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Post-Earthquake Residual Capacity of Reinforced Concrete Plastic Hinges

by

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A thesis submitted in partial fulfilment of the requirements for the degree of Doctor of Philosophy

Supervised by

Professor Kenneth John Elwood

Department of Civil and Environmental Engineering

The University of Auckland

December 2018
Abstract

Reinforced concrete buildings that satisfied modern seismic design criteria generally behaved as expected during the recent Canterbury and Kaikoura earthquakes in New Zealand, forming plastic hinges in intended locations. While this meant that life-safety performance objectives were met, widespread demolition and heavy economic losses took place in the aftermath of the earthquakes. The Christchurch central business district was particularly hard hit, with over 60% of the multi-storey reinforced concrete buildings being demolished. A lack of knowledge on the post-earthquake residual capacity of reinforced concrete buildings was a contributing factor to the mass demolition.

Many aspects related to the assessment of earthquake-damaged reinforced concrete buildings require further research. This thesis focusses on improving the state of knowledge on the post-earthquake residual capacity and reparability of moderately damaged plastic hinges, with an emphasis on plastic hinges typical of modern moment frame structures. The repair method focused on is epoxy injection of cracks and patching of spalled concrete. A targeted test program on seventeen nominally identical large-scale ductile reinforced concrete beams, three of which were repaired by epoxy injection following initial damaging loadings, was conducted to support these objectives. Test variables included the loading protocol, the loading rate, and the level of restraint to axial elongation.

The information that can be gleaned from post-earthquake damage surveys is investigated. It is shown that residual crack widths are dependent on residual deformations, and are not necessarily indicative of the maximum rotation demands or the plastic hinge residual capacity. The implications of various other types of damage typical of beam and column plastic hinges are also discussed.

Experimental data are used to demonstrate that the strength and deformation capacity of plastic hinges with modern seismic detailing are often unreduced as a result of moderate earthquake-induced damage, albeit with certain exceptions. Special attention is given to the effects of prior yielding of the longitudinal reinforcement, accounting for the low-cycle fatigue and strain ageing phenomena. A material-level testing program on the low-cycle fatigue behaviour of grade 300E reinforcing steel was conducted to supplement the data available in the literature.

A reduction in stiffness, relative to the initial secant stiffness to yield, occurs due to moderate plastic hinging damage. This reduction in stiffness is shown to be correlated with the ductility demand, and a proposed model gives a conservative lower-bound estimate of the residual stiffness following an arbitrary earthquake-type loading. Repair by epoxy injection is shown to be effective in restoring the majority of stiffness to plastic hinges in beams. Epoxy injection is also shown to have implications for the residual strength and elongation characteristics of repaired plastic hinges.
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Valuable contributions from many people and organizations enabled me to complete the work described in this thesis.

My supervisor, Professor Ken Elwood, invited me to follow him to the University of Auckland to undertake my doctoral studies. Without his encouragement, I would never have begun a Ph.D. program, or had the great experiences I have had over the past several years in New Zealand. Ken also supplied much appreciated guidance and enthusiasm throughout the duration of my research.

My experimental work was funded by a grant from the Natural Hazards Research Platform. My co-supervisor, Professor Charles Clifton, was the principal investigator of this grant and graciously covered the substantial expenses associated with large-scale experimental testing. Additional support for my experimental work was provided by Atlas Concrete, BBR Contech, and Sika New Zealand.

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Finally, this thesis is dedicated to my fiancé, Rosie Johnson.
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Chapter 1

Introduction

Over the past century, advances in earthquake engineering have led to greatly improved understanding on how buildings respond to seismic excitations. One of the primary strategies that has emerged for preventing collapse during major earthquakes is to allow for controlled damage to develop, which permits the building to absorb a portion of the seismic energy through material inelasticity. In reinforced concrete buildings, this is achieved through the yielding of reinforcing steel and the associated formation of plastic hinges, which are ‘capacity-designed’ to occur in specific locations that are detailed to withstand substantial inelasticity without exhibiting strength degradation. This strategy has been highly effective in preventing loss of life and has become a key component of the seismic provisions of many modern concrete design codes [1-4]. However, there is no guarantee about the post-earthquake usability or reparability of buildings designed in this manner. Recent research has addressed this by investigating low-damage technologies that aim to either prevent damage or localize it to easily replaceable sacrificial elements, but the majority of new and existing reinforced concrete buildings still consist of conventional monolithic construction.

Over 60% of the multi-story reinforced concrete buildings in the Christchurch central business district were demolished following the Canterbury earthquakes of 2010-2011 [5]. Many of these buildings were designed for ductility using capacity design principles, which were first introduced into New Zealand with the 1976 loading standard NZS 4203:1976 [6] and the 1982 reinforced concrete structures standard NZS 3101:1982 [7]. Reinforced concrete moment frames that were designed to these codes (and later iterations) largely behaved as intended during the Canterbury earthquakes, exhibiting plastic hinging in beams and limited damage to columns and joints, and yet many of these buildings were also ultimately demolished [8]. Similar observations were made following the 2016 Kaikoura earthquake, which resulted in the demolition of a number of relatively modern multi-story reinforced concrete frame buildings in Wellington [9].

A variety of factors influenced the decisions to demolish earthquake-damaged buildings following these recent earthquakes in New Zealand, including insurance policies and governmental regulations. However, the level of damage and assessed costs of repair were in many cases the governing considerations [5]. A lack of knowledge on the ability of earthquake-damaged buildings to withstand future seismic loadings (i.e. the ‘residual capacity’) contributed to the degree of demolition that took place, as engineers had little guidance on which to base their damage assessments or repair recommendations.
1.1 Objective

The overarching objective motivating this research is the development of formal guidelines for assessing the post-earthquake residual capacity and reparability of reinforced concrete buildings. A draft methodology for such a guideline, introduced in Appendix C, highlights several areas of research need, many of which are outside the scope of what is intended to be accomplished by the work described herein. This thesis focusses on improved understanding on the residual capacity and reparability of moderately damaged plastic hinges in reinforced concrete moment frame structures. The repair method focussed on is epoxy injection, due to it being one of the most well-established methods for repair of moderate earthquake damage. Photographs showing examples of the type of damage targeted in this thesis are shown in Figure 1-1.

1.2 Specific concerns after recent earthquakes in New Zealand

A number of specific areas of concern with regards to the residual capacity and reparability of reinforced concrete plastic hinges were identified following the 2010-2011 Canterbury earthquakes and the 2016 Kaikoura earthquake. The extent of cracking that was observed in plastic hinge zones was in some cases different from that typically seen in laboratory tests, with fewer and less well distributed wide cracks having formed than expected [10]. It was put forward that this may have been a consequence of (i) loading rate effects, (ii) light longitudinal reinforcement contents, (iii) higher than expected concrete tensile strengths, and (iv) higher than expected bond strengths (due to less bond degradation than in laboratory tests, where a progressively increasing cyclic loading that causes gradual bond deterioration is often used). Recommendations were made by the Canterbury Earthquakes Royal Commission for additional research into these factors [11].

The primary concern related to these unexpected crack patterns was the potential for higher than expected plastic strains in the longitudinal reinforcement. Prior plastic strain, in conjunction with the low-cycle fatigue and strain ageing phenomena that affect plastically-strained steel, can cause the residual strain capacity to be reduced relative to that of virgin steel [12]. This potential for reduced strain capacities in the reinforcing steel led in turn to a lack of confidence in the residual deformation capacity of plastic hinges. Many damaged buildings in Christchurch were subjected to invasive removal and testing of reinforcement in order to better assess the residual strain capacities. Concerns about the inability of epoxy injection repair to mitigate reduced strain capacities in the reinforcement may have also limited the uptake of repair in Christchurch, as in many cases full removal and replacement of the reinforcement was recommended as necessary to restore satisfactory capacity. A number of recent research projects in New Zealand have investigated the issues of low-cycle fatigue and strain ageing [12-14], but questions remain about the residual deformation capacities of moderately damaged plastic hinges.
Figure 1-1: Examples of moderate plastic hinging damage in reinforced concrete moment frames following the Christchurch and Kaikoura earthquakes
Elongation of ductile plastic hinge zones in the beams of moment frames was widespread [8, 9]. This beam elongation caused displacement incompatibilities with the flooring systems, resulting in wide cracks forming at the interface between precast floor slabs and beams, and potentially limiting the ability of the floor systems to sustain diaphragm actions and resist gravity loadings. In some cases, the magnitude of beam elongation was sufficient to cause collapse of precast flooring units due to a loss of support seating. The damage to floor systems was understandably the primary concern related to beam elongation, and the residual capacity of these floor systems is being addressed in a concurrent research project [15]. However, beam elongation also meant that additional forces and deformations not accounted for in design were imposed on the plastic hinges in the beams themselves. The impact that such forces and deformations have on the damage development and load-deformation response of plastic hinges warrants further research.

1.3 Existing methods for assessing residual capacity

Two guidelines for the detailed post-earthquake evaluation of damaged reinforced concrete buildings have previously been published. In the United States, the Applied Technology Council produced the guideline *FEMA 306: Evaluation of earthquake damaged concrete and masonry wall buildings* (1999) [16], along with two companion reports, FEMA 307 [17] and FEMA 308 [18]. The usage of FEMA 306 in post-earthquake evaluations is unknown and may be limited, possibly due to a lack of major earthquakes in the United States since its publication. In Japan, the Japan Building Disaster Prevention Association (JBDPA) published the Guideline for post-earthquake damage evaluation and rehabilitation (1991) (herein referred to as the ‘JBDPA Post-Earthquake Guideline’), which has since been revised several times [19]. This document has been used as a standard for post-earthquake evaluation of reinforced concrete buildings in Japan following a number of strong earthquakes. An English translation of the JBPDA Post-Earthquake Guideline has not been published, and this review therefore draws on information from a number of conference papers that were published with the purpose of describing the guideline [20-22], as well as personal communication with the authors.

A method for evaluation of buildings damaged by the 2010-2011 Canterbury earthquakes began being developed in New Zealand shortly after the September 2010 Darfield earthquake [23]. Although this document was used as a reference on which detailed post-earthquake evaluations were based [24], it has not progressed past a preliminary draft phase. Furthermore, the section applicable to reinforced concrete moment frames makes minimal recommendations regarding damage and is more reflective of a ‘pre-earthquake’ seismic assessment. This document is therefore not discussed here in detail.
1.3.1 **FEMA 306**

The FEMA 306 methodology is based around non-linear static assessment procedures, which are performed on models of both the undamaged and damaged structure. Undamaged and damaged backbone curves for the structural components are input into a building model in order to develop the respective undamaged and damaged global capacity curves. The displacement demand is determined using a response spectrum corresponding to the desired hazard level and either of the ‘displacement coefficient method’ or ‘capacity spectrum method’. The reduction in capacity of the damaged structure is quantified as the change in the ratio of displacement capacity to displacement demand between the undamaged and damaged models. If the reduction in capacity causes the structure to fail to meet its performance objectives, repair is recommended, and analysis must again be performed using component backbone curves corresponding to the repaired capacities.

The undamaged component-level backbone curves are defined based largely on the calculation procedures and modelling parameter tables of FEMA 273 [25], one of the precursors of the modern ASCE 41-17 [26] seismic assessment code in the United States. The backbone curves for the damaged and repaired components are given in FEMA 306, and are derived using reduction factors ($\lambda$ factors) that relate the residual stiffness, strength, and deformability of the damaged components to that of the undamaged components (Figure 1-2).

![Figure 1-2: FEMA 306 component backbone curves and reduction factors, after [16]](image)

A database of experimental tests with static cyclic loading protocols was used to derive the reduction factors by direct observation of the cycle-to-cycle changes in the load-deformation responses. Components are separated into different classes based on (i) the member type and its relative strength within the structural system (e.g. strong wall, weak wall, weak coupling beam, etc.), and (ii) the behaviour mode (i.e. the expected failure mode). The identification of component classes can be an iterative procedure involving both theoretical calculations and damage observations.
Critical damage indicators, which are used to define five discrete damage states (insignificant, slight, moderate, heavy, and extreme) for each component class, were also determined based on observations from the experimental database. The differentiation between damage states is based on a combination of crack widths and qualitative descriptions of the damage, sometimes accompanied by an illustration.

The approximate relationships between the three reduction factors and the damage states is shown in Figure 1-3. For every component class, different reduction factors are provided for each of the five damage states. Cyclic tests could not be used to develop the reduction factors for deformability as there was no benchmark deformation capacity against which the reduction factor could be calibrated. Instead, tests that evaluated the effects of variable loading protocols (e.g. El-Bahy et al. [27]) are cited as the basis for the deformability reduction factor values. In addition to the deformability reduction factor, the deformation capacity of damaged components is considered to be directly reduced by the observed residual deformation (Figure 1-2).

As FEMA 306 is targeted at wall buildings, the component types for which reduction factors are given are limited to walls and coupling beams. The reduction factors for the components classified with predominantly flexural behaviour modes are of particular interest for the purposes of this review. For these flexural elements, only stiffness degradation is considered to have occurred for components in an ‘insignificant’ or ‘slight’ damage state, which are demarcated by maximum residual crack widths of no larger than 3/16 in. (4.8mm) or 1/4 in. (6.4mm), respectively. Degradation of strength ($\lambda_Q = 0.8$) and deformability ($\lambda_D = 0.9$) is considered to first occur for components in a ‘moderate’ damage state, which is defined by spalling or vertical cracking of the cover concrete in the plastic hinge zone.

For components with ductile behaviour modes and damage states up to and including ‘moderate’, repair by patching of loose concrete and epoxy injection of cracks is considered to fully restore the strength and deformation capacities and restore 80-90% of the initial stiffness. For damage states
Introduction

beyond ‘moderate’, full replacement of the damaged component is usually recommended. The information on which the repaired reduction factors are based is not made clear.

1.3.2 JBDPA Post-Earthquake Guideline

The JBDPA Post-Earthquake Guideline builds on the seismic performance index \((I_5)\) methodology that is used in the Japanese seismic assessment code for reinforced concrete buildings [28]. The \(I_5\) index is a measure of the strength and ductility of each story in a building, and also accounts for structural irregularities, building age, and other relevant parameters. In order to determine the residual seismic capacity, the \(I_5\) index must be calculated for both the undamaged (i.e. pre-earthquake) and damaged (termed \(\rho I_5\)) states. The residual seismic capacity is then quantified as the ratio of the damaged seismic capacity index to the undamaged seismic capacity index \((\rho I_5 / I_5)\).

The methods for determining the undamaged component contributions to the \(I_5\) index are laid out in the seismic assessment code [28]. Different calculation methods can be used, depending on the level of detail of the evaluation. The reduction in component contributions due to earthquake damage (i.e. the component contributions to the \(\rho I_5\) index) are quantified in terms of a single reduction factor (termed \(\eta\) factor) for each component.

The \(\eta\) factors were derived empirically, based on the energy-dependent theoretical framework shown in Figure 1-4, using experimental data from cyclic tests on a variety of reinforced concrete components (e.g. [21]). Components are separated into different categories depending on the member type (column, beam, or wall) and whether they are ‘ductile’, ‘quasi-ductile’, or ‘brittle’. Component damage is discretized into five different damage classes, with classes I-IV being defined based on a combination of maximum residual crack widths and a description of the degree of spalling.

\[
\eta = \frac{E_r}{E_d + E_r}
\]

*Figure 1-4: Theoretical framework for reduction factors in the JBDPA Post-Earthquake Guideline, after [22]*
Experimental residual crack width data were compared with the experimental \( \eta \) factors, in order to derive a relationship between residual crack widths and the \( \eta \) factor (shown in Figure 1-5(a) and Figure 1-5(b) for ‘ductile’ and ‘brittle’ components, respectively). A complete list of the \( \eta \) factor values for all different component categories and damage classes is given in Table 1-1. It can be seen that ductile components are reduced to a \( \eta \) factor of 0.5 or less after reaching damage class III, which is defined by residual crack widths of ‘about 1.0-2.0mm’ or localized spalling of cover concrete. A maximum residual crack width of 2.0mm is given as the limit between damage classes III and IV.

![Figure 1-5: JBDPA Post-Earthquake Guideline reduction factor equations and comparison against component test data for (a) ductile, and (b) brittle columns, after [21]](image)

**Table 1-1: Reduction factors in the JBDPA Post-Earthquake Guideline, after [22]**

<table>
<thead>
<tr>
<th>Damage class</th>
<th>Ductile column</th>
<th>Quasi-ductile column</th>
<th>Brittle column</th>
<th>Ductile beam</th>
<th>Brittle beam</th>
<th>Ductile shear wall</th>
<th>Brittle shear wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
</tr>
<tr>
<td>II</td>
<td>0.75</td>
<td>0.7</td>
<td>0.6</td>
<td>0.75</td>
<td>0.7</td>
<td>0.7</td>
<td>0.6</td>
</tr>
<tr>
<td>III</td>
<td>0.5</td>
<td>0.4</td>
<td>0.3</td>
<td>0.5</td>
<td>0.4</td>
<td>0.4</td>
<td>0.3</td>
</tr>
<tr>
<td>IV</td>
<td>0.2</td>
<td>0.1</td>
<td>0</td>
<td>0.2</td>
<td>0.1</td>
<td>0.1</td>
<td>0</td>
</tr>
<tr>
<td>V</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

It is noted that early versions of the JBDPA Post-Earthquake guideline (and the Japanese seismic assessment code [28]) focussed predominately on non-ductile or story-collapse failure mechanisms, as evidenced by the \( I_s \) index being primarily a measure of story behaviour. The supporting research has therefore largely focussed on column and wall behaviour. Only the most recent version of the guideline considers the possibility of a strong-column-weak-beam building mechanism [22].
1.3.3 Discussion

Both FEMA 306 and the JBDPA Post-Earthquake Guideline provide guidance on how to assess the residual capacity of earthquake-damaged reinforced concrete buildings. The primary way in which these post-earthquake assessment guidelines differ from the ‘pre-earthquake’ seismic assessment guidelines in place in their respective countries is through a structural capacity that is modified to account for the observed damage. Certain aspects of the methodologies employed to calculate the residual capacity of damaged components are discussed here.

