POST-EARTHQUAKE RESIDUAL CAPACITY OF DAMAGED REINFORCED CONCRETE BUILDINGS

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Abstract

Many buildings experience intermediate levels of damage during seismic events. In these situations, a detailed assessment of the building in its damaged state is required for owners and other stakeholders to make informed decisions about the building's future. The aftermath of the Canterbury earthquakes of 2010-2011 brought into sharp attention the lack of available resources for engineers conducting detailed assessments of damaged buildings. In this paper, a proposed framework for the post-earthquake assessment of damaged reinforced concrete buildings is presented. Within the bounds of this framework, the need for models estimating the residual strength, stiffness, and deformability of damaged reinforced concrete components is also discussed. Preliminary results are presented from an experimental program evaluating the residual capacity of damaged and repaired reinforced concrete beams. This work is part of an effort funded by the New Zealand Government to develop guidelines for assessing the post-earthquake residual capacity of damaged structures.

Introduction

The majority of modern reinforced concrete (RC) buildings are designed to withstand seismic loads by trading off strength for controlled damage, through formation of plastic hinges in pre-determined locations. This design philosophy permits RC buildings to dissipate a significant amount of energy and achieve large ductility demands while ensuring life safety of building occupants, but does not focus on post-earthquake reparability. The Canterbury earthquakes of 2010-2011 resulted in the demolition of approximately 60% of multi-storey RC buildings in Christchurch, New Zealand (Marquis et al., 2015). In some cases, the buildings were so heavily damaged that the need for demolition was immediately apparent, but in other cases there was only a moderate, and potentially reparable, amount of damage. However, engineers tasked with conducting detailed assessments on these moderately damaged buildings were left with limited technical resources on which to base their evaluation.

The work discussed in this paper aims to address this area of research need at both the system-level and component-level. The system-level work involves a proposed framework for the detailed assessment of earthquake-damaged RC buildings. The component-level research aims to comprehensively evaluate and quantify the post-earthquake residual capacity of damaged and repaired RC plastic hinges, including an experimental investigation on large-scale RC beams (presented in section 3). It is noted that 'residual capacity' here refers to the ability of an earthquake-damaged member to resist future seismic loading, including any changes in stiffness, strength, energy dissipation, or fatigue life. Reduced strain capacity of the longitudinal reinforcing bars due to low-cycle fatigue and strain ageing effects was a particular area of concern following the Canterbury earthquakes, due to the formation of a small number of large cracks in the beam plastic hinge zones resulting in large inelastic strains in the reinforcement crossing those cracks (Mander & Rodgers, 2013). Reports on damage from Christchurch indicate the potential importance of strain rate effects and reinforcement ratios on the crack distribution in plastic hinge zones. Previous prism testing (Huffadine et al., 2015) has shown that static (slow) loading produces an initial pattern of widely spaced cracks, with secondary cracks forming between the initial cracks, while seismic dynamic loading produces only the initial pattern of widely spaced cracks.

This study is limited to components with ductile, flexure-controlled failure mechanisms (i.e. plastic hinging). Members exhibiting signs of non-ductile failure modes would be more likely to be required to undergo retrofitting or replacement regardless of its assessed residual capacity, and hence are considered a lower priority in the development of residual capacity assessment guidelines. Many older buildings that were designed prior to the codification of capacity design may have entirely shear-controlled failure mechanisms, and would therefore be outside the scope of this research.

A variety of methods are available for retrofitting or structural enhancement of damaged RC components (e.g. concrete jacketing, FRP wrapping, etc.), but comparatively fewer methods are available for simple repair or restoration of original capacity (e.g. epoxy injection) (Applied Technology Council, 1999). This study focuses on the repair method of epoxy injection as its application is relatively consistent regardless of component type or detailing, and its effects can therefore be quantified for general use. Furthermore, epoxy injection with a high strength epoxy tends to force cracks to form elsewhere upon reloading and the epoxy-filled cracks do not re-open (Celebi & Penzien, 1973; French et al., 1990). This has implications for mitigation of any reduced deformation capacity in plastically-strained reinforcing steel, as the highest strain demands will be concentrated at the new crack locations.

