Reviewing uncertainties in seismic experimentation following the unexpected performance of RC structures in the 2010-2011 Canterbury earthquakes

2014 NZSEE Conference

G.J. Morris, D.K. Bull[1] & B.A. Bradley

Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, New Zealand.

[1]Holmes Consulting Group, New Zealand.

ABSTRACT: The performance of conventionally designed reinforced concrete (RC) structures during the 2011 Christchurch earthquake has demonstrated that there is greater uncertainty in the seismic performance of RC components than previously understood. RC frame and wall structures in the Christchurch central business district were observed to form undesirable cracks patterns in the plastic hinge region while yield penetration either side of cracks, and into development zones, were less than theoretical predictions. The implications of this unexpected behaviour: (i) significantly less available ductility; (ii) less hysteretic energy dissipation; and (iii) the localization of peak reinforcement strains, results in considerable doubt for the residual capacity of RC structures. The significance of these consequences has prompted a review of potential sources of uncertainty in seismic experimentation with the intention to improve the current confidence level for newly designed conventional RC structures. This paper attempts to revisit the principles of RC mechanics, in particular, to consider the influence of loading history, concrete tensile strength, and reinforcement ratio on the performance of 'real' RC structures compared to experimental test specimens.

1 INTRODUCTION

The current understanding of the seismic performance of structural components is largely based on the outcomes of on-going research developments by methods of experimental testing and, in more recent times, numerical modelling techniques. As damaging earthquakes occur relatively infrequently, many of the current assumptions for structural behaviour from these research developments are adopted in seismic design standards and guidelines in the form of empirical expressions. For a wide range of engineering applications, the information gained from examining the effects of damaging earthquakes provides the unique opportunity to assess whether the previous "research-based understanding" provides a reasonable comparison to field observations. In light of the 2010-2011 Canterbury earthquakes, the unexpected performance of several reinforced concrete (RC) buildings has highlighted the need to re-consider several aspects of seismic experimentation that may lead to an unreasonable representation of how 'real' RC structures perform under severe seismic actions. Wider aspects of the performance of RC buildings in the Christchurch central business district (CBD) have been documented by Kam et al., (2011), Bull (2012), Canterbury Earthquakes Royal Commission (CERC, 2012) and Fenwick (2013), among others.

2 EXPERIMENTAL OBSERVATIONS VS. REALITY

2.1 Typical experimental performance of RC components

Laboratory experiments of RC components subjected to quasi-static loading protocols have historically exhibited plastic hinge zone (PHZ) formation adjacent to a component's fixed end region. Figure 1(a) schematically illustrates the formation of diagonal flexure-shear crack patterns under seismic actions which promotes the redistribution of inelastic reinforcement bar strains. Countless experimental tests have shown this progressive cracking and spreading of incompatible strains between reinforcement and concrete, thereby increasing the length of the PHZ. This desired behaviour ensures the PHZ can sustain multiple cycles of inelastic deformation with significant hysteretic energy dissipation. The spreading of cracks is dependent on member geometry, whether there is sufficient tension force in the longitudinal reinforcement, and the strength of the concrete to resist tension. The size of the tension force is influenced by the stress-strain relationship of the reinforcement, the quantity of longitudinal reinforcement. If secondary cracks cannot form between primary cracks, very high reinforcement strains are induced and limited ductility can be sustained before the reinforcement fails (Fenwick, 2013).