The use of a single component-level reduction factor in the JBDPA Post-Earthquake Guideline allows for a simpler, and likely more expedient, analysis procedure than FEMA 306. However, the use of a single component-level reduction factor in the JBDPA Post-Earthquake Guideline is made possible due to its compatibility with the analysis methods employed in the Japanese seismic assessment code for reinforced concrete buildings [28]. A single component-level reduction factor is not compatible with either the New Zealand seismic assessment guidelines [29] or ASCE 41-17, and the decomposition of residual capacity into residual stiffness, strength, and deformation capacity, as is done in FEMA 306, is therefore preferred in a New Zealand context.

FEMA 306 and the JBDPA Post-Earthquake Guideline both provide guidance on how to quantify the residual capacity of components in severe damage states. They also provide guidance on how to quantify the residual capacity of components exhibiting damage consistent with non-ductile or ‘brittle’ failure modes. These types of damage are critical for determining the aftershock collapse vulnerability of a building, and the observation of such damage in post-earthquake assessments can necessitate emergency repairs. However, attempting to quantify the damaged capacity of components exhibiting such severe damage may be impractical, as they would likely be required to undergo either replacement or retrofit, regardless of their assessed residual capacity. There is little rationale in conducting detailed post-earthquake assessments to quantify the residual capacity of severely damaged buildings that will either be demolished or have their capacity completely altered through repair or retrofit. This is particularly relevant for more complex and time-consuming assessment procedures, such as that of FEMA 306. The simpler JBDPA Post-Earthquake Guideline is partially intended for identifying buildings requiring emergency repair.

FEMA 306 and the JBDPA Post-Earthquake Guideline both use the observable damage as the primary metric upon which the reduction factors for damaged components are determined. The reduction factors were derived using experimental data from reversed-cyclic tests with progressively increasing deformation demands. Such tests necessarily exhibit progressively increasing damage states, which makes it difficult to isolate the effects that a particular damage state has on the component residual capacity. Additionally, in reversed-cyclic laboratory experiments, damage observations are typically made at or near the peak demand. However, the
observable damage in a post-earthquake situation may not always reflect the peak demands incurred, but rather the residual state of the building. Experiments that can better isolate the effects of particular damage states and better reflect the damage states after earthquake loadings are required. Incorporation of other metrics besides the observable damage, such as an estimate of the maximum deformation demand incurred during the damaging earthquake, may also be beneficial for assessing residual capacity. This is reflected in the draft methodology for post-earthquake assessments presented in Appendix C.

Finally, the methods for calculating component residual capacity employed in FEMA 306 and the JBDPA Post-Earthquake Guideline do not directly address any of the concerns previously discussed in Section 1.2, including the implications of poorly distributed flexural cracking, the residual strain capacity and fatigue life of previously yielded reinforcement, and the effects of axial elongation. Experiments are required to investigate these issues.

### 1.4 Conceptual framework of residual capacity assessments

Figure 1-6 illustrates a conceptual framework of the different types of residual capacity assessments that may be conducted on earthquake-damaged reinforced concrete buildings that are not demolished. The framework is based on the assertion that the post-earthquake assessment of buildings that will not be occupied prior to repair should focus on quantifying the repaired capacity. The work in this thesis is largely focussed on moderately damaged buildings, which may or may not be occupied prior to repair. In either case, it is necessary to quantify the repaired capacity. However, in moderately damaged buildings that are occupied in post-earthquake situations, it may also be necessary to quantify the residual capacity in the damaged state, to verify that continued occupancy is safe. A conservative approach is to assume that any building exhibiting structural damage is not suitable for occupancy, but this level of disruption to society may not be necessary in cases of moderate damage. For example, ASCE 41-17 allows for some inelasticity to occur in well-detailed flexural elements prior to reaching the ‘immediate occupancy’ acceptance criteria, which is stated to occur at plastic rotations of 0.01 radians in the most ductile classification of beams and 0.005 radians in the most ductile classification of columns. Research is required to validate the appropriateness of these limits.

Moderate plastic hinging damage is here defined as damage that does not necessarily indicate a critical life-safety concern. It is further classified by typically being repairable through concrete patching and epoxy injection. Both FEMA 306 (ductile walls and coupling beams), and Lehman et al. [30] (ductile bridge columns) define comparable ‘moderate’ damage states, which include concrete cracking and spalling, but exclude any occurrence of reinforcement buckling or fracture, or any crushing of core concrete. A similar range of damage states that constitute ‘moderate’
damage is employed here, as shown in Table 1-2. Wide diagonal cracks crossing transverse reinforcement is here considered to be a severe damage state, based on the assumption that it indicates yielding of the transverse reinforcement. The definition of ‘wide’ used here is taken as 1/16in (1.5mm), based on the distinction between ‘insignificant’ and ‘moderate’ shear crack widths used in FEMA 306. The classification of this damage state as severe is a conservative approach, as minor yielding in the transverse reinforcement does not necessarily indicate a life-safety concern or a lack of reparability by epoxy injection.

Figure 1-6: Conceptual framework of the different types of residual capacity assessments for earthquake-damaged reinforced concrete buildings

<table>
<thead>
<tr>
<th>Damage</th>
<th>Moderate / Severe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexural cracking</td>
<td>Moderate or less</td>
</tr>
<tr>
<td>Longitudinal cracking</td>
<td>Moderate</td>
</tr>
<tr>
<td>Cover concrete spalling</td>
<td>Moderate</td>
</tr>
<tr>
<td>Sliding plane crack(s)</td>
<td>Moderate</td>
</tr>
<tr>
<td>Wide (&gt;1.5mm) diagonal crack(s) crossing transverse reinforcement</td>
<td>Severe</td>
</tr>
<tr>
<td>Longitudinal reinforcement buckling</td>
<td>Severe</td>
</tr>
<tr>
<td>Core concrete crushing</td>
<td>Severe</td>
</tr>
<tr>
<td>Reinforcement fracture</td>
<td>Severe</td>
</tr>
</tbody>
</table>
1.5 Definition of plastic hinge residual capacity

The term ‘residual capacity’ is herein used to refer to the key structural characteristics of a plastic hinge after an earthquake, namely the residual stiffness, residual strength, overstrength, and residual deformation capacity, as illustrated in Figure 1-7. These parameters are consistent with those used to define the capacity of undamaged components in design or assessment (also shown in Figure 1-7). The definition of residual capacity shown in Figure 1-7 is compatible with non-linear assessment procedures, such as those used in the New Zealand seismic assessment guidelines [29] or the non-linear static or dynamic procedures of ASCE 41-17. For this definition to be compatible with linear assessment procedures, the deformation capacity parameter must be modified (e.g. converting to an $m$-factor for use with the linear assessment procedure of ASCE 41-17). However, no guidance is given in this thesis on how to perform such a modification, and non-linear assessment procedures are here preferred.

The following chapters provide guidance on how to calculate the residual capacity, both in a damaged and an epoxy-repaired state, for moderately damaged plastic hinges in reinforced concrete moment frames. In some cases, it is recommended that the residual parameters be calculated as a function of the undamaged (pre-earthquake) parameters, i.e. the ‘reduction factor’ method used in FEMA 306. Explicit calculations of the residual capacity parameters using first principles are recommended in other situations. In certain cases, it is recommended that the residual deformation capacity be calculated using a curvature capacity and equivalent plastic hinge length approach, which is one of the options included in in the New Zealand seismic assessment guidelines, but is not fully compatible with empirical rotation-based limits, such as those used in ASCE 41-17.

It is noted that cumulative parameters such as energy dissipation are not included in the plastic hinge residual capacity definition shown in Figure 1-7. Cumulative parameters are not considered in modern standards for seismic design or assessment, but there are concerns that some portion of the energy dissipation capacity or fatigue life can be ‘consumed’ during a damaging earthquake, making neglect of such parameters no longer appropriate in post-earthquake situations.

The research in the forthcoming chapters shows that neglecting cumulative parameters in a post-earthquake assessment is often justified. However, cumulative parameters may be necessary to consider in certain situations. Research focussed on developing an assessment procedure capable of accounting for cumulative parameters is ongoing [14].
Introduction

Figure 1-7: Comparison of the parameters used to define plastic hinge residual capacity with the parameters typically used in design or assessment

1.6 Summary

Many relatively modern reinforced concrete moment frame buildings that exhibited moderate levels of damage have been demolished following recent earthquakes in New Zealand. While several factors contributed to the decisions to demolish these buildings, this thesis focusses on the concerns related to the residual capacity and reparability of the lateral load-resisting system. Two existing documents, FEMA 306 and the JBDPA Post-Earthquake Guideline, provide guidance on these issues, but additional research is required.

A conceptual framework was introduced to illustrate that, depending on the situation, it can be useful to quantify the residual capacity of moderately damaged buildings in either their damaged or repaired states. The remaining chapters of this thesis aim to improve the state of knowledge on the residual capacity of damaged and epoxy injection-repaired plastic hinges in moment frames. Indirectly related topics that were raised as concerns in the aftermath of recent earthquakes are also investigated.
1.7 Layout of thesis

Chapter 2 describes an experimental program on seventeen nominally identical reinforced concrete beams that was conducted as part of this work. Detailed test results for individual specimens are also presented. Comparative analysis between specimens is not shown, and is instead included in the relevant sections of Chapters 3 to 6.

Chapter 3 investigates the effects that load history, loading rate, and axial elongation have on the seismic performance of plastic hinges in reinforced concrete moment frames, with a focus on the residual behaviour after moderate earthquake loadings. Literature reviews for each topic are included, followed by analysis of the experimental data and comparison of the data against relevant models.

Chapter 4 examines the effects that moderate earthquake-induced damage has on the residual capacity of plastic hinges in reinforced concrete moment frames. Discussion is also provided on the degree to which visual damage metrics are representative of the peak deformation demands incurred during the damaging earthquake.

Chapter 5 investigates the effects that yielded longitudinal reinforcement has on the residual capacity of plastic hinges in reinforced concrete moment frames. Special attention is given to the low-cycle fatigue and strain ageing phenomena, using data from a variety of material-level studies.

Chapter 6 makes use of available test data to quantify the effects of repair by epoxy injection on the residual capacity of plastic hinges in reinforced concrete moment frames.

Chapter 7 presents the recommendations and conclusions arising from this work.

Appendix A provides additional information on the data processing methods used in analysis of the data from the experimental program described in Chapter 3.

Appendix B provides a discussion and photographs of the damage progression for each of the test specimens from the experimental program described in Chapter 3.
Appendix C presents a draft framework for a detailed post-earthquake assessment procedure in New Zealand, and discusses the research required before such a framework is practically applicable.
Chapter 2

Experimental program

This chapter presents a description of, and results from, an experimental program investigating the seismic performance of seventeen large-scale nominally identical ductile reinforced concrete beams. Comparative analysis between specimens or against models is not included; that analysis is instead available in the relevant sections of Chapters 3 to 6. The test data have been made publicly available at DOI 10.17603/DS2SQ2K.

2.1 Objectives and test matrix

The experimental program was designed to provide data on a number of objectives relevant to the behaviour of reinforced concrete plastic hinges, including assessment of the residual capacity following an initial damaging earthquake loading. The primary objectives were as follows:

- Improve understanding on how variations in the loading characteristics (i.e. cycle content and loading rate) at various levels of displacement demand affect the ultimate capacity of plastic hinges in modern reinforced concrete ductile moment frame buildings, including capturing any possible low-cycle fatigue effects.
- Provide data for validation of models estimating the residual stiffness, strength, and deformation capacity of earthquake-damaged reinforced concrete components.
- Evaluate the efficacy of typical quasi-static cyclic testing procedures for use in determining residual properties following moderate displacement demands.
- Investigate the effects of epoxy-injection repair on the residual capacity of a plastic hinge, relative to the capacity of that same hinge if left unrepaired in a damaged state.
- Determine the effects that elongation-restraining boundary conditions have on the performance of a beam plastic hinge.

To achieve these objectives, seventeen nominally identical beam specimens were tested under various loading protocols, loading rates, and levels of restraint to axial elongation, as shown in Table 2-1. This included use of a novel loading protocol whereby specimens were subject to dynamic earthquake loading followed by cycles to failure, as discussed in Section 2.5. Given the large number of objectives and the time-consuming nature of large-scale testing, it was decided that variations in specimen detailing and design, although desirable, was not feasible for this test
program. Three specimens were repaired by epoxy injection following an initial damaging earthquake loading.

**Table 2-1: Test matrix**

<table>
<thead>
<tr>
<th>Specimen name</th>
<th>Initial damaging loading</th>
<th>Failure loading</th>
<th>Repair</th>
<th>Elongation restraint</th>
</tr>
</thead>
<tbody>
<tr>
<td>MONO</td>
<td>-</td>
<td>Static monotonic</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>CYC</td>
<td>-</td>
<td>Static cyclic</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>CYC-DYN</td>
<td>-</td>
<td>Dynamic cyclic</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>P-1</td>
<td>Dynamic pulse-type displacement history to 1.36% drift</td>
<td>Static cyclic (cycles above 1.36% drift only)</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>LD-1</td>
<td>Dynamic long duration displacement history to 1.36% drift</td>
<td>Static cyclic (cycles above 1.36% drift only)</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>P-2</td>
<td>Dynamic pulse-type displacement history to 2.17% drift</td>
<td>Static cyclic (cycles above 2.17% drift only)</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>P-2-S</td>
<td>Static pulse-type displacement history to 2.17% drift</td>
<td>Static cyclic (cycles above 2.17% drift only)</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>LD-2</td>
<td>Dynamic long duration displacement history to 2.17% drift</td>
<td>Static cyclic (cycles above 2.17% drift only)</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>LD-2-S</td>
<td>Static long duration displacement history to 2.17% drift</td>
<td>Static cyclic (cycles above 2.17% drift only)</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>CYC-NOEQ</td>
<td>-</td>
<td>Static cyclic (cycles above 2.17% drift only)</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>LD-1-R</td>
<td>Dynamic long duration displacement history to 1.36% drift</td>
<td>Static cyclic (cycles above 1.36% drift only)</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>LD-2-R</td>
<td>Dynamic long duration displacement history to 2.17% drift</td>
<td>Static cyclic (cycles above 2.17% drift only)</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>LD-2-ER</td>
<td>Dynamic long duration displacement history to 2.17% drift</td>
<td>Static cyclic (cycles above 2.17% drift only)</td>
<td>No</td>
<td>Yes: ~40kN/mm</td>
</tr>
<tr>
<td>CYC-ER</td>
<td>-</td>
<td>Static cyclic</td>
<td>No</td>
<td>Yes: ~40kN/mm</td>
</tr>
<tr>
<td>LD-2-LER</td>
<td>Dynamic long duration displacement history to 2.17% drift</td>
<td>Static cyclic (cycles above 2.17% drift only)</td>
<td>No</td>
<td>Yes: ~15kN/mm</td>
</tr>
<tr>
<td>CYC-LER</td>
<td>-</td>
<td>Static cyclic</td>
<td>No</td>
<td>Yes: ~15kN/mm</td>
</tr>
<tr>
<td>LD-2-LER-R</td>
<td>Dynamic long duration displacement history to 2.17% drift</td>
<td>Static cyclic (cycles above 2.17% drift only)</td>
<td>Yes</td>
<td>~15kN/mm</td>
</tr>
</tbody>
</table>

1 Naming convention: for specimens with an initial damaging loading, LD or P refers to the type of earthquake (long duration or pulse), −1 or −2 refers to the peak drift demand of the earthquake loading rounded to the nearest whole number, and −S refers to the earthquake being ran at a quasi-static loading rate. For specimens with axial restraint, –ER refers to a high level of restraint and –LER refers to a light level of restraint. All specimens that were repaired following the initial damaging loading are marked by the suffix –R.
2.2 Test specimen

The test specimen was chosen to be a beam due to this being the primary location of plastic hinging in modern moment frame structures, which are generally required to be capacity designed to form a strong-column-weak-beam mechanism. The specimen was scaled by a factor of 0.8 from the second-story beams of the perimeter frame in the prototype moment frame building described in [31], a design guideline for reinforced concrete buildings in New Zealand. An elevation view of one of the perimeter moment frames is shown in Figure 2-1. The building was designed according to NZS 3101:2006 [1] and NZS 1170.5:2004 [32] for the pre-Canterbury Earthquakes design spectrum in Christchurch, with a ductility factor $\mu$ of 4 and a structural performance factor $S_P$ of 0.7, resulting in a design base shear equivalent to 3.2% of the weight of the building.

The corresponding column and beam-column joint from the building was not included in the test specimen, as the frame building was capacity-designed to create a strong-column-weak-beam mechanism and prevent damage in other components. Instead, a foundation designed to remain elastic was used to provide fixity at the base of the beam specimens.

![Figure 2-1: 2D perimeter frame elevation of selected prototype building, after [31]](image-url)
2.2.1 Specimen details

A technical drawing of the specimen detailing is presented in Figure 2-2 and a list of the design material properties is shown in Table 2-2. The specimen detailing meets all requirements for a ductile (or special) moment frame beam in the seismic provisions of modern reinforced concrete design codes (e.g. NZS 3101:2006, ACI 318-14 [2], CSA A23.3-14 [3], EN 1998-1:2004 [4]). The design longitudinal reinforcement ratio was 0.6%, which is approximately 1.25 times the minimum requirement in both NZS 3101:2006 and ACI 318-14. This low reinforcement ratio was desirable as a low reinforcing ratio can accentuate crack widths and reinforcement strain [33], potentially exacerbating any low-cycle fatigue effects and likelihood of bar fracture. Some minor modifications were made in the detailing of the test specimen transverse reinforcing as compared to the detailing of the beams in the prototype building. These modifications are not expected to have had any effect on specimen response. The longitudinal reinforcement had a development length into the foundation of 475mm and standard 90° hooks as per NZS 3101:2006.