System-Level Framework for Post-Earthquake Assessment

Two existing documents that deal with detailed post-earthquake evaluation of damaged RC buildings have been published internationally (Applied Technology Council 1998; Nakano et al., 2004). Note that detailed evaluation indicates assessment for long-term use of the building (i.e. for a future ultimate limit state (design-level) event) rather than short-term aftershock risk. The general methodology employed in both of these guidelines involves establishing reduction factors that are used to modify the capacity of damaged components. The reduction factors are based on a combination of the component type and detailing, and the severity of the observable damage. Once the reduction factors have been applied to all damaged components, a global analysis can be conducted to quantify any reduction in the ability of the building to meet its performance objectives. Potential limitations of these methodologies include the subjectivity of the building inspection, the fact that reduction factors and damage indicators are calibrated off of static cyclic experimental tests or are judgment-based, the inherent variability of visual damage, and the lack of consideration of number of loading cycles or loading protocol.

Due to these limitations, a modified assessment methodology is proposed in this study. A flowchart showing this proposed framework is shown in Figure 1. The principal difference is the focus on estimating the peak demands incurred during the damaging earthquake, and using these peak demands as a basis for assessing residual capacity. The visual damage is used to refine the estimates of peak demands rather than as the basis for the entire assessment. At the global level, the overall distribution of damage in the structure can be compared against that predicted using an analytical model. Differences should be investigated and attempted to be reconciled. At the local level, detailed information on the in-situ damage can be used to better estimate the peak demands that a particular component experienced.

Once the best estimate of the demands on an individual component in the damaging earthquake have been established, models are required that can predict the residual stiffness, strength, and deformability of that component as a function of its peak demand. If it is determined that repair is needed to restore the component to a satisfactory performance level, models that can predict the post-repair behavior are also required. Modern hysteretic models for RC components that incorporate stiffness and strength degradation require complete information on the prior component response in order to define the level of degradation, such as the energy dissipation parameters used in the popular Ibarra model (Ibarra et al., 2005). In a post-earthquake situation, only incomplete estimates of prior response are available, and residual capacity models must reflect this.