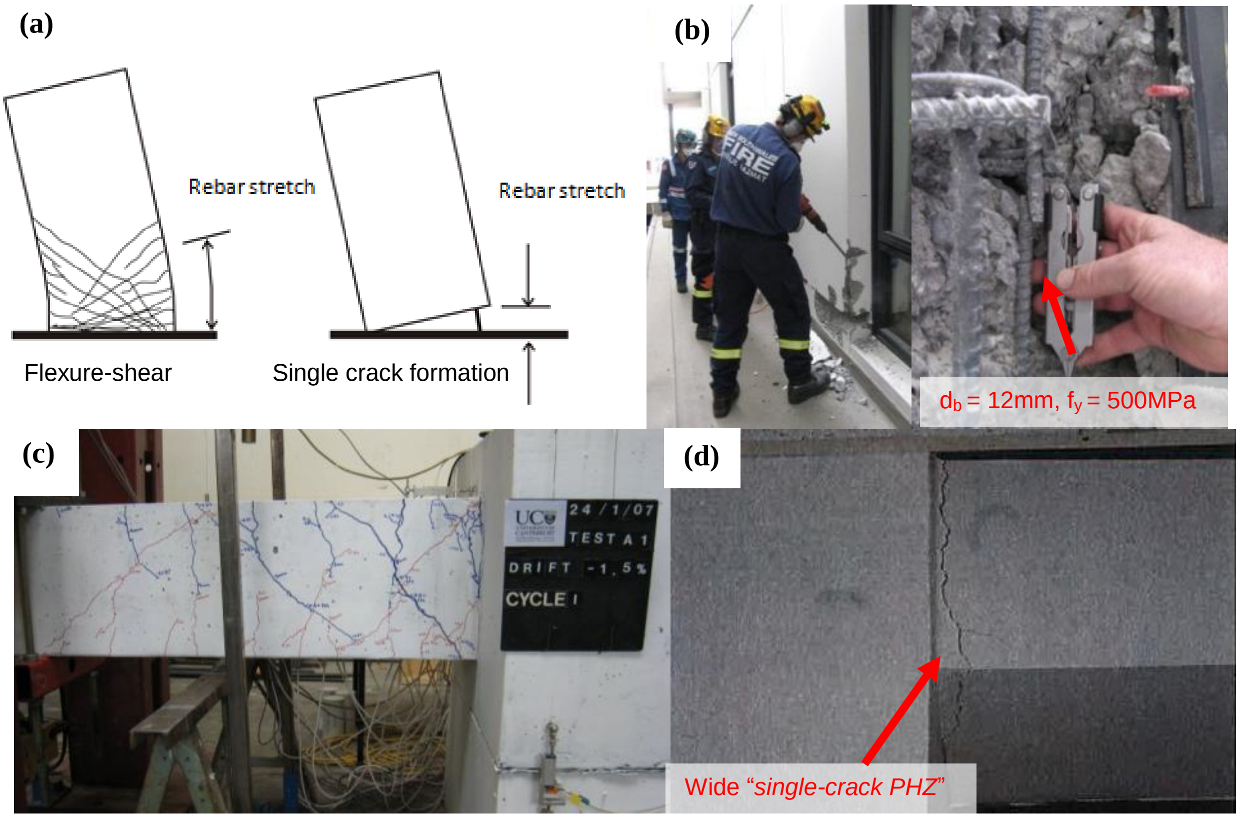


Figure 1. (a) Crack patterns in deflected walls; (b) 11 storey wall building in Christchurch CBD with USAR team removing cover concrete of fractured longitudinal reinforcement (Bull, 2012). (c) Distributed cracks observed in experimental testing (Walker, 2007); (d) inelastic deformations concentrated at the column face of RC frame buildings in Christchurch CBD (Smith and Devine, 2012a).

The spread of plasticity in real RC structures under seismic actions has long been expected to be consistent with experimental observations from laboratories around the world (Figure 1(c) for example). Many of the current assumptions for structural behaviour are based on observations described in the literature, such as Priestley and Park's (1984) compilation of research on the seismic performance of RC bridge columns. For the purpose of estimating ductility in design stages, or using seismic assessment methods (e.g. NZSEE, 2006), there is consistent agreement among researchers that half of the section depth, h_c, is a reasonable approximation for the "equivalent plastic hinge" length, L_p (Paulay and Priestley, 1992). Equations 3 is a widely adopted expression for L_p which is recommended by Priestley et al., 2007 to be more accurate (than say $L_p = 0.5h_c$). This expression was empirically derived from a database of experimentally measured section and member deformations such that curvature and displacement ductility relationships in Equations 1 and 2 could be re-arranged and solved for L_p. It is important to note all deformations were measured during the application of a generalised quasi-static loading protocol.