Figure 2-2: Test specimen reinforcement details
### Table 2-2: Design material properties for test specimen

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specified 28-day concrete strength, $f_c'$</td>
<td>30</td>
</tr>
<tr>
<td>Longitudinal reinforcing yield strength, $f_y$</td>
<td>300</td>
</tr>
<tr>
<td>Transverse reinforcing yield strength, $f_{yt}$</td>
<td>500</td>
</tr>
</tbody>
</table>

#### 2.2.2 Specimen construction

Photographs of the construction process are shown in Figure 2-3. Construction of the first fourteen tested specimens took place on-site at a commercial precast concrete yard. The beams were cast monolithically with the foundation.

![Photographs of specimen construction process](image)

**Figure 2-3:** Photographs of specimen construction process, showing (a) reinforcing cage; (b) reinforcing cage set in formwork; (c) poured and finished concrete; and (d) completed specimen with formwork stripped away
Standard precast lifting lugs were cast into each specimen away from the plastic hinge location in order to facilitate transportation to the laboratory. All specimens were inspected after shipment to ensure no cracking occurred during transportation. A high early strength concrete mix was used to allow for the specimens to be moved within 24 hours of casting. Each specimen had a unique concrete batch as they were cast sequentially over a period of several months. For ease of construction, the specimens were cast with the beams lying on their side (i.e. the 720mm-long side was the free surface).

The final three tested specimens were constructed in the University of Auckland Structures Test Laboratory, several months after the last of the other specimens had been cast. Three sets of typical timber formwork were built, and all three specimens were cast simultaneously via a pour from a standard ready-mix concrete truck. These specimens were also cast with the beams lying on their side, in order to match the process used for the other fourteen specimens.

2.3 Test setup

Specimens were tested at the University of Auckland Structures Test Laboratory using the set-up shown in Figure 2-4. The cantilever beams had a shear span ratio $M/V$ of 2.58, based on assuming an inflection point at the mid-span of the beam in the prototype building. Loading was applied using a displacement-controlled 300kN-capacity actuator, which was attached to the specimens using post-tensioned plates. The specimens were tested in a vertical orientation for simplicity of setup, which had no effect on specimen response, as self-weight was negligible and gravity loads were neglected in these tests. The specimens were fixed to the laboratory strong floor using six post-tensioning rods, which passed through corrugated metal ducts cast into the specimen foundations. The foundation-strong floor interface was grouted to ensure a good bond, and each post-tensioning rod was stressed to a force of 300kN prior to testing. Specimens were kept in-plane during testing by a steel frame, providing lateral out-of-plane support at a height of 2.175m above the foundation top. Ultra-high-molecular-weight polyethylene (UHMWPE) strips were glued to both sides of the specimen at the location of the lateral support, creating a low-friction UHMWPE-steel interface. The end columns of the lateral support frame were also used as reference points for instrumentation.

2.3.1 Axial restraint modification

For selected specimens (–ER and –LER specimens listed in Table 2-1), an axial restraint system was employed to approximately simulate the restraint to beam elongation that can be present in moment frame buildings. A drawing of the test setup including the axial restraint modification is shown in Figure 2-5 and photographs both with and without the axial restraint are shown in Figure 2-6. The magnitude of restraint for beams in moment frame buildings is uncertain, as it can include...
contributions from both floor systems and columns interacting with the axial behaviour of the beam plastic hinge [34, 35]. The beam axial force – elongation relationship is typically nonlinear and heavily dependent on the design details of the particular building considered. For the test specimens a simple approach was adopted, whereby an elastic restraint system was used to induce an axial compression force linearly proportional to the beam elongation. The axial restraint system consisted of a spreader beam across the top of the specimen that distributed load to two 26.5mm diameter high-strength post-tensioning rods, one on either side of the specimen. The stiffness of the axial restraint provided by the rods could be varied between tests by addition of Belleville spring washers to the post-tensioning rod-spooler beam connection. Prior to testing, the restraining rods were tensioned to provide an initial axial force of 50kN (≈0.5% \( A_g f'_c \)) in order to avoid the potentially dangerous situation of slack in the setup during dynamic testing. This system was chosen in an effort to bound the level of axial force that would be induced in a beam in a moment frame building, with the unrestrained tests providing a lower bound and the heavily-restrained (ER) tests (without spring washers) providing an estimate of an upper-bound restraint.

Figure 2-4: Schematic of test setup
Figure 2-5: Schematic of test setup with axial restraint system

Figure 2-6: Photographs of test setup both without (left) and with (right) the axial restraint modification
2.3.2 Instrumentation

Specimen load-displacement was measured using the linear variable differential transformer (LVDT) and load cell of the actuator. Inertial forces were negligible in these experiments and therefore the actuator load cell measurements were assumed to be equivalent to the shear force imparted on the specimens. For the specimens with axial restraint, load cells were used to measure the induced force in the restraining bars. An assortment of displacement gauges and string potentiometers, shown in the schematic in Figure 2-7, were employed to measure detailed deformations in the plastic hinge region, including strain in longitudinal reinforcement, beam elongation, and shear deformation, and to verify that foundation rocking or sliding was negligible. The accuracy of the actuator LVDT was verified through a string potentiometer (SP-2.580) mounted at the loading point (2.580m above the foundation top).

Three different instrumentation schemes were used for sensors in the plastic hinge region, as indicated in Figure 2-7. In Layout A and Layout B (Figure 2-7) strain in the reinforcement was captured using displacement gauges attached to threaded inserts that were spot welded [as shown in Figure 2-8(a)] to the longitudinal reinforcement on the west face of the specimens. Plastic tubes were placed around the inserts during pouring of the concrete, as shown in Figure 2-8(b), and were removed after hardening, creating small voids in the cover concrete for access to the threaded inserts [Figure 2-8(c)]. The plastic tubes had a wall thickness of 2mm, which allowed for bond slip without distorting the gauge readings. Three tensile tests each were conducted on both virgin reinforcement samples and those with spot welded inserts, which verified that the spot welding did not alter the reinforcement properties [Figure 2-8(d)].

The displacement gauges for measuring reinforcement strain (N and S instruments in Figure 2-7) were unable to accommodate the sliding shear deformations that developed in the majority of the tests, resulting in a loss of quality strain data after the initiation of sliding shear mechanisms. It is noted, however, that it is not appropriate to use axial strains as a measure of peak bar strain demand when the bar experiences localised strain due to dowel action across a sliding plane. In the specimens where sliding shear deformations did not develop (i.e. -ER specimens), or prior to the onset of sliding in the specimens where such deformations did develop, the data from the N and S instruments are of good quality.

Diagonally-oriented displacement gauges were not used in Layout A, which corresponds to the first five specimens tested (MONO, CYC, P-1, LD-1, LD-1-R). Following this, diagonally-oriented gauges were included in order to better measure the magnitudes of shear deformation and shear sliding. Gauge BS for measuring beam-foundation interface sliding was also added at this time.

The final three tested specimens used instrumentation Layout C (Figure 2-7). Relative to Layout A and Layout B, Layout C used longer gauge lengths for the longitudinal N and S sensors and did not
Experimental program

use the welded insert system shown in Figure 2-8. Displacement gauges were attached to metal rods that were epoxied into holes drilled in the concrete. Additionally, a shorter gauge length for the beam-foundation interface sensors (N1 and S1) was used (25mm versus 108mm), in order to better isolate the strain penetration measurements. The bottom bay of diagonally-oriented sensors was attached to the beam 25mm above the foundation, rather than attached directly to the foundation as was done in Layout A and Layout B. This modified system allowed for comparisons between the different measurement techniques.

Figure 2-7: Schematic and naming convention of instrumentation for the three different instrumentation layouts used
2.4 Material properties

Concrete cylinder compression tests and reinforcing steel tensile tests were conducted to determine as-built material properties for each specimen. The concrete mix design used was high-early strength with 20mm minus aggregate and 60mm slump. Casting and testing of the cylinders was conducted as per the requirements of NZS 3112.2:1986 [36]. All cylinders were placed into a fog room for moist curing from the time immediately after hardening until just before testing. For each specimen, compression testing of three cylinders was conducted within ±3 days of specimen testing. The cylinders for the first specimen (CYC) were fitted with LVDTs during compressive testing in order to obtain the full stress-strain relationship. For these cylinders, an average strain at peak stress of 0.0014 and an average secant stiffness to $0.4f'_c$ of 39,500MPa was observed (Figure 2-9). Only peak strength data were obtained during testing of the cylinders for the remaining specimens. The mean of the three concrete cylinder strengths for each specimen are reported in Table 2-3. In all cases, the measured concrete strength exceeded the specified design strength of 30MPa.
Experimental program

Figure 2-9: Stress-strain curves for compressive testing of three cylinders for specimen CYC

Table 2-3: Measured cylinder strengths (mean of three cylinders)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Age at testing (Days)</th>
<th>$f'_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MONO</td>
<td>97</td>
<td>40.5</td>
</tr>
<tr>
<td>CYC</td>
<td>87</td>
<td>40.9</td>
</tr>
<tr>
<td>CYC-DYN</td>
<td>201</td>
<td>45.8</td>
</tr>
<tr>
<td>P-1</td>
<td>154</td>
<td>45.0</td>
</tr>
<tr>
<td>LD-1</td>
<td>155</td>
<td>43.2</td>
</tr>
<tr>
<td>P-2</td>
<td>97</td>
<td>38.8</td>
</tr>
<tr>
<td>P-2-S</td>
<td>75</td>
<td>39.5</td>
</tr>
<tr>
<td>LD-2</td>
<td>126</td>
<td>44.5</td>
</tr>
<tr>
<td>LD-2-S</td>
<td>41</td>
<td>40.2</td>
</tr>
<tr>
<td>CYC-NOEQ</td>
<td>54</td>
<td>40.3</td>
</tr>
<tr>
<td>LD-1-R</td>
<td>158</td>
<td>44.3</td>
</tr>
<tr>
<td>LD-2-R</td>
<td>181</td>
<td>40.3</td>
</tr>
<tr>
<td>LD-2-ER</td>
<td>195</td>
<td>43.7</td>
</tr>
<tr>
<td>CYC-ER</td>
<td>195</td>
<td>43.0</td>
</tr>
<tr>
<td>LD-2-LER</td>
<td>340</td>
<td>41.2</td>
</tr>
<tr>
<td>CYC-LER</td>
<td>352</td>
<td>45.6</td>
</tr>
<tr>
<td>LD-2-LER-R</td>
<td>263</td>
<td>44.6</td>
</tr>
</tbody>
</table>

Mean±st.dev. for all specimens 41-352 41.5±3.9 (51 cylinders)

Longitudinal reinforcement was Grade 300E, and transverse reinforcement was Grade 500E. Manufacturing requirements for these reinforcing steel grades are stipulated by AS/NZS 4671:2001 [37]. The measured reinforcing steel properties are reported in Table 2-4, and example stress-strain curves are shown in Figure 2-10. The transverse reinforcement did not exhibit a defined yield point and therefore the yield strength and strain at onset of strain hardening ($\varepsilon_{sh}$) is not reported. Reinforcing steel was ordered in three batches, one batch for the first seven casted specimens, one batch for next seven casted specimens, and one batch for the final three casted specimens. For each
batch, three tensile tests were performed on samples of each of the longitudinal and transverse reinforcement; response similar to Figure 2-10 were measured for the steel in all three batches.

![Stress-strain curves](image)

Figure 2-10: Typical stress-strain curves (excluding necking) for longitudinal (a) and transverse (b) reinforcement

<table>
<thead>
<tr>
<th>Relevant specimens</th>
<th>Reinforcing</th>
<th>$f_y$ (MPa)</th>
<th>$\varepsilon_{sh}$</th>
<th>$f_u$ (MPa)</th>
<th>$\varepsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>MONO, CYC, P-1, LD-1,</td>
<td>Longitudinal</td>
<td>301</td>
<td>0.023</td>
<td>444</td>
<td>0.207</td>
</tr>
<tr>
<td>LD-1-R, LD-2-LER, CYC-</td>
<td>Transverse</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>0.113</td>
</tr>
<tr>
<td>LER</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CYC-DYN, P-2, LD-2, LD-2-R</td>
<td>Longitudinal</td>
<td>299</td>
<td>0.022</td>
<td>446</td>
<td>0.214</td>
</tr>
<tr>
<td>LD-2-ER, CYC-ER, LD-2-</td>
<td>Transverse</td>
<td>N/A</td>
<td>N/A</td>
<td>689</td>
<td>0.094</td>
</tr>
<tr>
<td>2-ER-R</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P-2-S, LD-2-S, CYC-</td>
<td>Longitudinal</td>
<td>301</td>
<td>0.023</td>
<td>441</td>
<td>0.230</td>
</tr>
<tr>
<td>NOEQ</td>
<td>Transverse</td>
<td>N/A</td>
<td>N/A</td>
<td>688</td>
<td>0.121</td>
</tr>
<tr>
<td>Mean±st.dev. for all</td>
<td></td>
<td>300±2</td>
<td>0.023±0.001</td>
<td>444±3</td>
<td>0.217±0.017</td>
</tr>
<tr>
<td>specimens (9 coupons)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Where $f_y$ is the yield stress, $\varepsilon_{sh}$ is the strain at the onset of strain hardening, $f_u$ is the ultimate stress, and $\varepsilon_u$ is the strain corresponding to the ultimate stress (uniform strain).

### 2.5 Loading protocols

The objective of investigating the residual capacity of plastic hinges after earthquake loadings necessitated the development of a unique loading protocol. This protocol (implemented in all specimens except MONO, CYC, CYC-DYN, CYC-ER, CYC-LER, and CYC-NOEQ) involved
Experimental program

separating each test into two ‘runs’, with the first run being an initial damaging displacement history representative of what would be expected during earthquake loading, and the second run being a standard quasi-static cyclic loading to failure (Figure 2-11). Cycles with a lower displacement demand than the peak displacement applied in the first run were omitted from the standard cyclic loading of the second run in order to isolate the effects of variations in protocol at or below the peak displacement of the first run. The loading characteristics of the first run were varied between the different specimens in terms of number of cycles, peak displacement demand, and loading rate. The derivation of the initial damaging earthquake displacement histories was based on nonlinear response history analysis of the prototype building, as discussed in Section 2.5.1.

![Baseline cyclic protocol](image1.png)  
![Example of novel loading protocol](image2.png)

**Figure 2-11:** Visual explanation of novel loading protocol, showing the replacement of lower level cycles in the baseline cyclic protocol with an earthquake displacement history (shown in red)

In order to evaluate the effects of loading characteristics relative to a typical loading protocol, it was necessary to conduct a baseline test (specimen CYC) with no initial damaging loading (i.e. a standard quasi-static cyclic test). The complete drift history for this standard cyclic protocol is given in Table 2-5. The protocol was taken as a modified version of that proposed by Park [38]. One initial cycle was applied at each of approximately 0.75 and 1.25 of the yield displacement $\Delta_y$. Cycles were then incremented by a displacement ductility ($\mu$) of 1 starting with $\mu = 2$ until reaching $\mu = 10$, at which point increments by $\mu$ of 2 were employed until failure. Two cycles were applied at each level. The value of the $\Delta_y$ was taken as the same for all tests and was determined during the baseline cyclic test using the method described by Park, where the displacement from an initial load-controlled cycle up to $\frac{3}{4}$ of the calculated yield strength is extrapolated to estimate the yield displacement. This resulted in a value of $\Delta_y = 7.0$mm (0.27% drift). It should be noted that this
method underestimated the actual yield drift of the beams, which can be estimated from the load-displacement plots to be close to 0.4% in all cases.

Table 2-5: Drift history used in baseline cyclic protocol

<table>
<thead>
<tr>
<th>Drift (%)</th>
<th>0.21</th>
<th>0.33</th>
<th>0.54</th>
<th>0.81</th>
<th>1.09</th>
<th>1.36</th>
<th>1.63</th>
<th>1.90</th>
<th>2.17</th>
<th>2.44</th>
<th>2.71</th>
<th>3.26</th>
<th>3.80</th>
<th>4.34</th>
<th>4.88</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. Cycles</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>

Note that not all cycles were applied to all specimens; tests were stopped after significant strength loss occurred and further testing was deemed unsafe.

Selection of the peak drifts imposed in the initial earthquake loadings was based on observations of damage levels at various peak drifts during the baseline cyclic test (CYC). Moderate damage levels were of greatest interest as severely damaged buildings are more likely to be demolished following an earthquake, making assessment of residual capacity less critical. It was therefore decided to select two damage levels, one with what could be considered low-moderate damage (characterised by concrete cracking and yielding of reinforcement), and one corresponding to moderate-heavy damage (characterised by spalling or delamination of cover concrete and significant yielding of reinforcement). Photographs from the baseline cyclic test (Figure 2-12) were used to select values of 1.36% drift for the low-moderate damage level, and 2.17% drift for the moderate-heavy damage level.

Figure 2-12: Photographs from baseline cyclic test (CYC) showing target damage levels of initial earthquake loadings: (a) low-moderate after cycles to 1.36% drift and (b) moderate-heavy after cycles to 2.17% drift

Plots showing the loading protocols by specimen (excluding the monotonic specimen) are shown in Figure 2-13, with the initial earthquake loading shown in red and the standard cyclic loading portion shown in black. It should be noted that the peak drift demand of approximately 4.9% shown in Figure 2-13 was not necessarily reached in all specimens. Instead, tests were stopped after failure
Experimental program

occurred, here defined as a 20% drop in strength. In two specimens (P-2-S and LD-2-S) the initial
damaging earthquake was applied at a quasi-static loading rate (all other earthquake loadings were
applied at the loading rate determined from the nonlinear response history analysis). This was
achieved by increasing the time increment between data points in displacement histories P-2 and
LD-2 by a factor of 25, which resulted in a peak actuator velocity of approximately 0.12% and
0.18% drift per second in specimens P-2-S and LD-2-S, respectively.

A quasi-static monotonic test (MONO) was conducted using a loading protocol that involved
dropping the specimen load to a state of zero force intermittently throughout the test, in order to
observe residual damage states after peak drifts corresponding to the peak drifts of cycles in CYC.
A dynamic cyclic test (CYC-DYN) was conducted using the same loading protocol as that applied
in the baseline static cyclic test (CYC), but with the applied displacement history following a sine
wave pattern with a constant period of 2.5 seconds. A static cyclic test with all cycles up to 2.17%
shift omitted (CYC-NOEQ) (i.e. with the earthquake loading omitted), was also conducted.