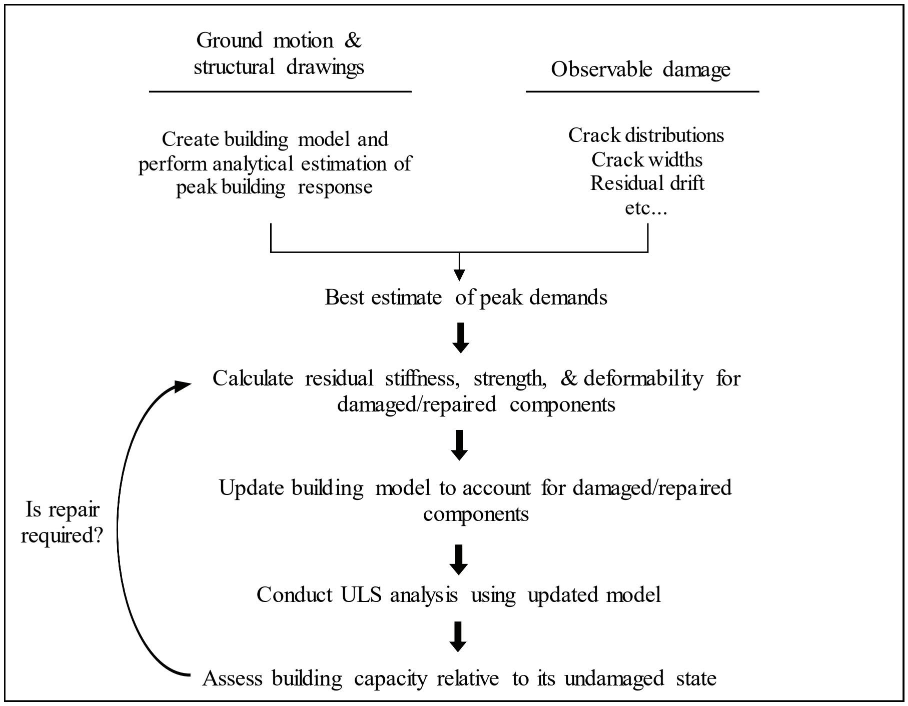


Figure 1. Flowchart of proposed post-earthquake assessment methodology.

Elwood et al. (2016) provides a detailed explanation of each step in the post-earthquake assessment methodology shown in Figure 1 and identifies research needs to be able to implement the methodology in engineering guidelines. A high-level summary of the identified research needs is provided below:

1. Procedure to determine locations and length of *observed* plastic hinges to be used in building analysis and to assess reinforcement strain demands based on axial loads and measured crack widths.
2. Determine a maximum crack width beyond which detailed assessment of bar strain is considered necessary.
3. Establish a relationship between observed damage (e.g. crack widths and distribution) and peak demands experienced by components.
4. Determine what characteristics of the damaging ground motion (e.g. pulse vs long duration) will require consideration of low-cycle fatigue (LCF).
5. Establish a methodology for assessing the residual capacity in terms of the number of cycles to LCF failure.
6. Establish models for stiffness and strength degradation based on peak (and/or residual) deformation demands. Determine how much stiffness degradation can be recovered with epoxy injection of cracks.
7. Establish when strain ageing needs to be considered in the residual capacity of damaged and repaired components.
8. Establish criteria based on the overall building capacity for determining when a repair or demolition should be recommended.

Several of these research needs are currently being addressed by ongoing research programmes at the Universities of Auckland and Canterbury, funded by the Natural Hazards Research Platform, MBIE Building Performance Branch, and QuakeCoRE. Research need #5 is addressed by Cuevas Ramirez &

Pampanin (2016). The experimental program described in the remainder of this paper aims to address research needs 1, 3, 6, and 7.