$$\mu_\phi = 1 + \frac{(\mu_\Delta - 1)}{3\left({}^{L_p}/_{L}\right)\left(1 - 0.5\,{}^{L_p}/_{L}\right)} \quad (1)$$

$$\mu_\Delta = 1 + 3(\mu_\phi - 1)\left({}^{L_p}/_{L}\right)\left(1 - 0.5\,{}^{L_p}/_{L}\right) \quad (2)$$

$$L_p = kL_c + L_{yp} \quad (3)$$

where:

$$k = 0.2\left(\frac{f_u}{f_y} - 1\right) \leq 0.08 \quad (4)$$

$$L_{yp} = 0.022 d_b f_y \quad (5)$$

Equation 4 defines the spread of plasticity along the member and is dependent on the constitutive stress-strain relationship of the reinforcement. Priestley and Park (1984) suggests that bond deterioration, due the tensile bar strains penetrating into development zones (e.g. wall footings and beam-column joints) and either side of flexural cracks, means the reinforcement may be expected to yield over a length of approximately 6-8 times the bar diameter. As RC joints are not fully rigid, the relative slip between the concrete and reinforcement may contribute significantly to inelastic deformation, particularly when subject to a large number of inelastic loading cycles, and is therefore considered when estimating the equivalent plastic hinge length. Equation 5 states the yield penetration length depends on the yield strength and diameter of the reinforcement; however, there is no consideration of bond mechanics at inelastic reinforcement strains.

2.2 **Observed performance of real RC structures**

Damage observations in the Christchurch CBD highlighted that several conventionally designed RC wall and frame structures (i.e. 'real' structures) had developed regions of concentrated inelastic deformations that compared poorly to the distributed PHZs observed in experimental testing. Instead, the potential PHZ comprised of wide single cracks (or wide well-spaced cracks) and limited strain penetrations (approx. 0.5-1.0d_b) were observed. Localized inelastic deformations resulted in the fracture of vertical reinforcement in numerous wall structures, including the Gallery Apartments building shown in Figure 1(b). Concentrated single crack PHZs were also widely observed at the critical regions of beams as shown in Figure 1(d). The implications of these observations is that RC structures generally behaved in a much less ductile manner than intended by the designer, and furthermore after the earthquake sequence there remains considerable uncertainty for the ability to resist future seismic actions. For example, Leeb hardness testing by Allington (2011) suggests there was approximately 1-2% of residual strain capacity over a very short length of longitudinal reinforcement sampled from the single-crack PHZ of beams that were designed as fully ductile plastic regions (NZS3101:2006).

In summary, damage states observed in earthquake field reconnaissance were not consistent with the spread of plasticity observed in previous experimental testing, thus highlighting the need to review and calibrate the current *research-basis* understanding for the behaviour of RC structures. The CERC (2012) made several recommendations for required research and revisions to the New Zealand Standard for Concrete Structures (referred to herein as NZS3101:2006) to address the unexpected performance of RC structures in the Christchurch CBD. The intention of interim practice guidelines by the NZ Structural Engineering Society (SESOC, 2012) is to offer some correction of long held assumptions on structural behaviour. The following sections discuss some potential factors that contributed to differences between experimental findings and recent field observations, including the use of quasi-static loading protocols, in-place concrete strength, and quantity of longitudinal reinforcement.