2.5.1 Earthquake displacement history derivation

The earthquake loading protocols employed in this experimental program were developed to
provide a realistic loading history that a beam might experience during a seismic event, while also
accounting for variations in cycle content and peak drift demand. In order to develop these loading
histories, non-linear response history analyses were run on a model of the selected prototype
building (shown in Figure 2-1) with an appropriately selected and scaled pair of ground motions.
The ground motion pair consisted of one pulse-type (P) and one long-duration (LD) ground motion
that were response spectrum-matched to each other.

2.5.1.1 Ground motion selection

A database of 486 pulse-type ground motions (an extension of the database presented in [39]), each
of which was response spectrum-matched to a long duration ground motion (using the method
described by Chandramohan et al. [40]), was used as the starting point for the ground motion
selection. The response-spectrum matched pairs were developed by the authors of Chandrahoman
et al. by scaling various long duration records to match the response spectrum of pulse-type records
available in the Next Generation Attenuation project [41]. The definition of what constitutes a
pulse-type or long duration earthquake is discussed in detail in [39] or [40], respectively. This
database was chosen as the differing properties of pulse-type and long duration motions provided a
wide variation in cycle content, while the spectral matching ensured that local variations in spectral
acceleration would not unduly affect building response.
A sequence of criteria were enforced on the database in order to reduce the number of potential ground motion pairs, and ultimately select the best pair for analysis. A modified version of the ground motion selection procedure in NZS 1170.5:2004 [32] was performed to remove pairs that did not match the target spectrum in the period range of interest, with the target spectrum taken as...
the design spectrum of the prototype building. Any pair of motions that had a root mean square difference between the logs $D_1$ greater than 1.3 in the range from 0.4-1.3$T_n$ was discarded. Any pair of motions where one or both required a scaling factor $k_1 < 0.33$ or $k_1 > 3.0$ was also discarded. The $k_1$ factor for the long duration motion included any scaling factor applied by Chandramohan et al. in order to achieve spectral matching with its pulse-type pair. Finally, pairs with a $D_1$ greater than 1.5 over the entire spectrum were removed from consideration.

The above steps reduced the database to 19 pairs of motions. This was further reduced by processing based on the effective number of cycles of the earthquake motion. Displacement histories for each ground motion were obtained based on elastic single degree of freedom (SDOF) analysis at the natural period of the representative building. The equivalent number of cycles at the maximum displacement $N_{eff}$ was then calculated by Equation (2.1).

$$N_{eff} = \frac{1}{2} \sum_{i=1}^{n} \left( \frac{u_i}{u_{max}} \right)^C$$

(2-1)

where $u_i$ is the peak displacement in half cycle $i$, $u_{max}$ is the absolute maximum displacement, $n$ is the number of half cycles, and $C$ is a constant which is used to give higher weighting to larger amplitude cycles, here taken as 2.0.

A low-amplitude cut off which discarded peaks where $u_i/u_{max} < 0.25$ was also used to prevent large numbers of small peaks in the elastic range from affecting the results. Analysis was performed that validated that identifying which ground motions had the largest number of cycles was not particularly sensitive to the chosen natural period or the constant $C$.

Ground motion pairs that had a large discrepancy in the effective number of cycles were desired, and therefore only those pairs that had both an above average $N_{eff}$ for the long duration motion and a below average $N_{eff}$ for the pulse-type motion were considered. This resulted in a final set of 7 potential ground motion pairs. The selection of the specific pair of ground motions for use in analysis was somewhat arbitrary, but was based on selecting the pair with scaling factors closest to unity. Correspondence with the authors of [40] confirmed that this pair was suitable for use and the ground motions did not have any unusual characteristics making them unfit for analysis. The pulse-type ground motion was recorded during the 1981 Westmorland earthquake in California, and the long-duration ground motion was recorded during the 2011 Tohoku earthquake in Japan. Acceleration time histories and response spectra for the chosen ground motion pair are shown in Figure 2-14.

A histogram showing the $N_{eff}$ values for the chosen ground motion pair relative to the range of $N_{eff}$ for all 972 ground motions in the database is shown in Figure 2-15. Approximately 66% of
all ground motions had $N_{eff}$ values between those of the chosen ground motion pair, while 20% had $N_{eff}$ values below the chosen pulse-type motion and 13% had $N_{eff}$ values above the chosen long duration motion.

![Figure 2-14: Acceleration time histories and pseudo-acceleration response spectra (5% damped) of chosen pair of ground motions used in derivation of dynamic loading protocols](image)

2.5.1.2 Building model

A simple non-linear model of the representative building was developed in the finite element modelling software OpenSees [42]. As the building is symmetric, the 3D behaviour of the building
was not modelled, and a 2D model of one of the perimeter moment frames (shown in Figure 2-1) was considered to be sufficiently representative of the entire building response.

Frame elements were modelled using a force-based distributed plasticity method with fiber sections and five integration points per element. An elastic shear spring was included in each element; inelastic shear deformations were not modelled. Rigid offsets at the beam-column joints were applied according to the recommendations in ASCE 41-17. P-∆ effects were captured in all column elements, and a flexible pin-connected leaning column, laterally constrained to the frame at each story, was used to capture the destabilizing effect of the additional gravity loads supported by gravity-resisting frames with limited lateral resistance.

![Pulse-type motions and Long duration motions](image)

**Figure 2-15: Number of effective cycles for all ground motions in database (elastic SDOF analysis)**

Expected material properties were used in all cases. The constitutive model used for the reinforcing steel was a Giuffré-Menegotto-Pinto model [43], as modified by Filippou et al. in [44]. Concrete was modelled using the material developed by Yassin [45], and the properties of confined concrete were calculated as per the Mander et al. model [46]. Bond slip was captured at all joint interfaces using a zero-length fiber section, with the reinforcing material model taken according to the hysteretic rules and material properties proposed by Zhao and Sritharan [47].
Eigenvalue analysis was performed to determine a natural period of 2.1s, which is consistent with that calculated in [31]. Dynamic analysis was performed assuming Rayleigh damping, with a damping ratio of 2.5%. Verification of the model took place by comparing results with an independent study being conducted on the same representative building, but modelled using an alternative lumped-plasticity approach.

2.5.1.3 Extraction of loading protocols

Beam drifts, taken as the average of the rotations of the two end nodes, were extracted from the dynamic analysis results. The beam with the largest peak drift was chosen as the critical member for deriving the earthquake loading protocols, although all beams in the lower stories of the moment frame had relatively similar drift demands.

The drift demands incurred with the ground motions scaled to the target 1/500 year design spectrum were too low for the desired damage levels in the test specimens. Dynamic analysis was therefore repeated, incrementally increasing the ground motion scale factor, until the desired peak drift was obtained in the critical beam. This drift history was then extracted and converted to a displacement history by multiplying by the shear span length of the test specimen. This process was repeated for both the pulse-type and long duration ground motions to obtain a pair of beam displacement histories at each of the two desired damage levels.

2.6 Repair methodology

Specimens LD-1-R, LD-2-R, and LD-2-LER-R were repaired following the initial damaging loading. For every repaired specimen, an identical specimen subjected to the same initial damaging loading was left unrepaired before testing to failure (Figure 2-16). This allowed for the effects of repair to be evaluated relative to the same beam left in its damaged state, which reflects a decision faced by building owners and engineers following earthquakes.

The repair involved epoxy injection of cracks and reconstitution of damaged concrete, and was conducted under the supervision of a specialist contractor with extensive experience in epoxy injection and concrete repair. The same general procedure was followed for all repaired specimens:

- All loose or delaminated concrete was removed by hand or jackhammer.
- Formwork was built in-situ around the specimen, and the concrete substrate was wetted in preparation for bonding to repair mortar.
- A self-compacting, high-early strength repair mortar was poured into a letterbox opening in the formwork.
- Once the repair mortar had set, the formwork was removed and plastic ports for epoxy injection were glued to the specimen along all cracks with a width greater than 0.2mm.
Experimental program

- All cracks connected to those being injected were sealed by covering with an epoxy resin putty.
- Low viscosity epoxy resin was injected into each port by hand pumping with a sealant gun. When the resin could no longer be injected into a given port, that port was sealed and injection continued on the next available port.
- Once the epoxy resin had set, the injection ports and putty were ground off until the beam surface was re-profiled.

Photographs illustrating the repair process are shown in Figure 2-17. The material strengths after 7 days curing at 20°C, as provided by the supplier of the repair materials, are given in Table 2-6. The repaired specimens were all retested between 8-12 days after the date of epoxy injection. The repair mortar was listed as having a compressive strength of approximately 50MPa after 7 days. However, three cylinders of mortar were cast for the final repaired specimen, and were tested for compressive strength after 11 days. The average measured compressive strength of the mortar cylinders was 27.6MPa.

*Figure 2-16: Repair concept*
### Table 2-6: Strengths of repair materials

<table>
<thead>
<tr>
<th>Material</th>
<th>Compressive strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Bond strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Epoxy resin</td>
<td>&gt;70</td>
<td>&gt;55</td>
<td>“greater than normal concrete”</td>
</tr>
<tr>
<td>Repair mortar</td>
<td>~50</td>
<td>Not given</td>
<td>2.0</td>
</tr>
</tbody>
</table>

**Figure 2-17:** Photographs of repair process, showing (a) removal of delaminated concrete; (b) construction of in-situ formwork; (c) installation of injection ports and sealant putty; and (d) injection of epoxy resin

#### 2.7 Test results

Detailed results on the behaviour of all seventeen beam specimens are presented in this section, including hysteretic behaviour, axial elongation, shear deformation, axial force, and damage progression. Comparative analysis between specimens is not included here, as it is given in the relevant sections of Chapters 3 to 6. The data processing methods used in extracting results from the instrumentation described in Section 2.3.2 is given in Appendix A.
2.7.1 Summary of results

Key response characteristics for all specimens in the test program are presented in Table 2-7, including the maximum shear force in each direction during both the static and dynamic parts of the loading history. In all dynamic tests, a spike in shear force occurred at the point of first yield (see Figure 2-19), and in all unrestrained tests the peak strengths of the beams correspond to these spikes. Beam drifts corresponding to various damage states are also provided, including the drifts at cover concrete delamination, bar buckling, and bar rupture (where applicable). The positive and negative drifts corresponding to a 20% drop in lateral strength relative to the maximum static strength are also reported. (For specimen CYC-DYN, a maximum static strength was not available, so the maximum strength not corresponding to the spike at first yield was taken as the benchmark peak strength).

Table 2-7: Summary of key information from experimental program

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_{\text{max}}$ static (kN)</th>
<th>$V_{\text{max}}$ dynamic (kN)</th>
<th>Drift (%) at $V&lt;0.8V_{\text{max}}$ (cycle)</th>
<th>Reason for strength loss</th>
<th>Drift at onset of delam. (%)</th>
<th>Drift at onset of buckling (%)</th>
<th>Drift at bar rupture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MONO</td>
<td>+135</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>CYC</td>
<td>+120 (cycle)</td>
<td>-120</td>
<td>+4.88 (1&lt;sup&gt;st&lt;/sup&gt;)</td>
<td>Bar rupture</td>
<td>2.44%</td>
<td>2.44%</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; cycle to 4.88%</td>
</tr>
<tr>
<td>CYC-DYN</td>
<td>N/A</td>
<td>+122</td>
<td>+3.80 (1&lt;sup&gt;st&lt;/sup&gt;)</td>
<td>Cyclic deg.</td>
<td>1.90%</td>
<td>2.17%</td>
<td>N/A</td>
</tr>
<tr>
<td>P-1</td>
<td>+124 (cycle)</td>
<td>-118</td>
<td>+4.88 (1&lt;sup&gt;st&lt;/sup&gt;)</td>
<td>Bar rupture</td>
<td>2.17%</td>
<td>2.17%</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; cycle to 4.88%</td>
</tr>
<tr>
<td>LD-1</td>
<td>+123 (cycle)</td>
<td>-120</td>
<td>+4.34 (2&lt;sup&gt;nd&lt;/sup&gt;)</td>
<td>Cyclic deg.</td>
<td>2.44%</td>
<td>2.71%</td>
<td>N/A</td>
</tr>
<tr>
<td>P-2</td>
<td>+123 (cycle)</td>
<td>-119</td>
<td>+3.26 (2&lt;sup&gt;nd&lt;/sup&gt;)</td>
<td>Cyclic deg.</td>
<td>2.44%</td>
<td>2.71%</td>
<td>N/A</td>
</tr>
<tr>
<td>P-2-S</td>
<td>+120 (cycle)</td>
<td>-117</td>
<td>+3.80(2&lt;sup&gt;nd&lt;/sup&gt;)</td>
<td>Bar rupture</td>
<td>2.71%</td>
<td>2.71%</td>
<td>2&lt;sup&gt;nd&lt;/sup&gt; cycle to 3.80%</td>
</tr>
<tr>
<td>LD-2</td>
<td>+114 (cycle)</td>
<td>-115</td>
<td>+3.26 (2&lt;sup&gt;nd&lt;/sup&gt;)</td>
<td>Cyclic deg.</td>
<td>2.17%</td>
<td>2.71%</td>
<td>N/A</td>
</tr>
<tr>
<td>LD-2-S</td>
<td>+115 (cycle)</td>
<td>-111</td>
<td>+3.80 (2&lt;sup&gt;nd&lt;/sup&gt;)</td>
<td>Cyclic deg.</td>
<td>2.17%</td>
<td>2.44%</td>
<td>N/A</td>
</tr>
<tr>
<td>CYC-NOEQ</td>
<td>+121 (cycle)</td>
<td>-114</td>
<td>+4.34 (1&lt;sup&gt;st&lt;/sup&gt;)</td>
<td>Cyclic deg.</td>
<td>2.71%</td>
<td>2.71%</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; cycle to 4.88%</td>
</tr>
<tr>
<td>LD-1-R</td>
<td>+125 (cycle)</td>
<td>-122</td>
<td>+4.34 (1&lt;sup&gt;st&lt;/sup&gt;)</td>
<td>Cyclic deg.</td>
<td>2.44%</td>
<td>2.71%</td>
<td>2&lt;sup&gt;nd&lt;/sup&gt; cycle to 4.34%</td>
</tr>
<tr>
<td>LD-2-R</td>
<td>+129 (cycle)</td>
<td>-123</td>
<td>+4.34 (2&lt;sup&gt;nd&lt;/sup&gt;)</td>
<td>Cyclic deg.</td>
<td>2.71%</td>
<td>2.71%</td>
<td>N/A</td>
</tr>
<tr>
<td>LD-2-ER</td>
<td>+168 (cycle)</td>
<td>-155</td>
<td>+4.34 (2&lt;sup&gt;nd&lt;/sup&gt;)</td>
<td>Bar rupture</td>
<td>2.71%</td>
<td>3.26%</td>
<td>2&lt;sup&gt;nd&lt;/sup&gt; cycle to 4.34%</td>
</tr>
<tr>
<td>CYC-ER</td>
<td>+164 (cycle)</td>
<td>-157</td>
<td>+4.34 (1&lt;sup&gt;st&lt;/sup&gt;)</td>
<td>Bar rupture</td>
<td>2.44%</td>
<td>3.26%</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; cycle to 4.34%</td>
</tr>
<tr>
<td>LD-2-LER</td>
<td>+148 (cycle)</td>
<td>-140</td>
<td>+4.34 (2&lt;sup&gt;nd&lt;/sup&gt;)</td>
<td>Bar rupture</td>
<td>2.17%</td>
<td>3.26%</td>
<td>2&lt;sup&gt;nd&lt;/sup&gt; cycle to 4.34%</td>
</tr>
<tr>
<td>CYC-LER</td>
<td>+147 (cycle)</td>
<td>-145</td>
<td>+4.34 (2&lt;sup&gt;nd&lt;/sup&gt;)</td>
<td>Cyclic deg. Bar rupture</td>
<td>2.44%</td>
<td>3.26%</td>
<td>2&lt;sup&gt;nd&lt;/sup&gt; cycle to 4.34%</td>
</tr>
<tr>
<td>LD-2-LER-R</td>
<td>+158 (cycle)</td>
<td>-142</td>
<td>N/A</td>
<td>N/A</td>
<td>2.44%</td>
<td>3.26%</td>
<td>1&lt;sup&gt;st&lt;/sup&gt; cycle to 4.34%</td>
</tr>
</tbody>
</table>
Delamination of cover concrete was chosen as a key damage state over spalling of concrete due to reduced variability in its definition and identification. Delamination was here defined as the point at which wide bond-splitting cracks interlink with wide flexural cracks to form a loose piece of cover concrete. However, onset of delamination and onset of spalling was highly correlated, with spalling quickly occurring within the subsequent cycles.

The definition of drift at onset of bar buckling is here defined as the maximum drift of the cycle in which noticeable curvature of the reinforcing steel is first observed. In all cases, bar buckling was exacerbated in subsequent cycles, but the first noticeable onset was deemed to be the best metric to define this damage state. In some cases, bar buckling may have occurred behind delaminated cover concrete, but as this could not be determined with confidence, the onset of bar buckling was only recorded once the bars were clearly visible. Photographs illustrating examples of both the onset of delamination and onset of bar buckling damage states are shown in Figure 2-18.

![Photographs showing the definitions used in this study for (a) onset of delamination and (b) onset of bar buckling](image)

**Figure 2-18: Photographs showing the definitions used in this study for (a) onset of delamination and (b) onset of bar buckling**

### 2.7.2 Hysteresis

Complete shear force vs. lateral drift plots for all cyclic specimens are shown in Figure 2-19, with the earthquake loading shown in red and the standard cyclic portion shown in black. For the specimens with axial restraint, the shear force value is corrected to remove the contribution of the horizontal component in the restraining bars.
2.7.3 **Axial elongation**

Data from the string potentiometers positioned along the longitudinal axis of the beams were processed to determine the axial elongation of each specimen, which is given for all cyclic specimens in Figure 2-20. Earthquake loading is shown in red and the standard cyclic portion is shown in black. Cycles above 2.5% drift are plotted in a semi-transparent light grey for visual clarity (except in the specimens with axial restraint).