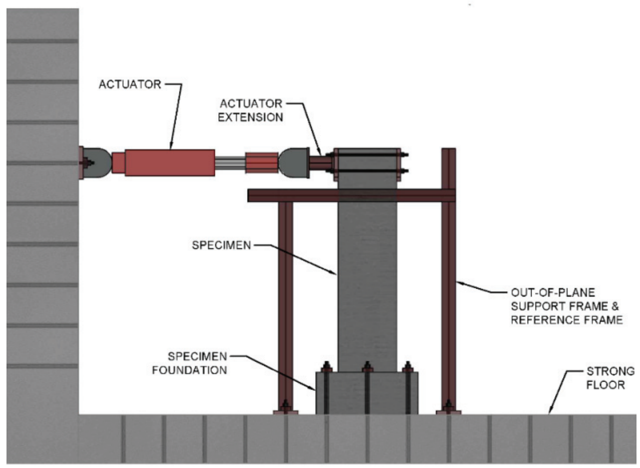


Figure 2. Rendering of test set-up

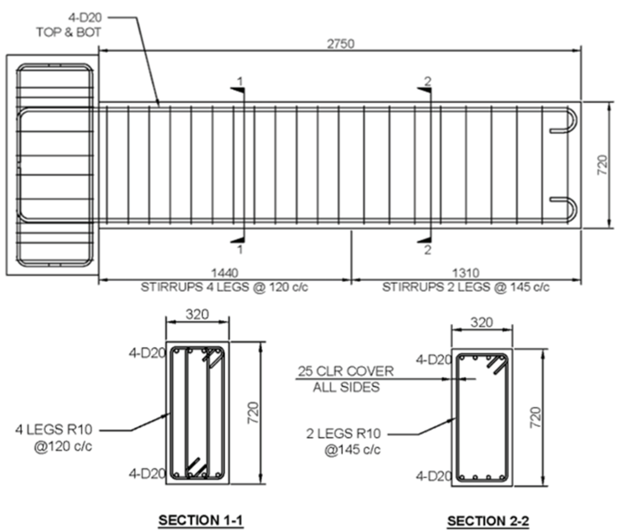


Figure 3. Specimen detailing

Experimental Program

Overview. An experimental program investigating the behavior of new, damaged, and repaired RC beams is underway as of the time of writing. A total of 14 identical large-scale cantilever beams are being tested, with the test set-up as shown in Figure 2.**Error! Reference source not found.** Given that the purpose of the research is to investigate the plastic hinging typical of capacity-designed buildings, the beam-column joint and column behavior is not included in the experiment, as they would be designed to remain elastic. Investigation of damaged buildings in Christchurch has shown the effectiveness of this design philosophy, with those elements showing at most only a few hairline cracks. Instead, a foundation designed to remain elastic and cast monolithically with the beam is used to anchor the specimens to the strong floor.

The specimen design is a 4/5 scale beam meeting all requirements for maximum allowable ductility in modern RC code seismic provisions (e.g. special moment frame beam in ACI 318-11 or ductile beam in

NZS 3101:2006). The beams were chosen to be relatively lightly reinforced (approximately 125% of minimum required) in order to accentuate potential crack widths and strain in the reinforcement. The relationship between reinforcing ratio and crack widths has been investigated in (Lu et al., 2015). Full specimen details are shown in Figure 3.**Error! Reference source not found.**

Objective and test matrix. The overarching objective of the experimental program is improving the state of knowledge regarding the residual capacity of damaged and epoxy-injection-repaired RC plastic hinges. The test program was designed to provide valuable data on the relationships between various loading inputs (dynamic vs. static loading rates; cyclic vs. monotonic vs. earthquake-type displacement histories), plastic hinge behavior (failure mechanism; drift at failure; spread of plasticity; strain in reinforcement), and observable damage (residual crack widths; crack distribution; drift at onset of spalling; etc.). The test program allows for these parameters to be compared at various levels of drift demand. The complete test matrix is shown in Table 1.**Error! Reference source not found.**