3 INFLUENCE OF LOADING HISTORY

3.1 Underlying issues with seismic experimentation

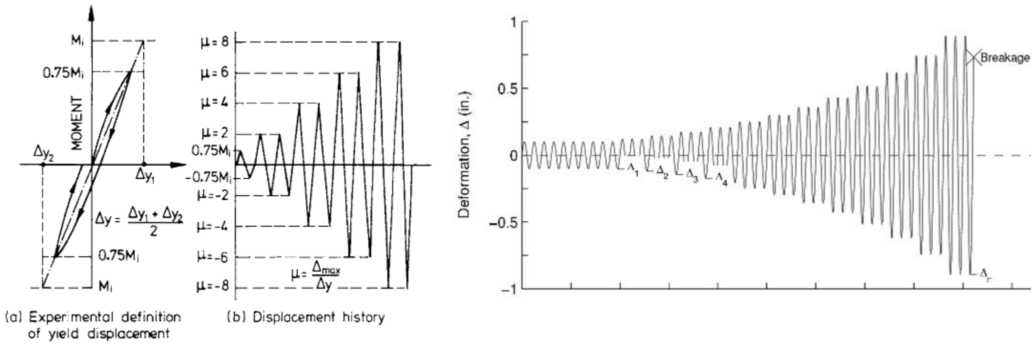
For many laboratories conducting seismic experimentation, shake-table or pseudo-dynamic testing is constrained by resources and practicality (e.g. cost, available equipment, required computer software, support of laboratory technicians). To avoid these constraints, quasi-static cyclic loading is the most widely implemented testing method for experiments (Leon and Deierlein, 1996). The results of quasi-static testing are assumed to provide a conservative lower bound for member strength capacity; however the same cannot be said for ductility and energy dissipation. The technical disadvantages are that quasi-static testing cannot consider: (i) the influence of the loading rate on governing failure mode and; (ii) variations in moment-shear ratios and axial load that largely influence the deformation and strength capacity. The deformation and strength capacity depends on the cumulative damage due to the path-dependent behaviour of RC in which the constituent materials have a memory of past inelastic cycles that influences the performance in future events (Krawinkler, 2009). For components within a real structure, the amplitude, frequency and number of cycles due to ground motion excitation depends on the:

- The influence of earthquake source rupture, seismic wave propagation and local site response on the features ground motion intensity: amplitude frequency content and duration.
- Configuration and relative strength of the component within the global system.
- Dynamic system properties such as stiffness, natural modes of vibration and characteristic inelastic response (ductility and hysteretic energy dissipation).

For several decades researchers have been aware of the need for generalized experimental loading protocols to reliably evaluate and compare the performance characteristics of structural components (Park, 1989). More recently, the popular notion of the performance-based design philosophy has highlighted the importance for performance indicators such as deformation capacity to be used in design procedures and standards. Loading protocols are recognised as a source of epistemic uncertainty associated with evaluating performance indicators (or damage states) in the development of component fragility functions used for performance-based seismic assessment (Bradley, 2010).

3.2 Quasi-static loading protocols

Liddel et al. (2000) found differences in the performance of RC components when subjected to varying quasi-static loading protocols used at different international research institutions. Loading protocols need to be reflective of the experimental objectives which may vary from determining potential failure modes to assessing the drift sensitivity of non-structural elements (FEMA-461, 2007). FEMA-461 suggests quasi-static loading protocols should be generalized such that the sequence of displacement cycles are in order of increasing magnitude to ensure that component performance is not unique for specific ground motions and configurations, but for a range of potential displacement histories. Figure 2(a) and 2(b) show typical loading protocols that have been widely used such that RC components undergo strength and stiffness degradation in a gradual manner. Under this type of loading, the spread of inelastic deformations is extensive with significant levels of deformation capacity and hysteretic energy dissipation being sustained, while premature failure modes such as bar buckling or bar fracture are mitigated. As described in Section 2.1, empirically-derived expressions for the equivalent plastic hinge length were based on outcomes of quasi-static testing using the loading protocol shown in Figure 2(a).



(a) Priestley and Park (1984) **(b)** Loading history stated in FEMA461 (2007)

Figure 2. (a) and (b) Typical loading protocols for quasi-static testing.

In contrast to typical loading protocols, near-source ground motions from damaging earthquakes, such as 1971 San Fernando (US), 1994 Northridge (US), 1995 Kobe (Japan) and 2011 Christchurch (NZ), can produce initially large amplitude, high frequency, and partially reversing loading histories without gradual increases in the amplitude and number of cycles. FEMA-461 (2007) ignores the influence of near-source ground motions on the basis that these motions generate fewer response cycles and therefore are not likely to control the number and relative amplitudes of the loading excursions in a loading history. Krawinkler (2009) discussed various loading protocols used for multi-institutional testing programmes and standards, such as those shown in Figure 3(a) and 3(b) for steel and timber structures that attempt to considered near-source ground motions with forward directivity; however no loading protocol for RC structures has been widely discussed in the literature.