2.7.4 **Shear deformation**

Shear force vs shear deformation plots for all cyclic specimens are shown in Figure 2-21, with the earthquake loading shown in red and the standard cyclic portion shown in black. Shear deformations were calculated as the difference between total lateral displacement and lateral displacement due to flexure. In the first five tested specimens, the shear deformations had to be extracted using string potentiometer data at a reference distance of 1.188m from the beam-foundation interface. For the remaining specimens, the modified displacement gauge layout allowed for these deformations to be calculated at both 0.588m and 1.188m from the beam-foundation interface, although the contribution of shear deformations above 0.588m was negligible in all cases. The plots in Figure 2-21 only include data from cycles up to 2.5% drift, as there was a loss of quality in diagonal displacement gauge data at high drift levels.

2.7.5 **Specimen MONO**

The shear force versus lateral drift, axial elongation, and shear deformation responses of specimen MONO are given in Figure 2-22.

2.7.6 **Axial force**

For the specimens with axial restraint, the compressive force vs elongation relationships are shown in Figure 2-23, with the earthquake loading shown in red and the standard cyclic portion shown in black. The higher level of restraint (ER) was in the range of 40kN/mm, and the lower level of restraint (LER) was approximately 13kN/mm, or 1/3rd of the ER value. The restraint system was not perfectly elastic for the LER specimens, as the spring washers used to reduce the stiffness exhibited some inelastic deformation at higher drift levels.
Figure 2-19: Hysteretic shear force vs. lateral drift plots for all cyclic specimens. (Earthquake loading in red; standard cyclic loading in black)
Figure 2-19 continued
Figure 2-20: Axial elongation vs lateral drift plots for all cyclic specimens. Earthquake loading shown in red; standard cyclic loading shown in black. In all unrestrained cyclic tests, cycles at or above 2.5% drift are shown in light grey for visual clarity.
Figure 2-20 continued
Figure 2.21: Shear force vs shear deformation plots for all cyclic specimens — cycles under 2.5% drift only. (Earthquake loading in red; standard cyclic loading in black)
Figure 2.21 continued
Figure 2-22: Hysteresis, axial elongation, and shear deformation for specimen MONO
Figure 2-23: Axial compression vs axial elongation plots for all restrained specimens. (Earthquake loading in red; standard cyclic loading in black)
2.7.7 Damage progression

Detailed damage data were collected during testing, including photos at each load step, videos of dynamic tests, widths of all non-hairline (greater than 0.2mm) cracks, and general observations. This section presents only a general overview of the damage progression of the specimens. Appendix B includes a detailed description of the damage progression for each specimen, along with a selection of photos at critical points during each test. The complete set of photos, videos, and crack widths are available at DOI 10.17603/DS2SQ2K.

The first sign of observable damage in all specimens was the formation of hairline (<0.2mm) flexural cracks, which occurred when the cracking moment was exceeded. This was followed by the formation of non-hairline flexural cracks, which occurred once the yield point had been reached. As the tests progressed, the number of non-hairline cracks would grow, reaching a peak number (ranging from 4 to 8) during loading cycles in the range of 1.5-2.5% drift. Longitudinal cracks also formed in all specimens, typically first appearing during loading cycles in the range of 1.0-1.5% drift. Specimens that were tested dynamically-then-statically tended to have additional flexural and longitudinal cracks form during the first static loading cycle. An example crack map, showing the typical extent of cracking that developed in the test specimens, is provided in Figure 2-24. Note that many of these cracks were hairline, as all cracks in Figure 2-24 are marked the same regardless of width. Non-hairline cracks only extended up to a maximum of approximately 500mm from the beam-foundation interface in all cyclic specimens (up to 700mm in specimen MONO).

In the twelve unrestrained specimens, sliding shear deformation was a key contributor to the damage progression at moderate-high drift levels (i.e. at drift levels in excess of 1.0-2.0%). Shear sliding was enabled by the formation of wide horizontal cracks through the depth of the member. The width of these cracks was in many cases very large, as multiple non-hairline flexural cracks on the beam ends would converge together into single cracks as they propagated towards the centres of the beams. In all cases, these sliding plane cracks occurred either at the beam-foundation interface or at the location of one of the stirrup sets in the plastic hinge region. In some specimens, multiple sliding plane cracks appeared, while in others only one dominant sliding crack occurred. Photographs showing examples of damage concentration around these major cracks are shown in Figure 2-25; similar damage patterns developed in all of the unrestrained specimens (with the exception of MONO). Table 2-8 lists the locations of the major cracks through the depth of the member for all specimens. Some subjectivity was required in identifying the cracks, as secondary cracks through the depth of the member occurred in some specimens.
Figure 2-24: Example crack map typical of all cyclic tests

RED = crack developed during loading in south direction
BLUE = crack developed during loading in north direction

Figure 2-25: Typical damage states at failure for cyclic unrestrained specimens, showing (a) damage concentrated at one major crack through the depth of the beam (located at the 2nd stirrup), and (b) damage concentrated at two major cracks through the depth of the beam (located at the interface and the 3rd stirrup)
Shear deformations also contributed to buckling of longitudinal reinforcing, by means of dowel action. This meant that the bar buckling was not only a function of compression instability between stirrups, but also the lateral displacements across the sliding plane. Degradation of the core concrete near the beam ends was also prevalent at high drift demands. In many cases, the 135° returns on the transverse reinforcement came unbent following the core concrete degradation, further reducing the restraint to bar buckling. Ultimate failure (20% drop in strength) of the unrestrained specimens was predominantly due to cyclic degradation, with two instances of rupture of buckled bars.

Table 2-9 compares the number of wide cracks through the depth of the beam specimens with their ultimate drift capacity (excluding restrained and repaired specimens). Unrestrained specimens that had only one dominant crack on which shear sliding occurred [e.g. Figure 2-25(a)] tended to exhibit faster strength degradation and lower deformation capacities than specimens which had multiple wide cracks through the depth of the beam [e.g. Figure 2-25(b)]. Specimens with a single dominant crack also tended to have increased pinching of the load-deformation hysteresis (Figure 2-19),
reduced maximum elongation (Figure 2-20), and increased shear deformation (Figure 2-21). Further analysis on the effects of sliding shear is given in Section 4.2.2.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Number of wide cracks through depth of beam</th>
<th>Drift capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CYC</td>
<td>2</td>
<td>4.34</td>
</tr>
<tr>
<td>CYC-DYN</td>
<td>1</td>
<td>3.26</td>
</tr>
<tr>
<td>P-1</td>
<td>2-3</td>
<td>4.88</td>
</tr>
<tr>
<td>LD-1</td>
<td>2</td>
<td>4.34</td>
</tr>
<tr>
<td>P-2</td>
<td>1</td>
<td>3.26</td>
</tr>
<tr>
<td>P-2-S</td>
<td>2(^1)</td>
<td>3.8</td>
</tr>
<tr>
<td>LD-2</td>
<td>1</td>
<td>3.26</td>
</tr>
<tr>
<td>LD-2-S</td>
<td>1</td>
<td>3.8</td>
</tr>
<tr>
<td>CYC-NOEQ</td>
<td>1-2(^2)</td>
<td>4.34</td>
</tr>
</tbody>
</table>

\(^1\) A non-interface sliding crack did not develop until late in the test in specimen P-2-S.

\(^2\) The wide crack through the depth of specimen CYC-NOEQ at the second stirrup was more inclined than in other specimens, and damage therefore did not progress the same as in other single-crack specimens.

In the heavily restrained (–ER) specimens, shear deformations were significantly less impactful than in the unrestrained specimens. Damage progression at higher drift levels was instead dominated by buckling of longitudinal reinforcement with progressively increasing severity with increased drift demand. The buckling in these specimens was entirely due to instability between stirrups rather than dowel action from shear sliding. Degradation of the core concrete near the beam ends was significantly reduced. The lightly restrained (–LER) specimens exhibited damage patterns intermediate between that of the unrestrained and heavily restrained specimens. Ultimate failure of all restrained specimens was predominantly due to rupture of buckled bars, although cyclic degradation did result in a 20% drop in strength in one specimen (CYC-LER). Comparative analysis on the effects different levels of axial restraint had on the damage pattern of the test specimens is further discussed in Section 3.3.3.5.

### 2.8 Summary

An experimental program investigating the seismic behaviour of seventeen nominally identical ductile reinforced concrete beams was described in detail. The information presented in this chapter includes the test setup, the measured material properties, the derivation of a novel earthquake-then-
cyclic loading protocol, and the repair methodology used for the epoxy-injected specimens. Test results presented include the hysteretic load-deformation behaviour, axial elongation, shear deformation, induced axial load, and damage progression. The data from this test program are publicly available at DOI 10.17603/DS2SQ2K. Analysis of the test data is given in the relevant sections of Chapters 3 to 6.
Chapter 3

The influence of loading characteristics and boundary conditions on plastic hinges

This chapter discusses the effects that three factors have on the performance of plastic hinges in reinforced concrete moment frames. The factors considered are the number of loading cycles, the loading rate, and the level of restraint to axial elongation. Background information available in the literature is discussed, followed by a detailed analysis of experimental results from the previously described test program on reinforced concrete beams (see Chapter 2). The implications of these results on existing design or assessment methods and modelling strategies are investigated. The section analysing the effects of the number of loading cycles focusses on the importance of moderate-level cycles, consistent with the objective of this thesis of better understanding the residual capacity of moderately damaged plastic hinges.

3.1 Number of loading cycles

3.1.1 Background

A number of experiments have previously been carried out that investigate the effects of loading history on the performance of reinforced concrete components with ductile seismic detailing [27, 48-54]. These studies have demonstrated both the limited effect of pre-yield cycles and the significant effect of large-displacement cycles on the deformation capacity. However, questions remain about the effects of ‘moderate’, but above yield, displacement cycles, and about the mechanics that lead to cyclic strength degradation in plastic hinges. For the following summary, ‘failure’ is defined using the commonly accepted criteria of a 20% drop in strength after reaching peak lateral load capacity.

El-Bahy et al. [27] tested 6 nominally identical circular columns with a diameter of 305mm, a concrete compressive strength $f'_c$ ranging from 35-40MPa, a total longitudinal reinforcing ratio $\rho$ of 2.0%, and a longitudinal steel yield strength $f_y$ of ~470MPa. The transverse reinforcement consisted of 4mm diameter spirals with a pitch of 19mm or 2 times the longitudinal bar diameter $d_b$, and a yield strength $f_{ye}$ of ~400MPa. The test setup was a cantilever column with a shear span to depth ratio $(M/V)/h$ of 4.5 and an applied axial compression load ratio of $0.1A_gf'_c$. The applied
loading protocols consisted of monotonic, cyclic (three cycles per increment), and low-cycle fatigue tests. The four low-cycle fatigue tests involved fully-reversed constant amplitude cycling at four different drift levels, 2%, 4%, 5.5%, and 7%. The monotonic specimen reached over 11% drift with gradually reducing lateral strength but no catastrophic failure occurring. The specimen subjected to cycling at 2% drift sustained over 150 loading cycles, after which point loading was reverted to a monotonic push, and the specimen was able to reach a drift of 10% before the test was stopped without sudden failure occurring. The specimens subjected to cycling at 4%, 5.5%, and 7% drift failed after the 26th, 10th, and 3rd cycles, respectively. The cyclic test failed at approximately 5.5% drift. The drift at yield was estimated by El-Bahy et al. as 1.5%.

Acun and Sucuoglu [48] tested 6 nominally identical square columns with a section depth of 350mm, a $f'_c$ of ~25MPa, a $\rho$ of 1.0%, and a $f_y$ of ~450MPa. The transverse reinforcement consisted of 8mm diameter stirrups and crossties spaced at 70mm or 5$\overline{d_b}$, with a $f_{yt}$ of ~470MPa. The test setup was a cantilever column with a $(M/V)/h$ of 5.7 and an axial load ratio of 0.2$A_gf'_c$. The applied loading protocols consisted of reversed cyclic tests with varying numbers and sequences of cycles in the range of 0.5-5.2% drift (Figure 3-1). Regardless of the loading protocol, the moment-rotation behaviour of the columns were similar. Limited cyclic strength degradation was observed during cycling at or below 3.5% drift, but cyclic strength degradation at the next load step of 5.2% drift was apparent. None of the columns exhibited a 20% drop in moment capacity until multiple cycles at 5.2% drift had been imposed. The yield point of these columns was visually estimated to be 1.0% drift. Note that the paper referenced in [48] includes additional test specimens not discussed here as they utilized plain (undeformed) longitudinal bars.

Pujol et al. [49] tested two different sets of nominally identical rectangular column assemblies, with three identical specimens in the first set and two in the second set. The only variable changed between the two sets was the stirrup spacing, which was 57mm or 3$\overline{d_b}$ in the first set and 76mm or 4$\overline{d_b}$ in the second set. All other variables were identical. The columns had dimensions of 305x152mm, a $f'_c$ ranging from 27-36MPa, a $\rho$ of 2.4%, and a $f_y$ of ~450MPa. Transverse reinforcement consisted of 6mm stirrups spaced as previously discussed, with a $f_{yt}$ of ~400MPa. The test setup for each specimen consisted of two columns joined by a centre stub. Each column had a $(M/V)/h$ of 2.3 and an axial load ratio of 0.1$A_gf'_c$. The applied loading protocols consisted of low-cycle fatigue cycling to failure at 3% drift, which in some specimens was preceded by an initial cycling at lower drift levels (Figure 3-2). For the three specimens with a stirrup spacing of 3$\overline{d_b}$, one specimen was subjected to an initial loading of seven cycles at 1% drift, another was subjected to an initial loading of seven cycles at 2% drift, while the third had no initial loading prior to cycling to failure at 3% drift. The specimen that was subjected to seven cycles at 2% drift exhibited a 20% drop in peak-to-peak stiffness (i.e. average strength in the two loading directions)
The influence of loading characteristics and boundary conditions on plastic hinges during the 12\textsuperscript{th} cycle at 3\% drift. In the other two specimens a 20\% drop in peak-to-peak stiffness did not occur until the 15\textsuperscript{th} cycle. For the two specimens with stirrup spacing of 4d\textsubscript{b}, one specimen was subjected to an initial loading of seven cycles at 2\% drift, while the other had no initial loading prior to cycling to failure at 3\% drift. The specimen that had previously been cycled at 2\% drift exhibited a 20\% drop in peak-to-peak stiffness during the 5\textsuperscript{th} cycle at 3\% drift, while the specimen with no initial loading did not exhibit such degradation until the 7\textsuperscript{th} cycle. The drift at yield was estimated by Pujol et al. to be 1.0\%. Note that the paper referenced in [49] includes additional tests not discussed here as they did not involve varying loading protocols on nominally identical specimens.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figures/figure3-1.png}
\caption{Loading protocols applied to six nominally identical specimens in the Acun and Sucuoglu [48] experimental program}
\end{figure}

Nojavan [50] tested five nominally identical large-scale rectangular columns with dimensions of 914x711mm, a f'\textsubscript{c} ranging from 31-37MPa, a \(\rho\) of 1.6\%, and a f\textsubscript{y} of \approx 500MPa. The transverse reinforcement consisted of 15.9mm diameter stirrup sets spaced at 127mm or 4.44d\textsubscript{b}, with a f\textsubscript{y,t} of \approx 450MPa. The test setup was a cantilever column with a (M/V)/h of 2.7 and an axial load ratio of 0.15A\textsubscript{g} f'\textsubscript{c}. The first loading protocol applied consisted of a single cycle to nearly 12\% drift, which
The influence of loading characteristics and boundary conditions on plastic hinges

represented the stroke limit of the actuator. This specimen exhibited a stable lateral load-carrying capacity, although the lateral strength did gradually reduce with drift. The second loading protocol consisted of a typical cyclic test, with three-cycle sets at incrementally increasing displacement demands until failure, which occurred during the cycles to 5.7% drift. The third loading protocol consisted of the same progressively increasing cyclic protocol as the previous specimen, but only including those cycles up to 4.6% drift, after which a large displacement cycle matching that of the first loading protocol was imposed. This specimen had already exhibited significant strength loss in the negative loading direction during the cycles to 4.6% drift, but was able to maintain stable levels of strength in the positive direction during the large displacement cycle to nearly 12% drift.

The fourth loading protocol was similar to the third one, except the cyclic test was stopped after the cycles to 3.7% drift. In this specimen, the lateral load carrying capacity was stable until nearly 10% drift in both directions of the large displacement cycle. The loading protocol for the fifth specimen was somewhat distinct from the others and was designed to represent a ratcheting-style loading, which was again followed by a large displacement cycle. Once again, the performance in the large displacement cycle was very similar to the specimen which had no initial loading prior to being subjected to a large displacement cycle. The drift at yield was determined by Nojavan to be ~0.8%.

Note that additional specimens in this test program are not discussed here as they did not involve varying loading protocols on nominally identical specimens.