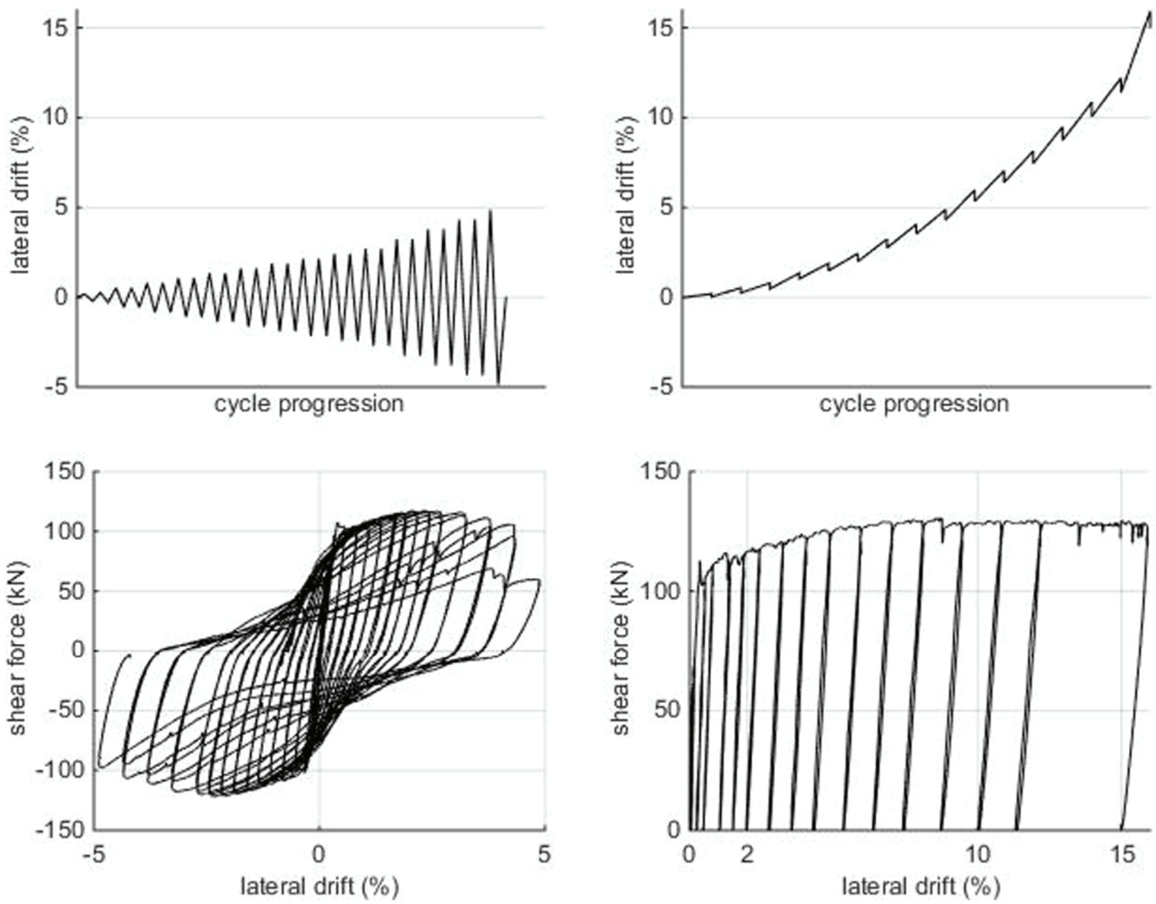
Table 1. Test Matrix

Specimen name[1]	*Initial damaging loading type*	*Failure loading type*	*Is specimen repaired after initial damage?*
Cyc	-	Static cyclic	No
Mono	-	Static monotonic	No
Cyc-ER	-	Static cyclic	No
Mono-ER	-	Static monotonic	No
LD-1.5	Dynamic long duration displacement history to ~1.5% drift	Static cyclic (cycles above 1.5% drift only)	No
LD-1.5-R	Dynamic long duration displacement history to ~1.5% drift	Static cyclic (cycles above 1.5% drift only)	Yes
LD-2.5	Dynamic long duration displacement history to ~2.5% drift	Static cyclic (cycles above 2.5% drift only)	No
LD-2.5-R	Dynamic long duration displacement history to ~2.5% drift	Static cyclic (cycles above 2.5% drift only)	Yes
P-1.5	Dynamic pulse-type displacement history to ~1.5% drift	Static cyclic (cycles above 1.5% drift only)	No
P-1.5-R	Dynamic pulse-type displacement history to ~1.5% drift	Static cyclic (cycles above 1.5% drift only)	Yes
P-2.5	Dynamic pulse-type displacement history to ~2.5% drift	Static cyclic (cycles above 2.5% drift only)	No
P-2.5-R	Dynamic pulse-type displacement history to ~2.5% drift	Static cyclic (cycles above 2.5% drift only)	Yes
LD-2.5-SA*	Dynamic long duration displacement history to ~2.5% drift	Static cyclic (cycles above 2.5% drift only)	No
LD-2.5-SAR*	Dynamic long duration displacement history to ~2.5% drift	Static cyclic (cycles above 2.5% drift only)	Yes

Note [1]: For specimens with initial damaging loadings, naming convention follows X-#, where X refers to the characteristics of the ground motion of initial damaging loading (long duration (LD) or pulse-type (P)) and # refers to the maximum drift applied in the initial damaging loading. Suffix -R indicates that the specimen is repaired by epoxy injection after the initial loading. Suffix -SA indicates specimen will be "strain aged" for a period of 3 months following the initial damaging loading.