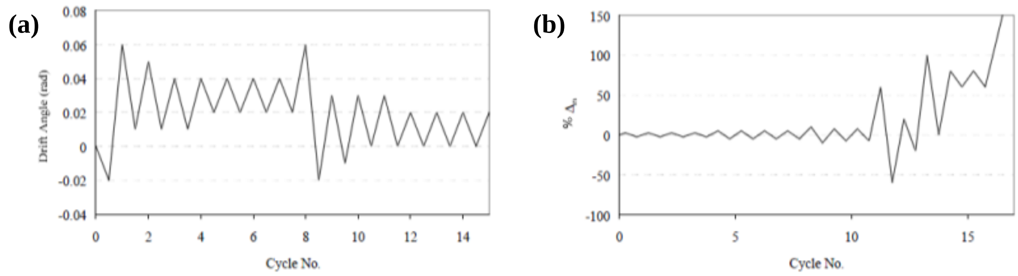


Figure 3. "Near-source" loading protocols for (a) structural steel components, and (b) timber components (Krawinkler, 2009).

3.3 Loading rate

Despite the awareness that near-source events result in dynamic large amplitude ground motions, there are few consistent conclusions in the literature for the influence of loading rate on the seismic performance of RC components. Quasi-static loading potentially mitigates brittle failure modes that are otherwise realistic for RC structures under actual seismic actions. Vos and Reinhardt (1982) found deformed bars had greater bond resistance when subject to faster loading rate and this influence was more pronounced for lower quality concrete. As the concrete matrix becomes more uniform in high quality concrete, the relative micro-crack propagation is limited and less relative concrete degradation occurs. Shah and Chung (1989) investigated the effect of loading rate on small scale anchorage-bond and beam-column joint specimens and observed fracture of the reinforcement when subjected to faster loading rates.

Phan et al. (2007) and Choi et al. (2010) compared shake-table motions containing large asymmetric pulses to motions from far-field earthquakes when testing RC bridge columns containing relatively high quantities of longitudinal reinforcement (between 2.0-3.6%). There was no evidence of concentrated inelastic deformations, which is in agreement with the CERC (2012); that the ductility of components with moderate or high reinforcement content is unlikely to be influenced by loading rates, however further investigations should be carried out for lightly reinforced components.

In the interest of producing favourable experimental outcomes without being constrained by the speed at which loading is applied, laboratory facilities within New Zealand must be upgraded so shake-table or pseudo-dynamic testing can be performed at an appropriate geometric scale and rate of loading.

4 IN-PLACE CONCRETE STRENGTH

Damage observations and materials testing from Christchurch CBD buildings indicated that the strength of concrete surrounding the reinforcement was notably higher than that specified in design. Higher than expected tensile strength of the concrete meant there was insufficient tension force in lightly reinforced components to progressively form cracks and distribute inelastic bar strains. The CERC (2012) report described the unexpected performance of several RC structures to be largely due to the re-occurring issue of higher than expected concrete strength. This section briefly discusses some evidence of higher than expected concrete strength, various factors that largely influence the concrete strength, and considerations for future laboratory research.

4.1 Materials testing

Material testing of samples extracted from a number of Christchurch CBD buildings illustrated the in-place strength was significantly higher than expected (CERC, 2012). The wall structure shown in Figure 1(b) is an example where the specified 28-day compressive strength, $f'_{c,28\,days}$, was 30 MPa. However, Allington (2011) found the strength of two extracted cores to be 46.5 MPa and 56.0 MPa, respectively, and Schmidt Hammer testing indicated a range of compressive strengths from 54 MPa to 70 MPa. Two split cylinder tests had measured tensile strengths of 2.4 MPa and 3.4 MPa, while Henry (2013) suggested the mean and upper characteristic flexural tensile strengths of the concrete may be as high as 4.3 MPa and 5.6 MPa, respectively (based on empirical correlations between the compressive and tensile strength). For cases where the effective concrete tensile strength was greater than that of the reinforcing steel, deformations were concentrated at a single crack as described in Section 2.2.