Xing et al. [51] tested eleven nominally identical rectangular columns, with dimensions of 250x200mm, a $f'_{c}$ ranging from 24-35MPa, a $\rho$ of 2.5%, and a $f_y$ ranging from 375-411MPa. The transverse reinforcement consisted of 8mm diameter stirrup sets spaced at 50mm or $2.5d_b$, with $f_{yt}$ ranging from 305-328MPa. The test setup for each specimen consisted of two columns joined by a centre stub. Each column had a $(M/V)/h$ of 4.4 and an axial load ratio of 0.2$A_gf'_{c}$. The applied loading protocols consisted of one monotonic test, six constant amplitude low-cycle fatigue tests at various displacements benchmarked against the yield displacement (1.5$\Delta_y$, 2$\Delta_y$, 2.25$\Delta_y$, 2.5$\Delta_y$, 3$\Delta_y$, and 4$\Delta_y$), and four variable amplitude low-cycle fatigue tests that involved cycling at two discrete drift levels. The monotonic test had a stable lateral load-carrying capacity until the test was stopped at 10.3% drift, although the lateral strength did gradually reduce with drift. The constant amplitude low-cycle fatigue tests were able to withstand 1048, 268, 135, 13, 7.5, and 2 cycles at approximately 1.9%, 2.6%, 2.4%, 3.1%, 3.8%, and 5.2% drift, respectively. Higher concrete strength in the specimen cycled at 2.25$\Delta_y$ caused a reduced yield displacement relative to the specimen cycled at 2$\Delta_y$, meaning that despite having a higher target displacement ductility, it had a reduced drift demand. Results from the four variable amplitude low-cycle fatigue tests are difficult to interpret given the limited information provided in the paper. However, they do show that the application of 193 cycles at 2$\Delta_y$ (2.3% drift) did not greatly reduce the number of cycles to failure
The influence of loading characteristics and boundary conditions on plastic hinges

when the same specimen was subsequently cycled at $3\Delta_y$ (6.5 cycles to failure, comparable to the 7.5 of the previously discussed specimen subjected to constant amplitude cycling at $3\Delta_y$ only).

It should be noted that these tests all involved columns with axial load ratios equal to or greater than $0.1A_gf'_c$. These findings are therefore not directly applicable to beams, where the number of loading cycles can cause an increase in axial elongation, which may in turn affect the degradation behaviour [55]. Beams generally also have a lower curvature at yield than columns, meaning that for a given drift demand, beams have increased ductility demands as compared with columns. Furthermore, for modern buildings designed to achieve a strong-column-weak-beam mechanism, plastic hinging is expected to be more significant in beams.

Additional test programs by Ou et al. [52] and Goodnight et al. [53] (columns), and Ingham et al. [54] (beams) have also investigated the effect of load history on the performance of nominally identical ductile reinforced concrete members. The loading protocols employed in these studies make it difficult to isolate the effects of moderate or low level cycles, and so they are not discussed.
here in detail. However, these test programs do consistently show that increasing the cycle content at all loading levels has a detrimental effect on deformation capacity.

3.1.2 Experimental results

This section discusses results from the ten unrepaired and unrestrained beam specimens, as previously described in Chapter 2. The focus area is the effect of the number of moderate-level (up to 2.17% drift) loading cycles.

3.1.2.1 Overview of critical cyclic and monotonic specimens

The effect the number of loading cycles had on the hysteretic behaviour can be clearly identified by observing the difference between the quasi-static cyclic test (CYC), the quasi-static monotonic test (MONO), and the quasi-static cyclic test that did not have cycles applied at or below 2.17% drift (CYC-NOEQ). Shear force versus lateral drift results from specimens CYC and MONO are compared in Figure 3-3. (It should be noted that the monotonic test was periodically dropped to states of zero force throughout the test to measure residual crack widths, but this is not included in Figure 3-3 for visual clarity.) Also shown in Figure 3-3 is the cyclic envelope (representing only the peak of cycles) of specimen CYC-NOEQ. The importance of cyclic loading on the ultimate behaviour is readily apparent. Specimen CYC began to exhibit cyclic strength degradation relative to specimen MONO at cycles upwards of 2.5% drift, with a 20% drop in strength first occurring in the second cycle to 4.34% drift. The monotonic test was able to sustain its lateral load carrying capacity at upwards of 15% drift, at which point the actuator reached its stroke limit and the test had to be stopped. Similar discrepancies between monotonic and cyclic tests have been obtained for reinforced concrete columns in other test programs [27, 50, 51], as previously discussed. However, the rate of cyclic strength degradation in specimens CYC and CYC-NOEQ was similar despite the differences in loading protocol. This limited effect of the number of loading cycles below 2.17% drift is further investigated in the following sections by considering the other test specimens.

3.1.2.2 Quantifying the variation in the number of moderate-level loading cycles

The nine unrestrained cyclic specimens provide information about the effect of variations in load history at moderate drift demands. All of these specimens were subjected to the same loading protocol at cycles above 2.17% drift, and it is therefore only necessary to quantify the differences in the number of cycles applied at or below 2.17% drift. To this end, two different metrics are used: (i) dissipated energy; and (ii) effective number of cycles at peak drift.

In is noted that variations in the cycle content at or below 1.36% were also considered in the experimental program, as that was the peak drift applied in the initial earthquake loadings P-1 and
The influence of loading characteristics and boundary conditions on plastic hinges

LD-1; however, minimal discussion regarding this is provided. This omission is justified due to the forthcoming conclusion that the cycle content variations up to 2.17% drift were of limited significance. It is assumed that the same conclusions can be expected to hold for any lesser drift demands.

![Hysteresis comparison for specimens CYC, MONO, and CYC-NOEQ](image)

The dissipated energy due to cycles up to 2.17% drift is shown in Table 3-1. Dissipated energy was determined from the specimen load-displacement response, and is therefore partially dependent on variations in the specimen behaviour (e.g. degree of pinching), rather than being solely a function of differences in the loading protocol. Nonetheless, it serves as an indirect representation of the cycle content.

Calculating the number of effective cycles at the peak drift \(N_{eff}\) avoids the response-dependent problem with the energy dissipation metric; however, quantification is a difficult endeavour due to the large number of ways in which cycles can be defined [56]. In this study, a displacement range counting methodology was used to determine the effective number of cycles at the peak applied drift. Ranges were defined as the peak-to-trough distances in the displacement histories, but those less than 14mm (2x the estimated value of the yield displacement) were omitted from the range identification algorithm to prevent large numbers of cycles in the elastic range from unduly affecting results. An example of the results from the range identification algorithm compared with the virgin displacement history for loading protocol LD-2 is shown in Figure 3-4. Once the list of displacement ranges was identified, \(N_{eff}\) was calculated using Equation (3-1).

**Figure 3-3: Hysteresis comparison for specimens CYC, MONO, and CYC-NOEQ**
The influence of loading characteristics and boundary conditions on plastic hinges

\[ N_{eff} = \frac{1}{2} \sum_{i=1}^{n} \left( \frac{u_{r,i}}{u_{r,max}} \right)^C \]

where \( u_{r,i} \) is the peak-to-trough displacement in range \( i \), \( u_{r,max} \) is taken as \( 2 \times \) peak drift (the peak-to-trough displacement range for a fully reversed cycle at the peak drift), \( n \) is the number of displacement ranges, and \( C \) is a constant which is used to give higher weighting to larger amplitude cycles.

Equation (3-1) was derived by Malhotra [57] as a combination of the Coffin-Manson low-cycle fatigue law [58, 59] and Miner’s summation [60]. The value of \( N_{eff} \) is highly sensitive to the choice of the parameter \( C \), which serves to give higher importance to larger displacement cycles. Given that \( C \) is an unknown value, it was decided to present the number of cycles for a range of possible \( C \) values, as shown in Figure 3-5. The \( N_{eff} \) values shown in Table 3-1 and used in the following discussion are based on \( C = 2 \) (a typical value recommended by Malhotra); however, similar results hold for \( C = 1 \) to 3.

Malhotra [57] also showed that the \( N_{eff} \) experienced by a structure is dependent on the natural period \( T_n \) and the ground motion characteristics. Malhotra conducted analysis on a suite of 71 ground motions and found that, for \( C = 2 \) and \( T_n = 2.0 \text{s} \) (the period of the prototype structure used to derive the displacement histories used in this study), \( N_{eff} \) ranged from approximately 1 to 8. As shown in Table 3-1, a comparable range of \( N_{eff} \) values (0.5 to 7.0) were applied to the test specimens. However, it should be noted that for shorter period structures with \( T_n < 0.5 \text{s} \), Malhotra found that certain ground motions resulted in \( N_{eff} \) values as high as 20.
The influence of loading characteristics and boundary conditions on plastic hinges

Figure 3-5: Number of effective cycles at 2.17% drift for the various loading protocols

Table 3-1: Variations in energy dissipation and $N_{\text{eff}}$ due to the different applied loading protocols at or below 2.17% drift

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Energy dissipation – cycles up to 2.17% drift (kNm)</th>
<th>$N_{\text{eff}}$ (for $C = 2$) – cycles up to 2.17% drift</th>
</tr>
</thead>
<tbody>
<tr>
<td>CYC</td>
<td>97.2</td>
<td>5.6</td>
</tr>
<tr>
<td>CYC-DYN</td>
<td>92.4</td>
<td>5.6</td>
</tr>
<tr>
<td>P-1</td>
<td>73.1</td>
<td>4.1</td>
</tr>
<tr>
<td>LD-1</td>
<td>114.8</td>
<td>7.0</td>
</tr>
<tr>
<td>P-2</td>
<td>12.4</td>
<td>0.5</td>
</tr>
<tr>
<td>P-2-S</td>
<td>10.9</td>
<td>0.5</td>
</tr>
<tr>
<td>LD-2</td>
<td>81.5</td>
<td>5.4</td>
</tr>
<tr>
<td>LD-2-S</td>
<td>67.4</td>
<td>5.4</td>
</tr>
<tr>
<td>CYC-NOEQ</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

3.1.2.3 Effect of moderate-level loading cycles on deformation capacity

Considering either dissipated energy or effective cycles at peak drift, the effect of variations in the cycle content at or below 2.17% drift on the deformation capacity of the specimens is negligible. Figure 3-6(a) illustrates the lack of correlation between either dissipated energy or $N_{\text{eff}}$ and drift capacity. Instead, the drift at failure tended to be related to the damage pattern that developed, with concentrated damage negatively affecting deformation capacity (see Section 2.7.7 for more information). The concentration of damage that developed due to cycles at or below 2.17% drift was independent of the load history. For example, Figure 3-6(b) shows relatively well-distributed damage in specimens P-1 and LD-1 and relatively concentrated damage in specimen P-2, despite
P-1 and LD-1 having been subjected to a larger number of loading cycles up to that point. Measurable cracks (wider than 0.2mm) were distributed over a length of 450-550mm from the beam-foundation interface in P-1 and LD-1, but only extended to less than 300mm in specimen P-2.

There was also a lack of correlation between the cycle content at or below 2.17% drift and the residual energy dissipation capacity, which is here defined as the dissipated energy due to all cycles above 2.17% drift until a 20% drop in strength was reached. Cumulative energy dissipation curves for all nine specimens considered here are shown in Figure 3-7. This plot illustrates how the spread in dissipated energy due to variations in the lower-level cycle content is carried through until failure. Specimens with less energy dissipation prior to 2.17% drift did not have increased energy dissipation capacity from that point onward.

Figure 3-6: (a) Comparison of cycle content metrics and drift capacity, and (b) photographs of damage state of selected specimens after all cycles at 2.17% drift.
The influence of loading characteristics and boundary conditions on plastic hinges

Figure 3-7: Cumulative energy dissipation comparison

The limited effect of the number of moderate level loading cycles on deformation capacity is consistent with past testing programs on reinforced concrete columns [27, 48-51]. This does not imply that these cycles do not have any deleterious effects, as evidenced by the modest but appreciable negative impact on deformation capacity of cycling at 2% drift in the Pujol et al. tests [49]. Instead, the data here suggests that the deleterious effects of moderate-level loading cycles are of limited importance when considering the number of cycles likely to occur in an earthquake. These findings conflict with the concept of a reference energy dissipation capacity, or energy-based hysteretic degradation rules, and support the current practice of assessing structural performance based on peak drift demands. A drift limit of 2.5% is commonly imposed in modern building codes, and the tests discussed here indicate limited importance of the number of cycles below that drift limit. In the context of post-earthquake damage assessment of reinforced concrete beams, these results indicate that consideration of the number of loading cycles applied during the damaging earthquake may be less important in situations where inter-story drifts were in the range of 2% or below.

It must be highlighted that these conclusions only hold for moderate level damage. Consideration of the number of large-drift cycles can be essential for collapse analysis, as evidenced by other experimental programs [52-54]. Based on the results of this and previous studies, it is the author’s opinion that the number of loading cycles prior to damage localization is not a major contributing factor to the deformation capacity of flexural members. Damage localization is dependent on the failure mode of the component in question, and could take the form of bar buckling, degradation of core concrete, or yielding of transverse reinforcement. In the beam specimens discussed in this section, although localization of damage at sliding plane cracks did occur prior to reaching 2.17%
drift, all specimens except CYC-DYN exhibited a load-deformation envelope with a positive slope until beyond 2.17% drift. This implies that any damage or degradation that may have occurred due to cycles below 2.17% drift was not severe enough to cause cyclic strength degradation.

3.1.2.4 Effect of moderate-level loading cycles on elongation

Figure 3-8 shows the beam elongation versus lateral drift relationships (due to cycles at or below 2.17% drift) for four specimens (CYC, MONO, P-2, and LD-2) with wide-ranging cycle contents. It is evident that the specimens with increased cycle contents also had increased magnitudes of beam elongation. In Figure 3-9(a-b), the cycle content metrics of Table 3-1 are compared against the magnitude of elongation at 2.17% drift, with dissipated energy providing a closer correlation to elongation than \( N_{eff} \). The correlation between the number of loading cycles and the magnitude of elongation in reinforced concrete beams has previously been identified in the literature [61]. As seen in Figure 3-9(c-d), the differences in elongation at 2.17% drift did not affect the maximum elongation that occurred later in the tests, after reverting to identical cyclic loadings. The elongation after all cycles to 2.7% drift, which was close to the maximum elongation in all cases and therefore taken as a proxy metric, was independent of the varied loading protocols at 2.17% drift and below.

![Figure 3-8: Beam elongation versus lateral drift (due to cycles at or below 2.17%) for selected specimens](image)

Similar to the energy dissipation and \( N_{eff} \) metrics, no correlation was found between the magnitude of elongation at 2.17% drift and the ultimate deformation capacity of the beams, as shown in Figure 3-10(a). However, the magnitude of elongation at 2.7% drift was correlated with the ultimate deformation capacity [Figure 3-10(b)]. This was because the variance in elongation at 2.17% drift was largely governed by the different applied loading protocols, while the specimens with lower elongations at 2.7% drift had developed mechanisms such as shear sliding or longitudinal bar buckling that both limited further elongation and caused cyclic strength degradation.
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Figure 3-9: Elongation at 2.17% drift versus (a) dissipated energy and (b) $N_{ef}$, and elongation at 2.7% drift versus (c) dissipated energy due to cycles at or below 2.17% drift, and (d) $N_{ef}$ due to cycles at or below 2.17% drift.

Figure 3-10: Elongation (after all cycles to (a) 2.17% drift and (b) 2.7% drift) versus drift capacity
3.1.3 Comparison of test results with cumulative damage models

Explicitly modelling the effects of load history on the behaviour of reinforced concrete plastic hinges is an area of active research. A simplified analysis method that is commonly used in earthquake engineering is to use models that incorporate cumulative measures such as energy dissipation in order to assess the impacts of load history. These models require calibration against test data to be useful.

In the nominally identical beam specimens of this test program, the rates of strength degradation and deformation capacities were found to be variable, yet independent of the variations in load history that were applied at or below 2.17% drift. In this section, cumulative damage models are calibrated against the test specimens in an effort to quantify the impact of (i) the variation in loading protocol at or below 2.17% drift, and (ii) the dispersion in the rate of cyclic degradation. Any effect of loading rate is neglected in this work, which is justified based on the inconclusive results of Section 3.2 and the fact that loading rate is typically neglected when modelling the seismic response of reinforced concrete structures. The specific models used in the calibration are (i) the Park and Ang damage index [62], and (ii) the Ibarra-Medina-Krawinkler degrading hysteretic model [63].

The fact that this dataset includes both monotonic and cyclic experiments makes it particularly appropriate for use with these models, which require an analytical estimate of the monotonic behaviour when only cyclic test data are available. It should be noted that there are many other proposed damage indices and hysteretic models that are based on similar principles. These specific models were chosen due to their widespread use in earthquake engineering research.

3.1.3.1 Park and Ang damage index

The Park and Ang damage index consists of a linear combination of the maximum deformation relative to the monotonic deformation capacity and the dissipated energy relative to the monotonic energy dissipation capacity, as shown in Equation (3-2). The point of failure (i.e. a 20% drop in lateral strength) is taken as the point when the index ‘D’ reaches a value of 1.0.

\[
D = \frac{\delta_M}{\delta_u} + \frac{\beta}{Q_y \delta_u} \int dE
\]  

(3-2)

where \( \delta_M \) is the maximum deformation, \( \delta_u \) is the deformation capacity under monotonic loading, \( Q_y \) is the yield strength, \( dE \) is the incremental energy dissipation, and \( \beta \) is a parameter that governs the rate at which the damage index increases with cyclic loading.