Test Results. As of the date of writing this paper, specimens Cyc and Mono have been tested. Loading protocols and hysteresis plots for the two specimens are shown in Figure 4. The plots illustrate the pronounced effect of loading protocol on drift at failure, here defined as the drift at which lateral load capacity drops below 80% of its peak value. It can be seen that the cyclic specimen lost lateral load-carrying capacity at the cycle to 4.88% drift, while the monotonic specimen maintained its lateral strength above 15% drift, at which point the test had to be stopped due to the actuator stroke limit being reached. Note that during the monotonic test the lateral load was dropped to zero after reaching specific peak drift demands to allow for residual damage states to be observed for comparison with Cyc after similar peak drift demands.

The recorded damage data during the tests includes residual crack widths, which were measured for all cracks over 0.2mm wide after each loading increment. As seen in Figure 5, both the maximum and total crack widths exhibit a clear correlation with the maximum applied drift, although the ratio of total to maximum crack width increases with increasing drift due to formation of additional primary cracks. Note that total crack width is the sum of all crack widths over 0.2mm on the tension side of the beam. The maximum and residual crack widths in the cyclic test were larger than in the monotonic test, which can be explained by the increased beam elongation in the cyclic test.



(a) (b)

Figure 4. Loading protocol and shear force vs lateral drift for specimen Cyc (a) and Mono (b).

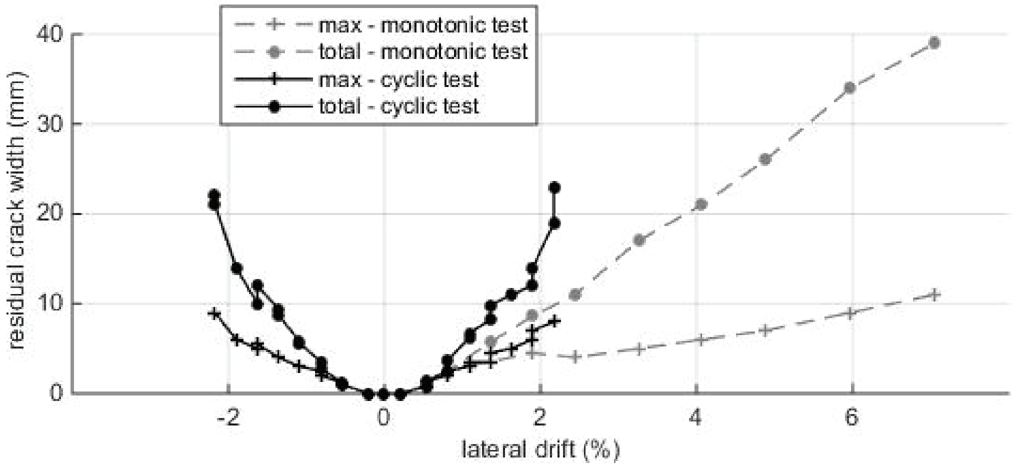


Figure 5. Residual crack widths (maximum and total) for specimen Cyc (black) and Mono (grey).