4.2 Sources of strength enhancement

For all concrete structures, the in-place strength will vary within and between concrete batches; and within and between components due to the influence of casting direction and size effects. The direction of casting relative to the orientation of the structural component will influence the concrete's mechanical properties due to water gain (Fenwick, 1982). Within-member strength will vary throughout the height of vertical elements as the concrete at the bottom of specimens is higher strength than the concrete in the middle, with lower strength at the top due to segregation of the mix materials. This suggests the concrete strength would have been substantially higher at the base of wall structures where limited crack propagation was observed. In the design of lightly reinforced components, Bull (2012) described the need to consider appreciable strength enhancements due to the following factors:

- Ready-mix suppliers targeting higher strength for quality assurance of the concrete product.
- The maturing process resulting in a time-strength development.
- Dynamic strength enhancements when subjected to rapid loading rates (the implications of which were alluded to in Section 3.3).
- Precast fabricators using high strength and high early strength mixes to meet specification quickly to ensure speed of production.

Flowable self-compacting mixes have led to extraordinarily high strengths that have not been anticipated by the design engineer. SESOC (2011) referred to a case example of a RC panel with a specified $f'_{c,28\,days}$ of 40 MPa however the self-compacting mix resulted in a 7-day of 90 MPa. Another case example, a relatively modern RC building, had precast wall panels with a 28-day strength of approximately 90 MPa such that the walls internal actions were higher than anticipated in design and subsequently contributed to failure of the foundations (Smith and Devine, 2012b).

4.3 Laboratory concrete

Many of the current design expressions that are influenced by concrete strength (such as minimum reinforcement content and development length) are empirically developed from experimental work. While such expressions may account for some scatter by carrying out an appropriate number of tests, there remains uncertainty in the use of concrete under laboratory conditions to be representative of concrete used in real construction. Since gaining an improved awareness of the importance of concrete strength on the formation of a desired PHZ, research laboratories and the New Zealand Concrete Industry should consider the use of laboratory concrete mixes for future experimental testing on specimens that are to be representative of real RC components. In experimental work, the as-tested concrete strength is typically very close to the specified design strength and the hardened concrete is relatively young (1-2 months) such that the tensile-compressive strength ratio may be appreciably less than for an existing structure of age. In test specimens, the tension force in the reinforcement is well proportioned to the tensile strength of the concrete such that there is no restriction on the progressive cracking along the RC component. There is a need for research into the rate at which tensile and compressive strengths develop with time.

5 MINIMUM LONGITUDINAL REINFORCEMENT

To achieve the desired flexural response of RC components, there needs to be sufficient tension force in the longitudinal reinforcement to progressively form cracks along the potential PHZ. The aim of code limitations for the minimum reinforcement quantity, ρ_{min}, is to prevent the formation of a single wide crack once the cracking moment of the section has been exceeded. To ensure a factor of safety against this undesired behaviour, the nominal moment capacity of a section with minimum reinforcement should be approximately 1.5-2.0 times the cracking moment (Paulay and Priestley, 1992). Henry (2013) further describes the background of the design expressions for the minimum reinforcement in RC beams and walls. The minimum reinforcement ratio stated in NZS3101:2006 for both walls and beams is given by:

$$\rho_n \geq \frac{0.25\sqrt{f_c'}}{f_y} \tag{6}$$

where f'_c = the specified 28-day strength (MPa) and ρ_n is calculated total longitudinal reinforcement as a ratio of the gross dimensions of the concrete member. While the expression appears identical for walls and beams, Henry (2013) described a number of differences between each component that will likely reduce the safety margin between the nominal and cracking moment capacity for walls. For example, the expression for walls is the total quantity of vertical reinforcement while for beams the expression represents only the quantity of reinforcement that is in tension. An important difference between RC test specimens and components in real structures is that test specimens will typically contain moderate and high quantities of reinforcement. To minimize concrete volumes and specimen weight, the geometry of test specimens is often reduced in scale such that tests specimens contain a higher proportion of reinforcement compared to real structures.

5.1 Wall structures

SESOC (2012) responded to poor performance of walls with a revised design recommendation for the minimum quantity and distribution of reinforcing (for walls that are likely to yield) to promote the spreading of inelastic deformation by control of concrete cracking. To account for the higher than expected concrete strength of up to 2.5 times the 28-day specified concrete strength, the NZS3101:2006 minimum vertical reinforcement for walls was modified as:

$$\rho_n \geq \frac{0.25\sqrt{2.5f_c'}}{f_y} \;\rightarrow\rightarrow\; \rho_n \geq \frac{0.4\sqrt{f_c'}}{f_y} \tag{7}$$

Henry (2013) presented results for the section analysis of the RC wall shown in Figure 1(b) for two cases: (i) with as-built details; and (ii) with the vertical reinforcement content approximately equal to the current NZS3101:2006 minimum. In the first case, the results were in agreement with the bar fracture that was observed while the second case depended on significant axial load to avoid sudden losses in strength after cracking. No experimental or analytical evidence was presented for the recommended quantity in Equation 7.