For each test specimen, the \( \beta \) parameter was calibrated to give an index value of \( D = 1.0 \) at the end of the half-cycle in which failure occurred. The complete list of \( \beta \) values is shown in Table 3-2. The monotonic deformation capacity \( \delta_u \) was taken as the maximum deformation reached by specimen MONO (therefore assuming failure was imminent when the test was stopped) and the
yield strength $Q_y$ was taken as the nominal strength $M_n/L$. The calibrated values of $\beta$ ranged from 0.10-0.31, with an average of 0.18 and a coefficient of variation of 0.38. Also shown in Table 3-2 are the values of $D$ at failure calculated using the average $\beta$ value of all specimens ($\beta_\mu$) and the $\beta$ values corresponding to $\pm$ one standard deviation ($\sigma$) from the average. The values of $D$ at failure ranged from 0.66-1.52 using $\beta_\mu$, and from 0.49-1.99 when considering $\beta_\mu \pm \sigma$.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Calibrated $\beta$ factor</th>
<th>$D$ at failure ($\beta = \beta_\mu$)</th>
<th>$D$ at failure ($\beta = \beta_{\mu+\sigma}$)</th>
<th>$D$ at failure ($\beta = \beta_{\mu-\sigma}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CYC</td>
<td>0.11</td>
<td>1.42</td>
<td>0.99</td>
<td>1.85</td>
</tr>
<tr>
<td>CYC-DYN</td>
<td>0.19</td>
<td>0.95</td>
<td>0.67</td>
<td>1.24</td>
</tr>
<tr>
<td>P-1</td>
<td>0.11</td>
<td>1.45</td>
<td>1.02</td>
<td>1.88</td>
</tr>
<tr>
<td>LD-1</td>
<td>0.10</td>
<td>1.52</td>
<td>1.05</td>
<td>1.99</td>
</tr>
<tr>
<td>P-2</td>
<td>0.31</td>
<td>0.66</td>
<td>0.49</td>
<td>0.83</td>
</tr>
<tr>
<td>P-2-S</td>
<td>0.23</td>
<td>0.83</td>
<td>0.61</td>
<td>1.05</td>
</tr>
<tr>
<td>LD-2</td>
<td>0.19</td>
<td>0.93</td>
<td>0.66</td>
<td>1.21</td>
</tr>
<tr>
<td>LD-2-S</td>
<td>0.18</td>
<td>0.99</td>
<td>0.71</td>
<td>1.28</td>
</tr>
<tr>
<td>CYC-NOEQ</td>
<td>0.18</td>
<td>0.98</td>
<td>0.71</td>
<td>1.24</td>
</tr>
<tr>
<td>Average</td>
<td>0.18</td>
<td>1.08</td>
<td>0.77</td>
<td>1.40</td>
</tr>
<tr>
<td>St. dev.</td>
<td>0.07</td>
<td>0.30</td>
<td>0.20</td>
<td>0.41</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>0.38</td>
<td>0.28</td>
<td>0.26</td>
<td>0.29</td>
</tr>
</tbody>
</table>

The coefficient of variation obtained for $D$ at failure using the calibrated $\beta_\mu$ was 0.28. Comparatively, Park and Ang [62], who calibrated their equation for $D$ against a database of 261 reinforced concrete beams and columns, found a coefficient of variation for $D$ at failure of approximately 0.50. Therefore, only a modest increase in the predictive ability of the Park and Ang index was obtained when calibrated against the nominally identical reinforced concrete beams of this test program.

The degree to which the dissipated energy due to variations in load history at or below 2.17% drift affected the calibrated $\beta$ values is shown in Figure 3-11. Specimens with more dissipated energy due to these lower level cycles tended to have lower $\beta$ values, indicating a slower rate of cyclic deterioration.
3.1.3.2  Ibarra-Medina-Krawinkler hysteretic model

The Ibarra-Medina-Krawinkler hysteretic model uses a parameter $\beta$ [Equation (3-3)], which is dependent on the cumulative energy dissipation relative to a ‘reference energy dissipation capacity’, to alter any of four possible degradation modes, namely cyclic strength, in-cycle strength, reloading stiffness, and unloading stiffness. The reference energy dissipation capacity is dependent on the parameter $\gamma$, as shown in Equation (3-4), and the method of calculating the degree of cyclic degradation for each mode is given by Equation (3-5). For a complete description of the model, see Ibarra et al. [63] and the modifications proposed by Lignos [64].

$$\beta_i = \left( \frac{E_i}{E_t - \sum_{j=1}^{i} E_j} \right)^c$$  \hspace{1cm} (3-3)

$$E_t = \gamma M_y$$  \hspace{1cm} (3-4)

$$F_i = (1 - \beta_i) F_{i-1}$$  \hspace{1cm} (3-5)

where $E$ is the dissipated energy, $E_t$ is the reference energy dissipation capacity, $M_y$ is the yield moment, $F$ refers to various parameters relating to any of the four degradation modes, $\beta$, $\gamma$, and $c$ are non-dimensional parameters, and $i$ and $j$ are integers representing cycle indices.

Two previous studies have extensively calibrated the Ibarra model against databases of reinforced concrete test specimens. Haselton et al. [65] calibrated the model against the PEER reinforced concrete column database [66] and Lignos [64] calibrated the model against a database of 200 reinforced concrete beam specimens. Both of these studies employed simplifications to the deterioration modelling, setting $c = 1$ and taking $\gamma$ to be the same for all degradation modes that were accounted for (with the exception of the unloading stiffness degradation, which was permitted...
to be unique from the other modes in the Lignos study). Peak-oriented reloading stiffness rules were employed in these studies, and therefore little emphasis was placed on accurately modelling any ‘pinching’ behaviour. These simplifications are also adopted in the analysis conducted here. Slightly different definitions were used for the reference energy dissipation capacity $E_t$ in the Haselton et al. and Lignos studies. The definition shown in Equation (3-4) is used in this work, due to this being the method currently implemented in the finite element analysis program OpenSees [42], which was used as a platform to conduct the analysis.

An example of the calibrated model behaviour versus experimental load-deformation response is shown in Figure 3-12 for specimen CYC, and the complete set of calibrated results for the other eight specimens is shown in Figure 3-13. Calibration was based on minimizing the residual sum of squares between the experimental and modelled maximum force during loading cycles above 2.17% drift (i.e. where identical loading protocols were applied to all specimens), as calculated by Equation (3-6). The calibration focussed on matching the cyclic degradation behaviour only. Any in-cycle degradation was not attempted to be modelled, and therefore the final cycle was neglected in specimens that exhibited longitudinal bar rupture (the only source of in-cycle degradation in this test program). Given that in-cycle degradation was ignored, it was unnecessary to calibrate a capping point (i.e. the rotation at which negative stiffness begins during monotonic loading). This is consistent with the behaviour exhibited by the monotonically-loaded specimen (MONO), which reached a drift of 15.9% without exhibiting a negative stiffness.

$$\text{error} = \sqrt{\sum_{i=1}^{n} (F_{\text{exp},i} - F_{\text{model},i})^2} \quad (3-6)$$

where $i$ is the cycle index, $n$ is the number of cycles, $F_{\text{exp},i}$ is the experimental maximum force in cycle $i$, and $F_{\text{model},i}$ is the modelled maximum force in cycle $i$.

Using a linearized post-yield stiffness calibrated against the data from specimen MONO significantly underestimated the actual post-yield strain hardening behaviour of the cyclic specimens, possibly due to increased axial elongation in the cyclic specimens resulting in higher levels of strain hardening. A strain hardening ratio of 0.04 of the secant stiffness to yield was employed to best match the cyclic test data. The strain hardening ratio, secant stiffness to yield, and yield moment were taken as the same in all specimens. Only the parameter $\gamma$, which governs the rate of cyclic degradation, was calibrated between the different specimens.

The calibrated $\gamma$ factors for the nine test specimens ranged from 0.55-1.65, with an average of 1.09 and a coefficient of variation of 0.37. Figure 3-14 shows the effect of the $\gamma$ factor variation on the behaviour of the model when subjected to the standard cyclic protocol of specimen CYC.
Haselton et al. [65] and Lignos [64] both assumed the $\gamma$ parameter to be lognormally distributed. In Figure 3-15, the calibrated $\gamma$ values from the experimental data are fitted to a lognormal distribution to facilitate comparison with these studies. The lognormal standard deviation $\sigma_{ln}$ of the fitted curve was 0.38. Haselton et al. used the PEER database [66] to derive an equation for the prediction of $\gamma$, with the error quantified as a $\sigma_{ln}$ of 0.50 (with outliers removed). Lignos calculated a $\sigma_{ln}$ of 0.77 using data from 200 reinforced concrete beam tests (with no effort to account for variation in parameters between specimens). The reduction in $\sigma_{ln}$ observed in this test program relative to the Haselton et al. and Lignos studies is perhaps less than would be expected, given that the calibration was against identical reinforced concrete beams instead of large databases.

It is noted that even more variability would have been observed had the in-cycle degradation that occurred in four specimens (CYC, CYC-NOEQ, P-1, P-2-S) been modelled. However, this would have required calibration of different values for the capping rotation, post-capping rotation, and post-capping negative stiffness for each of these specimens. This could not be reconciled with the true monotonic behaviour, given that all specimens were nominally identical. This, as well as the increased post-yield stiffness of the cyclic specimens relative to the monotonic specimen, indicates that the true monotonic and cyclic behaviours of these beams cannot readily be incorporated into the same Ibarra-Medina-Krawinkler model.

The degree to which the dissipated energy due to variations in load history at or below 2.17% drift affected the calibrated $\gamma$ values is shown in Figure 3-11. Similar to the finding for the $\beta$-factor of the Park and Ang index, specimens with more dissipated energy due to lower level cycles tended to have higher $\gamma$ values, indicating a slower rate of cyclic deterioration.
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Figure 3-13: Calibration of Ibarra-Medina-Krawinkler model against other test specimens
The influence of loading characteristics and boundary conditions on plastic hinges

Figure 3-14: Effect of variation in $\gamma$ parameter on model behaviour (loading protocol is that of specimen CYC)

Figure 3-15: Fitted lognormal distribution of $\gamma$

3.1.3.3 Implications for numerical studies on the impact of load history

A number of previous studies have used non-linear response history analysis to evaluate the effects of multiple earthquakes or earthquake duration on the seismic performance of ductile reinforced concrete moment frames [67-74]. These studies used a variety of distributed plasticity and lumped plasticity modelling techniques, both with and without degrading properties. One of the most commonly used methods for modelling plastic hinges was the Ibarra-Medina–Krawinkler model, which was employed in [68, 69, 72, 74]. The effects of multiple earthquakes or earthquake duration were quantified using a variety of metrics, including damage index values (the Park and Ang index
being among those used), deformation demands, or collapse fragilities. All studies reported some degree of negative impact when considering longer ground motions or earthquake sequences, although the reported importance varied considerably.

The conclusions of these studies were necessarily affected by the modelling parameters and output metrics that were used. The results presented here emphasise the importance of considering modelling uncertainties (e.g. as described by Liel et al. [75]) when conducting analysis that attempts to quantify the degradation of ductile reinforced concrete buildings as a function of the load history. It was shown that, even when near-ideal data are available for calibration purposes, a non-negligible portion of the variability in the cyclic degradation of reinforced concrete beams may not be predictable by these models.

It was also demonstrated that the calibrated parameters that govern the rate of cyclic deterioration can be partially dependent on the load history of the specimens they are calibrated against. Specimens with more lower-level loading cycles are more likely to result in calibrated deterioration parameters that indicate slower cyclic deterioration. For cases where the only available data consists of standard cyclic experiments with many lower-level loading cycles, this may result in non-conservative values.

It is noted that introducing non-linearity into the importance of energy dissipation on cyclic deterioration could reduce the effect of lower-level cycles. Both the Ibarra-Medina-Krawinkler model (by means of the $c$-factor) and the alternate equation proposed by Park and Ang (see [62]) have this capability, as do the low-cycle fatigue damage indices utilized by other researchers [27, 76]. However, these models require calibration of multiple parameters, which adds complexity and could result in multiple solutions for a given specimen.

![Figure 3-16: Influence of the variation in loading protocol on the calibrated $\gamma$-factor](image)
3.1.4 Summary

A literature review showed that, in previous test programs on ductile reinforced concrete columns, loading cycles in the range of 2% drift or below had a limited effect on strength degradation or deformation capacity. The test program on reinforced concrete beams conducted as part of this work showed similar results, with variations in the cycle content at or below 2.17% drift not being correlated with the ultimate deformation capacity or the residual energy dissipation capacity. The magnitude of axial elongation at 2.17% drift was correlated with the cycle content up to that point, but the maximum elongation, which occurred later in the tests, was independent of this earlier variation. The deformation capacity of the beams was independent of the variance in elongation at 2.17% drift, but correlated with the maximum elongation.

The variability in strength degradation and deformation capacity of the test specimens was analysed using cumulative damage models. It was found that significant variability was present despite the specimens being nominally identical, emphasising the importance of considering uncertainty when conducting analysis based on cumulative damage measures. It was also found that the calibrated model parameters were affected by the load history of the specific specimen being calibrated against. Specimens with a larger number of smaller displacement cycles tended to show a higher level of resistance to cyclic deterioration. This is consistent with the finding regarding the limited importance of moderate level loading cycles.

3.2 Loading rate

3.2.1 Background

There have been a limited number of past studies that have experimentally investigated the effects of loading rate on the cyclic behaviour of ductile reinforced concrete components [77-82], by testing otherwise identical specimen pairs or sets at both static and dynamic speeds. Aside from a consistent increase in peak strength due to dynamic loading, the results from these test programs are varied. However, two tendencies were identified for the majority of the dynamically loaded specimens, relative to the static specimens: (i) percentage strength increases were largest at, or near, the yield point; and (ii) slightly faster cyclic strength degradation and reduced deformation capacities. Faster cyclic strength degradation here refers to a steeper slope of the post-peak section of the load-deformation envelope or larger drops in strength between cycles at a particular deformation demand.

Mutsuyoshi and Machida [77] tested twelve small-scale rectangular columns with dimensions of 150x100mm, three different types of reinforcing configurations, two different shear span lengths, and no axial load. Variables considered in the applied protocols were the loading rate and whether
the loading was monotonic or reversed cyclic. An increased strength due to dynamic loading was observed in both the monotonic and cyclic tests. The strength increases were largest at yield and became lesser as the displacement was increased. The cyclic tests in particular exhibited limited strength increases due to dynamic loading at displacements larger than yield, which the authors attributed to the relatively small strain rates present at cycle peaks, due to that being the point of load reversal. The cyclic specimens that had larger shear span ratios displayed reduced deformation capacities when subjected to high loading rates. The specimens with smaller shear span ratios did not follow this trend. All cyclic specimens reached large drifts (above 4.0%) prior to any significant divergence in the rate of cyclic strength degradation between equivalent static and dynamic tests.

The monotonic specimens were taken to over 15% drift without reaching failure in both static and dynamic loading. The authors did not comment on the observable damage patterns between the various specimens, but did state that one of the dynamically tested specimens failed in shear, while its corresponding static equivalent failed in flexure. However, the specimen reported to have failed in shear exhibited a stable lateral load capacity until over 4.0% drift, indicating that the definition of ‘shear failure’ used by the authors may not be consistent with the modern usage.

Otani et al. [78] tested four pairs of columns, with each pair consisting of two nominally identical specimens, one tested statically and the other dynamically. The columns had dimensions of 300x200mm, and the variables altered between the pairs were the transverse reinforcement content and the shear span length. All of the specimens had zero axial load and were subjected to identical cyclic loading protocols. Strength increases at yield due to dynamic loading were observed in all four specimen pairs. As deformation demands increased, the strengths of the dynamically loaded specimens remained higher than their statically loaded equivalents until just before the point of failure. In three of the four pairs, the dynamically loaded specimens were reported as having a reduced deformation capacity, with the largest reduction being in the most slender specimen. However, the reported deformation capacities reported by Otani et al. are influenced by the definition used by the authors, a drop below 80% of the yield strength, as the dynamic specimens had significantly higher yield strengths. A significant divergence in the rate of cyclic strength degradation between any of the equivalent static and dynamic tests did not occur until around 3.0% drift. Otani et al. reported that crack patterns were similar regardless of loading rate.

Wang et al. [79] tested ten pairs of columns, with each pair consisting of two nominally identical specimens, one tested statically and the other dynamically. The columns all had dimensions of 200x200mm, but otherwise a range of variables was altered between the ten different column pairs. The majority of the specimens had an axial load ratio of 0.05\(A_g f'_c\). Applied loadings consisted of uniaxial monotonic (one pair), uniaxial cyclic (eight pairs), and biaxial cyclic (one pair) tests. In all specimens, the yield and ultimate strengths increased under dynamic loading, with the relative strength increase at yield being larger than that at ultimate in all but one pair. The dynamically
tested specimens exhibited faster cyclic strength degradation in all specimen pairs, but the rate of cyclic strength degradation did not begin to diverge from static equivalents until loading cycles at over 4.0% drift. No information was provided regarding the effects of dynamic loading on the observable damage.

Fan et al. [80] tested 15 nominally identical beam-column joints with varying levels of axial load. The same cyclic loading protocol was applied to all specimens, but with three different loading rates. Strength increases at both yield and ultimate were observed in the dynamically loaded specimens, with larger relative increases at yield. The authors concluded that dynamic cyclic loading caused an increase in the rate of cyclic strength degradation and a decrease in the distribution of damage, relative to static cyclic loading. However, the cyclic strength degradation conclusion is not clearly supported by the hysteretic data provided in the paper. In all equivalent specimen sets, the rate of cyclic strength degradation between dynamic and static specimens did not begin to diverge until around 2.0% drift.

Ghannoum et al. [81] conducted an experimental study on the effect of loading rate on the cyclic response of reinforced concrete columns. However, this study focussed on specimens with limited ductility, and is therefore not discussed here in detail. Tests by Shah et al. [82] were not considered due to the extreme small scale of the specimens.

Additional experimental work has been undertaken considering only monotonic loading of reinforced concrete, and a substantial number of test programs have investigated the effect of strain rate at the material level. Several studies have attempted to comprehensively summarize the state of knowledge on these topics [83-86]. Findings include increased strengths due to dynamic loading, but little consensus regarding its effect on ultimate strain or deformation capacities. Lower strength concrete and reinforcing steel were found to have larger increases in relative strength when subject to dynamic loading, as compared to higher strength materials. Concrete in tension was found to have a larger relative strength increase than concrete in compression. The increase in reinforcing steel strength in the strain hardening range was found to be less than in the yield plateau range. The difference between ‘upper’ and ‘lower’ yield strengths was found to increase with loading rate. In addition to strength increases of the raw materials, an increase in the bond strength between deformed reinforcing bars and concrete has been reported by a number of researchers [87-89].

Further understanding regarding the effects of dynamic loading rates is required to determine whether it is necessary to consider rate-effects when designing or assessing the seismic performance of structures. In particular, further research at the component-level and system-level is needed to complement existing material-level knowledge.
3.2.2 Experimental results

A total of nine dynamic-then-static tests, seven purely static tests, and one purely dynamic test were conducted as part of the experimental program described in Chapter 2. Three of the dynamic-then-static experiments and two of the purely static experiments were conducted on specimens with axial restraint, and are discussed in a later section. The remaining tests are considered here. For the dynamic-then-static specimens that were repaired following the initial loading (LD-1-R and LD-2-R), this section solely focusses on the dynamic pre-repair results, as the post-repair behaviour is discussed elsewhere.

