepo’s as closely as possible aids comparisons between the tests reported here and Restrepo’s work.

TEST RESULTS

Testing of the specimens was undertaken by personnel from the UOA led by Professor Jason Ingham and Dr Rick Henry. Testing took place between late April 2013 and early June 2013, with each individual test taking approximately two and a half days to complete. Testing was continued until the displacement limits of the test frame were reached. Due to the limited stroke of the actuators and the relative flexibility of the test frame this limits was typically approximately 2.5% drift, which exceeded the drift expected to occur in Clarendon Tower during a design level earthquake.

Results of each test are discussed in the following sub-sections, with the principal presentation of results taking the form of force-displacement hysteresis graphs. To aid comparison between different test results data is presented using non-dimensional measures. Interstorey shear was normalised by dividing the calculated value by the expected strength of the specimen, which is referred to as the relative storey shear. Displacement was normalised by dividing the measured lateral movement by the storey height.

H-specimen results

The performance of the two H-units tested was expected to differ owing to Specimen H-UG-1 having been both repaired and retrofitted while Specimen H-AB-2 was repaired only. Thus the performance of H-UG-1 was anticipated to exceed than the performance of the building
as it was built and prior to the earthquakes, while the performance of H-AB-2 was anticipated to be equal to the performance of the building as it was built and prior to the earthquakes.

Force-displacement graphs showing the response of specimen H-UG-1 and H-AB-2 are shown in Figure 6 as blue lines. Testing of both units was concluded after completion of two complete cycles to ductility $\mu = 5.0$. The force displacement response of Restrepo’s Unit 4 is overlaid on each graph as a red line, with the data scaled to fit the interstorey drift and relative storey shear force axes. Restrepo’s data is referred to in comparisons that are elaborated on later in this paper.

![Figure 6: Force displacement responses of specimens H-UG-1 (left) and H-AB-2 (right) overlaid with response of Restrepo Unit 4](image)

Considering Figure 6, for both specimens the yield displacement was equal to approximately 0.4% drift and the maximum storey shear force was equal to approximately 150% of the expected strength. The behaviour of the two specimens differed in the later load cycles, with specimen H-UG-1 maintaining strength longer and having less pinched hysteretic form. In detail:

- For specimen H-UG-1, the strength of the unit decreased by approximately 10% during the negative part of the first cycle of loading to ductility $\mu = -5.0$ and by approximately 20% for both loading direction during the second cycle to ductility $\mu = -5.0$. This strength decrease was the result of sliding occurring at the location of a steel plate included in the unit as part of the connection to the secondary (gravity) frame in the original structure.
- For specimen H-AB-2, the strength of the unit decreased by approximately 15% during the second cycle of loading to ductility $\mu = 3.0$, remained similar during the first cycle to ductility $\mu = 5.0$, and then decreased further to approximately 20% below the peak strength during the second cycle to ductility $\mu = 5.0$. This strength decrease was the result of spalling of concrete at the edges of the diagonal bar detail.

No distress of reinforcing bars was observed to have taken place during testing of either specimen. Images of the two specimens at the conclusion of testing are shown in Figure 7.

**Cruciform specimen results**

Specimen C-AB-3 was not repaired prior to testing as the repairs proposed for the west frame were minor and not expected to affect the structural performance. It was discovered during testing that one of the top reinforcing bars in specimen C-AB-3 had been cut during the demolition process on either side of and immediately adjacent to the column. This reduced the strength of the test specimen, but does not prevent useful valid conclusions being drawn from the testing. The strength reduction resulting from this cut bar was accounted for when calculating the relative storey shear force resisted by the specimen.
The beams on either side of the column of specimen C-AB-3 were tested independently, and results are presented as such. The two beams are referred to by their orientation when in the building as C-AB-3 (north) and C-AB-3 (south). Force-displacement graphs showing the response of the north and south beams of specimen C-AB-3 are shown in Figure 8. Note that the expected strength for negative loading (beam pulled downwards) was reduced by the cut bar and is thus lower than the expected strength for positive loading (beam pushed upwards).

A number of observations can be drawn from the force-displacement graphs:

- The yield displacement of the specimen was equal to approximately 0.65% drift.
- The maximum storey shear force resisted by the unit was equal to approximately 110% of the expected strength for positive loading and 130% of the expected strength for negative loading.
- For negative loading the force resisted by the beam continued to increase until the first cycle to ductility $\mu = 6.0$. For positive loading the force resisted by the beam began to decrease when the ductility exceeded $\mu = 4.0$ for the north beam and $\mu = 5.0$ for the south beam. For the north beam the strength decrease was approximately 20% during the second cycle to ductility $\mu = 4.0$, and approximately 50% during the cycle to ductility $\mu = 6.0$. For the south beam the strength decrease was approximately 15% during the first cycle to ductility $\mu = 6.0$, and approximately 50% during the second cycle to ductility $\mu = 6.0$.