5.2 **Frame structures**

At present there has been no suggestion of revising the minimum reinforcement for beams, however further experimental investigations may address this issue. From a desktop study of the structural drawings for 21 RC frame buildings in the Christchurch CBD conducted by the authors, the average reinforcement ratios at the ductile regions of beams within the lateral load resisting “seismic frames” was relatively low, with approximately 0.85% and 0.70% for top and bottom reinforcement, respectively. The study focused on beam elevations in the lower third of the frame height, though beams located in upper levels of high-rise buildings typically contain a much lower proportion of reinforcement.

The CERC (2012) report described the case of beams containing sufficient longitudinal reinforcement such that secondary cracks were able to form however crack widths were generally very narrow (less than 0.05mm) and were not clearly visible. Bar yielding at secondary cracks can only occur if there is significant strain hardening at the nearby primary crack, meaning that appreciable strains must be induced and primary cracks need to be sufficiently wide (up to say 5mm).

Pending the outcomes of future experimental investigations, the minimum reinforcement for beams may be re-assessed.

6 **CONSIDERATIONS FOR FUTURE RESEARCH AND DESIGN PRACTISE**

At the forefront of required research is an investigation into the influence of the age of concrete on the tensile and compressive strength ratios. In an existing structure, the ratio of tensile to compressive strengths may be much higher than in test specimens that are typically tested within a month of the concrete pour. While this issue was not thoroughly discussed in this paper, it must be recognised as an implication for the future design and assessment of lightly reinforced components.

One implication of the observed behavior described in Section 2.2 is that current assumptions for the effective stiffness of RC structures may be inappropriate. Design standards such as NZS3101:2006 use multipliers of the gross-section properties that are based on extensive flexural cracking. Given that the flexural cracking observed in real RC structures was less than expected, the fundamental vibration period is likely to be less than what the structure was designed for and consequently the seismic forces may be higher than expected. Fenwick (2013) recommends that design actions from structural analysis are compared with the effective stiffness that is assumed.

While the response of individual components has long been studied in experimental work, the interaction between components and the influence on the global system response may need further investigation. For instance, interactions between floor slabs and beam elongation, the horizontal restraint of floor slabs that increases the strength of coupled shear walls and restricts diagonal cracking in the web of regular walls. The spread of inelastic deformations in a system of interacting components is likely to deviate from the performance of an individual RC component.

The issues described in this paper also indicates that there are many uncertainties that must be recognised when carrying out seismic assessment methods, which are largely based on the assumptions that are adopted in conventional design practise. At present, the engineering industry is challenged on the subject of finding the residual capacity of conventionally designed RC structures, partly due to the severity and number of strong ground motions in the Canterbury earthquakes sequence, and partly due to uncertainty for the effectiveness of structural repair techniques.

7 **CONCLUSION**

This paper discussed some features of the typical experimental conditions for previous research on the seismic performance of RC structures. Those features, including the applied loading type and rate, concrete strength and reinforcement quantity, are recognised as being quite different in the conditions of seismic experimentation compared to an existing structure that is subjected to real ground motions.

Many unexpected damage observations from the Canterbury earthquakes have prompted a revision of the behaviour of RC structures. Future research on this subject should attempt to circumvent some of the issues and constraints that were imposed on previous experimental work which produced vastly different damage observations compared to those from the Canterbury earthquakes.

8 ACKNOWLEDGEMENTS

Discussions with Prof. Richard Fenwick are greatly appreciated. Mr Sam McHattie is acknowledged for comments on this paper.

REFERENCES

Allington, C. (2011). Materials testing in buildings of interest: *Prepared for the Canterbury Earthquakes Royal Commission*, November 2011, Christchurch, New Zealand.