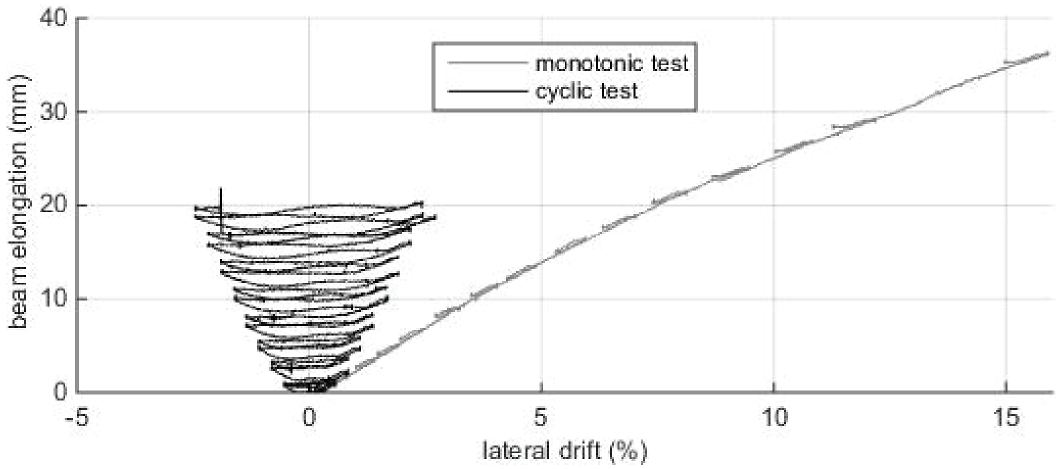


Figure 6. Beam elongation for specimen Cyc (black line) and Mono (grey line).

Influence of Elongation. Beam elongation, plotted in Figure 6, was found to have a substantial influence on the behavior of the cyclic and monotonic specimens. In the cyclic specimen, flexural cracks did not fully close upon loading reversal due to lack of axial load to help yield reinforcement in compression. This led to increased crack widths with each cycle and cracks extending through the depth of the beam, reducing the shear friction capacity in the plastic hinge zone and forcing shear to be transferred through dowel action of the longitudinal bars. This caused bending of the bars, which led to buckling of reinforcement on subsequent cycles. In the monotonic specimen, the lack of load reversals prevented this behavior from occurring, and elongation was entirely due to movement of the neutral axis in the cracked length of the beam. The beam elongation in the cyclic test was on the order of 3% of beam depth, which is consistent with results from previous test programs, as discussed in (Fenwick & Megget, 1993). The mechanism of beam elongation is also further discussed in that reference. Note that in this test program, elongation was calculated as the average extension reading of two string potentiometers mounted at the beam centerline, with one on each side of the beam.

The permanent plastic strain in the longitudinal bars that creates the elongation is illustrated in Figure 7, showing the curvatures and strains in the tension reinforcement over the height of the specimen. It can be seen that the curvature distributions remain similar in both tests at increasing drift levels, but the strain in the tension side reinforcement diverges due to permanent growth in the cyclic test.

The lack of axial restraint for these specimens has resulted in larger reinforcement strain than one would typically expect for beams in buildings where columns and slabs restrain beam elongation. Given the importance of reinforcement strain on residual capacity, it has been decided to test two identical specimens with an elongation restriction mechanism that is representative of the restraint that would be provided in a typical RC moment frame. These specimens are marked with the suffix ER in Table 1.

Implications of Number of Loading Cycles for Residual Capacity Assessments. The stark contrast between the performances of the cyclically and monotonically tested specimen appears to indicate that the number of loading cycles is essential knowledge in order to accurately predict residual capacity. However, it is not clear how much of the degradation of the cyclic specimen was due to the cycles at lower drift levels. For example, in specimen Cyc, negligible stiffness or strength degradation, both cyclic and in-cycle, was observed at drift levels below 2.5%. It is these moderate drift levels, and hence moderate damage levels, that are of particular importance for residual capacity assessments, as the likelihood of a building being in an economically reparable state is higher for lower drift demands.