Strength degradation was attributed to a combination of crushing of weaker topping concrete at the top of the beam, and buckling of longitudinal bars, the restraint of which was reduced.
by stirrup damage that occurred during demolition. No distress of reinforcing bars attributable to low cycle fatigue was observed to have taken place during testing. The condition of the specimen at the conclusion of testing is shown in Figure 9.

Figure 9: Specimen C-AB-3 at the conclusion of testing (south beam at right)

COMPARISON TO EXPECTED PRE-EARTHQUAKE PERFORMANCE

The key aim of the testing described was to determine how the performance of the repaired specimens compared to the performance that would have been expected of new as-built specimens. These comparisons are discussed in this section.

In order to compare the performance of H-UG-1 and H-AB-2 to the performance of the as-built building before the earthquakes, results were compared with those obtained by Restrepo (1995) from testing of his Unit 4. As shown in Figure 10 the design of Unit 4 is practically identical to the design of the beams in the north and south frames of Clarendon Tower; indeed Clarendon Tower was reputedly the prototype for Unit 4. Unit 4 was built in a laboratory and then tested in its undamaged state within three months of construction; thus it is reasonable to proceed on the basis that if beams from Clarendon Tower had been tested in their ‘as-new’ condition, their performance would have been the same as that of Restrepo’s Unit 4.

Referring to Figure 7, a number of observations can be made by comparison of the force-displacement responses of specimens H-UG-1 and H-AB-2 with that of Restrepo’s Unit 4:

- The yield drift of specimen H-UG-1, H-AB-2, and Restrepo’s Unit 4 are all approximately 0.4%.
- The relative strength of Specimen H-UG-1 and H-AB-2 was greater than that of Restrepo’s Unit 4. This is likely due to strain ageing and strain hardening of the reinforcement, and possibly variation of the reinforcement properties from the average values used to calculate the expected strength.
- Specimen H-UG-1 was able to sustain larger displacements than Restrepo’s Unit 4 without losing strength, and displayed less hysteretic pinching.
- The strength degradation and hysteretic pinching observed during testing of Specimen H-AB-2 were similar to that observed during testing of Restrepo’s Unit 4.
Based on the observations above, it was concluded that the performance of specimen H-UG-1 exceeded the performance that would have been observed if the specimen was tested prior to the earthquakes, while the performance of specimen H-AB-2 equalled the performance that would have been observed if the specimen was tested prior to the earthquakes.

Figure 10: Comparison of beam detailing used in Clarendon Tower and by Restrepo (Holmes Wood Poole & Johnstone Ltd 1987; Restrepo et al. 1995)

Rather than being compared to a single benchmark test, the performance of C-AB-3 in relation to that of the as-built building before the earthquakes has been assessed by comparing key performance measures to the performance that would generally be expected of a newly built moment resisting frame as indicated by the results of approximately 100 other beam-column joint tests undertaken internationally over the last forty years and collated and summarised elsewhere (Brooke 2011). Considering the stiffness, strength, and displacement capacity of specimen C-AB-3:

- The yield drift of approximately 0.65% observed during testing of specimen C-AB-3 is comparable to the value of 0.5% predicted using a simple method proposed by Priestley (Priestley 1998), and within the scatter observed when Priestley’s method is compared to a wide range of experimental results (Brooke 2011).
- The strength of specimen C-AB-3 during testing first equalled its expected strength, and then exceeded it as larger displacements were applied. This performance is typical of conventionally detailed moment resisting frames (Brooke 2011), and thus that the strength of west frame was unaffected by the earthquakes.
- The displacement capacity of specimen C-AB-3 was in excess of 3.0% drift. This is greater than the maximum drift permitted by New Zealand standards (NZS 1170.5 2004), and typical of the displacement capacity of conventionally detailed moment resisting frames (Brooke 2011).

Based on the observations above, it was concluded that the performance of specimen C-AB-3 equalled the performance that would have been observed if the specimen was tested prior to the earthquakes.

CONCLUSIONS

Three specimens extracted during the demolition of Clarendon Tower were tested using methods commonly used in New Zealand and internationally for testing of moment resisting frame buildings.
In all cases the performance of the repaired test specimens equalled or exceeded the performance that would have been observed if the specimens had never been affected by an earthquake.

No effects related to low cycle fatigue of reinforcing steel were observed during testing.

Thus, repair of Clarendon Tower was structurally viable

ACKNOWLEDGEMENTS

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REFERENCES