Bradley, B. A. (2010). Epistemic uncertainties in component fragility functions. *Earthquake Spectra*, 26(1), 41-62.

Bull, D. K. (2012). The performance of concrete structures in the Canterbury earthquakes: Lessons for concrete buildings, *Structural Engineers Association of California (SEAOC) Convention*, Santa Fe, NM.

Canterbury Earthquakes Royal Commission, (2012). Part 1: Volumes 1 to 3, Seismicity, Soils and the Seismic Design of Buildings. Available at: http://canterbury.royalcommission.govt.nz

Choi, H., Saadi, M. S., Somerville, P., and El-Azazy, S. (2010). Experimental study of reinforced concrete bridge columns subjected to near-fault ground motions. *ACI Structural Journal*, 107(1), 3-12.

Fenwick, R. C., and Sue, C. F. C (1982). The influence of water gain upon the tensile strength of concrete, Magazine of Concrete Research, 34(120): 139-145.

Fenwick, R. C. (2013). Aspects of the seismic performance of concrete buildings. *Conference proceedings of the New Zealand Concrete Society*, Queenstown, New Zealand.

Kam, W. Y., Pampanin, S., and Elwood., K. (2011). Seismic performance of reinforced concrete buildings in the 22 February Christchurch (Lyttelton) earthquake. *Bulletin of the New Zealand Society for Earthquake Engineering*; 44(4): 239-278.

Krawinkler, H. (2009). Loading histories for cyclic tests in support of performance assessment of structural components. *The 3rd International Conference on Advances in Experimental Structural Engineering*, October 15-16, San Francisco, CA.

Leon, R. T., and Deierlein, G. G. (1996). Considerations for the use of quasi-static testing. *Earthquake Spectra*, 12, 87-109.

Liddell, D., Ingham, J. M., and Davidson, B. J. (2000). Influence of loading history on ultimate displacement of concrete structures. *Report No. 597*, University of Auckland, Auckland, New Zealand.

NZSEE (2006). Assessment and improvement of the structural performance of buildings in earthquakes, *Recommendations of a study group of the New Zealand Society for Earthquake Engineering*.

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

Priestley, M. J. N., and Park, R. (1984). Strength and Ductility of bridge substructures. RRU Bulletin No. 71, National Roads Board, Wellington, New Zealand.

Paulay, T., and Priestley, M.J.N., (1992). Seismic design of reinforced concrete and masonry buildings. John Wiley & Son, Inc., New York, United States of America.

Phan, V., Saiidi, M. S., Anderson, J., and Ghasemi, H. (2007). Near-fault ground motion effect on reinforced concrete bridge columns. *Journal of Structural Engineering*, ASCE, 133(7), July, pp. 982-989.

Priestley, M. J. N., Kowalsky, M. J., and Calvi, G. M. (2007). Displacement-based seismic design of structures, IUSS PRESS, Pavia, Italy.

Smith, P.C., and Devine, J. (2012a). Independent assessment of earthquake performance of Craig's Investment Partners House – 90 Armagh Street, *Prepared for the Canterbury Earthquakes Royal Commission,*

Smith, P.C., and Devine, J. (2012b). Independent assessment of earthquake performance of Inland Revenue Department Building – 224 Cashell Street, *Prepared for the Canterbury Earthquakes Royal Commission.*

SESOC. (2011). Preliminary observations from Christchurch earthquakes. *Prepared for the Canterbury Earthquakes Royal Commission.*

SESOC. (2012). Practice note - Design of conventional structural systems following Canterbury earthquakes. *Prepared for the Canterbury Earthquakes Royal Commission.*

Standards New Zealand. (2006). Concrete structures standard, NZS3101, Parts 1 & 2. Standards New Zealand, Wellington, New Zealand.

Walker, A. (2007). Assessment of material strain limits for defining different forms of plastic hinge region in concrete structures. *Master of Engineering Thesis*, University of Canterbury, Christchurch, New Zealand.

Vos, E., and Reinhardt, H. W. (1982). Influence of loading rate on bond behaviour of reinforcing steel and prestressing strands, *Materials and Structures*. RILEM 15(85), 3-10.