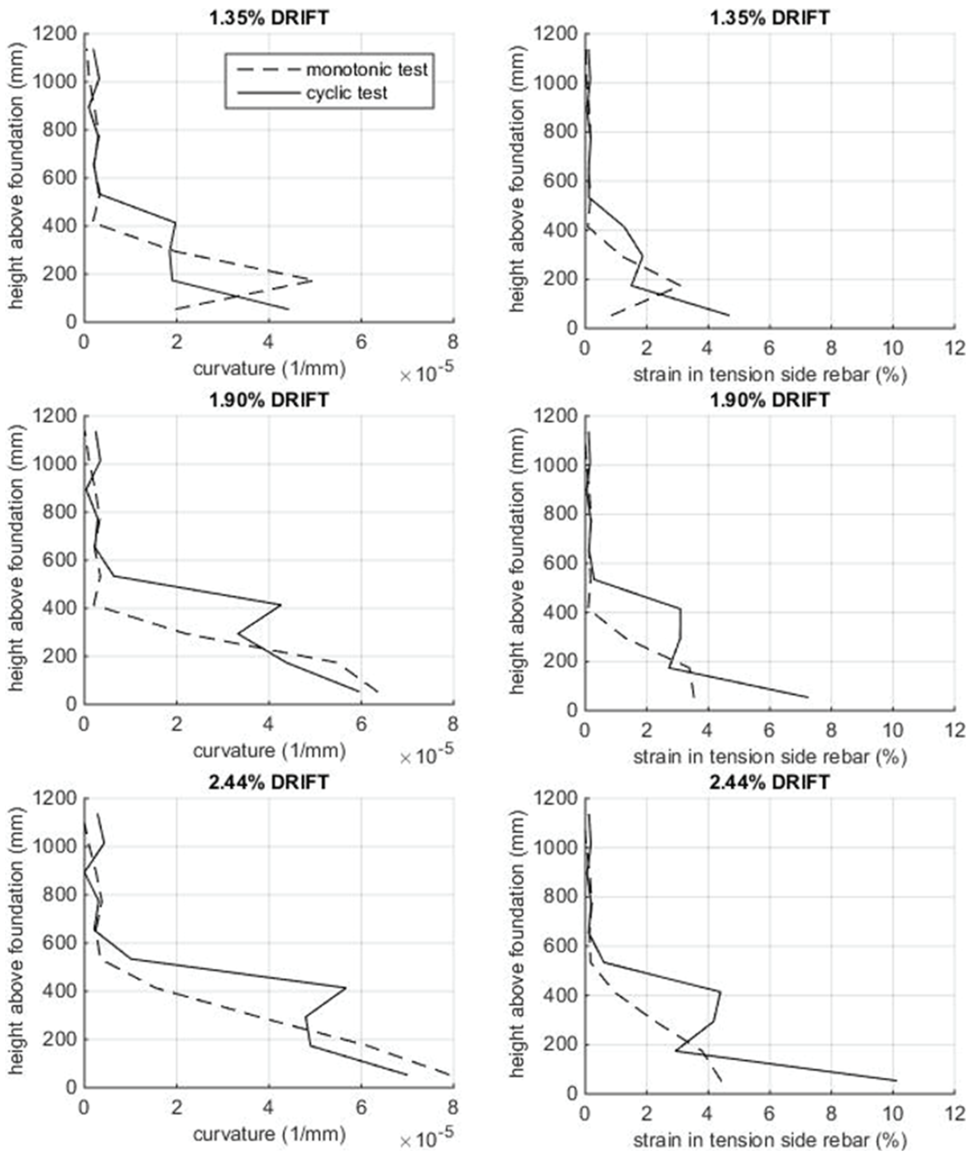


Figure 7. Distribution of curvatures and strains in tension side reinforcement at increasing drift levels for specimen Cyc (solid line) and Mono (dashed line).

Typical cyclic laboratory tests are primarily interested in ultimate or collapse limit states. For these purposes, cyclic loading protocols with progressively increasing displacements can be a conservative test methodology. However, limited information is gained about the effect of lower level cycles on future capacity. Upcoming test specimens in this experimental program will allow the degradation effect of cycles at lower drift levels to be comprehensively evaluated. Ongoing research presented in Cuevas Ramirez & Pampanin (2016) provides an assessment methodology for cases where the number of cycles are found to be critical.

Conclusions

A proposed framework for the detailed post-earthquake evaluation of RC buildings is introduced. Models required for such a framework to be applicable in engineering practice are discussed. Preliminary experimental results are presented emphasizing the importance of loading protocol and elongation on the behavior of RC beams subject to plastic hinging. The experimental program represents a critical component of a larger research effort to develop guidelines for the assessment of residual capacity of damaged RC buildings.

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