3.2.2.1 Quantifying the applied loading rates

The maximum loading rate of the earthquake displacement histories ranged from 2.13-4.50% drift per second, as previously shown in Figure 2-13. These loading rates are assumed to be representative of those typically induced in reinforced concrete moment frames during earthquakes, due to the displacement histories having been derived from non-linear response history analysis on a prototype frame building. However, it is noted that the relatively long natural period (2.1s) of the prototype building means that higher loading rates could reasonably be expected to occur in stiffer structures. The natural period of the prototype building also elongated to over 3.0s during the non-linear time history analysis. The period used in the cyclic dynamic test was 2.5s, which was chosen to represent the mid-range of the initial and elongated natural periods of the prototype building.

The strain rates induced in the longitudinal reinforcement during the earthquake displacement history tests are of particular interest. Longitudinal reinforcement strains were measured during the experimental program, as discussed in Section 2.3.2. However, using this strain data as a basis for estimating the strain rates of the longitudinal reinforcement has limitations, because (i) the measured strains only provide average strains across the gauge lengths and do not capture local maxima, (ii) the lowermost gauges, which usually exhibited the largest measurements, include deformations due to strain penetration, and (iii) in some specimens, the gauges did not provide accurate readings for the duration of the dynamic loadings. Nonetheless, these data are used to provide an approximate range of strain rates incurred during the dynamic loadings.

Figure 3-17 shows the measured longitudinal steel strain, strain rate, and hysteretic behaviour (for reinforcement on the south side of the beams) during the earthquake displacement history tests for specimens LD-1 and P-2. These specimens are shown as the strain measurement gauges were able to provide high quality data through the duration of the tests. However, the range of strain rates measured for LD-1 and P-2 are assumed to also be reasonably representative of LD-2 and P-1, respectively, due to modest differences in the drift velocity histories between the protocols. The data in Figure 3-17 correspond to the average strain along the two gauges immediately adjacent to the foundation (a total length of 228mm), which covered the majority of cracking that occurred.
during the earthquake loadings. The measurements include any strain penetration into the foundation that occurred, and no adjustment for this is provided; therefore, the average strains presented in Figure 3-17 are likely higher than the actual average strains induced in the steel. The peak strain rates in individual loading cycles are generally less than 0.03s\(^{-1}\), with the absolute maximum strain rates being 0.05s\(^{-1}\) for LD-1 and 0.10s\(^{-1}\) for P-2. These values are towards the upper bound of earthquake strain rates that have been suggested in the literature, with most estimates ranging from \(10^{-3}\) to \(10^{-1}\) s\(^{-1}\) [90-92]. In both specimens, the absolute maximum strain rate corresponded to the cycle in which yielding first occurred, despite the peak drift velocity not necessarily coinciding with the yield cycle.

3.2.2.2 Effect of loading rate on strength

In Figure 3-18, the hysteretic responses from the dynamic displacement history tests are compared against results from the static monotonic test and the shear forces corresponding to the nominal moment capacity \(M_n\) and the overstrength moment capacity \(M_o\). Moment capacities were calculated using NZS 3101:2006, which stipulates that material strengths of 1.35\(f_y\) and \((f'_c+15\text{MPa})\) should be used to determine \(M_o\). Design, rather than measured, material properties were used in calculating both \(M_n\) and \(M_o\), but since the measured yield strength of the longitudinal steel was within 1% of the design yield strength, use of measured material properties would have given very similar results. The beam overstrength was only slightly exceeded in three of the six specimens due to the spike in strength at yield during dynamic loading, with a maximum exceedance of 4.7%. However, it should be noted that overstrength factors are largely intended to account for variability in yield strength, not amplification in demand due to dynamic loading rates. Furthermore, had a stiffer structure been used to derive the displacement histories, higher strain rates could have further increased the beam strengths.

In Table 3-3, the strengths of both the static and dynamic specimens are provided at four specific points: (i) the first instance of cracking \(V_{cr}\), (ii) the first instance of yielding of the longitudinal steel \(V'_y\), (iii) the first instance of reaching a displacement of 14.0mm, which roughly corresponds to a displacement ductility \(\mu\) of 1.5 or a drift ratio of 0.54% \((V_{\mu=1.5})\), and (iv) the point of ultimate strength \(V_u\). Note that the ultimate (peak) strengths of the dynamic specimens coincided with the yield point in all cases. The cracking and yielding points were visually estimated from the force-displacement results of each specimen, but little judgement was required as distinct changes occurred in the force-displacement plots at these points.
The influence of loading characteristics and boundary conditions on plastic hinges

Figure 3-17: Strain rates and strain hysteresis for longitudinal reinforcement of (a) LD-1 and (b) P-2 specimens
Figure 3-18: Hysteretic responses of dynamic earthquake displacement history tests compared against static monotonic test data and nominal and overstrength capacities calculated as per NZS 3101:2006

Dynamic increase factors (DIFs), or the ratio of the dynamic strength to the static strength, are also included in Table 3-3 for the dynamic tests. On average, the dynamic displacement history tests had DIFs at cracking, yielding, $\mu = 1.5$, and ultimate of 1.41, 1.17, 1.09, and 1.11, respectively. The high DIF values at cracking are as expected due to the relatively large effect that dynamic loading rates have on the tensile strength of concrete [85]. The higher strength increases observed at yield than at $\mu = 1.5$ are also expected, given the significance of an ‘upper’ yield strength increases with loading rate [77]. For the two pairs of specimens subjected to identical dynamic loadings (LD-1 &
The influence of loading characteristics and boundary conditions on plastic hinges

LD-1-R; LD-2 & LD-2-R), similar strengths at yield and $\mu = 1.5$ were achieved (< 5% difference), but differences in cracking strengths were on the order of 10-15%. This is likely due to the high variability of the tensile strength of concrete and the sensitivity of the tensile strength to loading rate. The strength increases due to dynamic loading resulted in corresponding increases in the secant stiffness to yield, as loading rate was not found to affect the yield drift, which was in the range of 0.38-0.46% in all specimens.

The flexural strength of tension-controlled ductile reinforced concrete beams, such as the specimens in these experiments, is largely dependent on the strength of the longitudinal reinforcing steel. Equations proposed by Malvar [86], Eibl [91], and Soroushian and Obaseki [92] were used to calculate the theoretical dynamic increase factors shown in Table 3-4 for 300MPa reinforcing steel yield strengths due to strain rates in the range of those measured on the longitudinal reinforcement during the dynamic loading at first yield (i.e. 0.05 to 0.1s$^{-1}$). It should be noted that these equations were developed for monotonic loading. In cyclic loading, the strain rate varies from a peak value near the midpoint of the cycle to zero at the zenith, but it is unknown how this varying strain rate impacts the strength increase. For strain rates ranging from 0.05 to 0.1s$^{-1}$, the theoretical DIF at yield from the three equations considered here ranges from 1.14-1.36. These values are comparable to the DIFs at yield of the dynamic displacement history tests, which ranged from 1.13-1.22 (Table 3-3), with the DIF formulations from Eibl and Soroushian and Obaseki providing closer matches than that of Malvar.

Table 3-3: Lateral strengths and dynamic increase factors at cracking, first yield, the first instance of reaching a displacement ductility of 1.5, and the point of reaching ultimate strength. Note that $V_u = V_f$ for all dynamic specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_{cr}$ (kN)</th>
<th>$V_f$ (kN)</th>
<th>$V_{\mu=1.5}$ (kN)</th>
<th>$V_u$ (kN)</th>
<th>DIF$_{cr}$</th>
<th>DIF$_f$</th>
<th>DIF$_{\mu=1.5}$</th>
<th>DIF$_u$</th>
</tr>
</thead>
</table>
| Static tests
| CYC | 52 | 110 | 107 | 120 | - | - | - | - |
| P-2-S | 44 | 116 | 108 | 120 | - | - | - | - |
| LD-2-S | 45 | 107 | 109 | 115 | - | - | - | - |
| CYC-NOEQ | 46 | 113 | 106 | 121 | - | - | - | - |
| MONO | 52 | 115 | 108 | Not used | - | - | - | - |
| Avg. | 47.8 | 112.2 | 107.6 | 119.0 | - | - | - | - |
| Dynamic displacement history tests
| P-1 | 63 | 130 | 114 | 130 | 1.31 | 1.16 | 1.06 | 1.09 |
| LD-1 | 60 | 135 | 123 | 135 | 1.26 | 1.20 | 1.14 | 1.13 |
| LD-1-R | 66 | 133 | 118 | 133 | 1.38 | 1.19 | 1.10 | 1.12 |
| P-2 | 74 | 137 | 119 | 137 | 1.55 | 1.22 | 1.11 | 1.15 |
| LD-2 | 75 | 127 | 114 | 127 | 1.57 | 1.13 | 1.06 | 1.07 |
| LD-2-R | 66 | 129 | 115 | 129 | 1.38 | 1.15 | 1.07 | 1.08 |
| Avg. | 67.3 | 131.8 | 117.2 | 131.8 | 1.41 | 1.17 | 1.09 | 1.11 |
| Cyclic dynamic test
| CYC-DYN | 66 | 122 | 109 | 122 | 1.38 | 1.09 | 1.01 | 1.03 |

83
Table 3-4: Dynamic increase factors for 300MPa steel yield strength, as per equations proposed by various researchers

<table>
<thead>
<tr>
<th>Strain rate (s⁻¹)</th>
<th>Malvar</th>
<th>CEB Bulletin 187</th>
<th>Soroushian and Obaseki</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>1.32</td>
<td>1.14</td>
<td>1.19</td>
</tr>
<tr>
<td>0.1</td>
<td>1.36</td>
<td>1.15</td>
<td>1.21</td>
</tr>
</tbody>
</table>

Even though existing DIF formulations may be able to adequately determine the change in yield strength of reinforced concrete beams, knowledge regarding the strain rate during a specific earthquake is required in order to apply such factors in design. Furthermore, the dynamic amplification of yield force is very short lived during the earthquake and not of significant consequence to the system behaviour. Therefore, the current practice of neglecting strength increases due to loading rate is understandable.

Perhaps the primary reason for quantifying strength increases due to seismic loading rates is to ensure that overstrength calculations adequately account for such potential effects. Beam overstrengths are used to ensure that a beam-sway mechanism can form, and that premature shear failure will not occur. However, the fact that dynamic strength increases are largest at first yield makes it unlikely that beam overstrengths would be exceeded due to loading rate effects. This is because, in practice, beams in moment frame buildings will have an effective flange width of the slab that contributes to the overstrength. This effective flange width will not significantly contribute to beam strength at first yield, as larger tension strains in the beam are required to activate the full effective width of the flange. Furthermore, strain hardening of the beam longitudinal steel, another source of increased strength, by definition does not occur until after yielding.

3.2.2.3 Effect of loading rate on deformation capacity and damage pattern

Specimen CYC-DYN was the only test that was loaded dynamically until failure. Force-displacement plots for specimens CYC and CYC-DYN are compared in Figure 3-19. As previously discussed, the dynamic cyclic test exhibited a strength increase of approximately 10% relative to the static cyclic test at low displacement demands. However, faster cyclic strength degradation in the dynamic test resulted in the static test having a higher peak strength in all cycles above 2% drift. A 20% drop in strength in the dynamic test first occurred during the second cycle to 3.26% drift, while the same drop in strength did not occur until the second cycle to 4.34% drift in the static test. These results imply that dynamic loading rates had a negative influence on cyclic strength degradation and deformation capacity; however, both specimens exceeded typical code drift limits of 2.5% and the curvature ductility limit $K_d = 19$ (equivalent to 2.85% drift in these beams) of NZS 3101:2006.
The influence of loading characteristics and boundary conditions on plastic hinges

In Figure 3-19, envelopes of the cyclic portions of the tests for specimens CYC and CYC-DYN are compared against the hysteretic behaviour of specimen CYC. Dynamic loading rates up to 1.36% drift seemingly had little effect on the ultimate behaviour of the specimens. The rate of cyclic strength degradation is very similar between the three specimens. However, a caveat to these results is that specimen LD-1-R, as seen in Figure 3-23, may have exhibited worse performance had it not been repaired following the initial dynamic loading.

In Figure 3-20, envelopes of the cyclic portions of the tests for specimens P-1 and LD-1 are compared against the hysteretic behaviour of specimen CYC. Dynamic loading rates up to 1.36% drift seemingly had little effect on the ultimate behaviour of the specimens. The rate of cyclic strength degradation is very similar between the three specimens. However, a caveat to these results is that specimen LD-1-R, as seen in Figure 3-23, may have exhibited worse performance had it not been repaired following the initial dynamic loading.

In Figure 3-21, envelopes of the cyclic portions of the tests for specimens P-2 & P-2-S, and LD-2 & LD-2-S, are compared against the hysteretic loops of specimens CYC and CYC-DYN (recall that ‘-S’ in the specimen name indicates the earthquake input was run at static loading rates). The specimens that had previously been loaded dynamically (shown in red) exhibited faster cyclic strength degradation in the negative loading direction, similar to that of specimen CYC-DYN. This early degradation trend for the dynamically loaded specimens is not as obvious in the positive loading direction.

The faster cyclic strength degradation in specimens CYC-DYN, P-2, and LD-2 resulted in a 20% drop in strength first occurring during the cycles to 3.26% drift, while the same strength loss did not occur until 3.80 to 4.34% drift in purely static specimens CYC, P-2-S, and LD-2-S. However, it should be recognized that the post-peak portion of the envelope curve was also highly variant between all specimens, even between those loaded at static rates. For example, specimen LD-2-S was able to maintain over 70% of its peak load-carrying capacity up to 4.88% drift, while specimen P-2-S, despite having exhibited less cyclic strength degradation up to that point, lost all strength due to rupture of longitudinal reinforcement during the cycles to 3.80% drift.
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Figure 3-20: Cyclic envelopes of the static portions of P-1 and LD-1, compared against the baseline static test (CYC)

Similar to the conclusions regarding the effects of load history, it is here found that the predominant damage pattern, which usually developed early in the test, was a better indicator than loading rate for determining the deformation capacity. A comparison of the visual damage states after all loading cycles to 2.17% drift of the equivalent static and dynamic specimens of Figure 3-21 is shown in Figure 3-22. Specimens CYC-DYN and P-2 exhibited increased damage as compared to their equivalent static specimens. However, the damage states of LD-2 and LD-2-S were similar, with both the static and dynamic specimens exhibiting heavily concentrated damage. The CYC and LD specimens exhibited wider crack widths than the P specimens because they had increased elongation due to a larger number of applied loading cycles, as previously discussed.

In all specimens with an initial dynamic loading followed by static cyclic loading, additional flexural (both hairline and non-hairline) and longitudinal cracks opened up during the first static loading cycle. This was likely due to loading rate effects rather than drift demand, as additional cracks had stopped appearing in purely static specimens by equivalent points in the tests. However, these additional cracks did not necessarily lead to significant spreading of inelastic damage. For example, the localized damage in specimens P-2 and LD-2 was carried through until failure despite the fact that some additional cracks formed during the first static loading cycle. A reduction in the number of non-hairline cracks under dynamic loading rates has previously been cited as a concern due to its potential for increasing the strains in the reinforcing steel [11]. However, in this test program, increased steel strains were not found to be a serious issue. In all specimens, longitudinal cracks, cover concrete delamination, and longitudinal bar buckling occurred prior to failure, indicating substantial spreading of the yielding zone. The detrimental effect of fewer cracks instead primarily manifested in more sliding shear and localized bending of longitudinal bars due to dowel action.
Figure 3.21: Cyclic envelopes of the static portion of the tests for P-2, P-2-S, LD-2, and LD-2-S, compared against the (a) the baseline static test (CYC) and (b) the baseline dynamic test (CYC-DYN)
Figure 3-22: Photographs of plastic hinges after all loading cycles at or below 2.17% drift. The left column of specimens were subjected to purely static loading up to this point, and the right column of specimens were subjected to equivalent dynamic protocols.

Significant variability in visual damage states was observed between identical dynamic tests. Specimens LD-1-R and LD-2-R were subjected to the same dynamic long duration protocols as specimens LD-1 and LD-2, respectively, but markedly different levels of damage were observed between the two nominally identical pairs (Figure 3-23). Specimen LD-1-R exhibited increased damage as compared to LD-1, while specimen LD-2-R exhibited reduced damage as compared to LD-2. The effects of these altered damage patterns on the ultimate capacities of the beams could not be directly evaluated, as specimens LD-1-R and LD-2-R were repaired following dynamic
loading. The differences in observable damage between the specimen pairs in Figure 3-23 are similar to the differences between P-2 and P-2-S, as seen in Figure 3-22. This indicates that the differences in visual damage states between specimens may have been predominantly due to variability in the locations of the large cracks through the depth of the beams, and the loading rate was not a significant factor.

![Images of specimens LD-1, LD-1-R, LD-2, and LD-2-R](image)

**Figure 3-23**: Variability in observable damage after completion of the initial dynamic portion of the test for two pairs of specimens with identical applied earthquake loadings.

As previously discussed, studies in the literature that tested identical ductile specimens subjected to cyclic loading at both static and dynamic loading rates tended to show reduced deformation capacity in dynamic tests, albeit with several exceptions. The results from this experimental program on reinforced concrete beams seemingly support the notion that a localization of damage and a decrease in displacement capacity may be more likely to occur for beams under dynamic loading. However, a number of caveats apply:

- In this experimental program, the detrimental effect of dynamic loading was due to increased susceptibility to shear sliding damage. This effect may be reduced for beams in
buildings, where sliding is potentially restrained by boundary conditions or increases in axial load due to beam elongation. A discussion of the experiments on axially-restrained beams that investigate this hypothesis is provided in Section 3.3.

- The dynamic specimens that exhibited enhanced strength degradation (P-2, LD-2, and CYC-DYN) developed the same damage pattern, with one large crack coinciding with a particular stirrup set dominating the failure mode. It is possible that such a failure mode is strongly affected by the variability of where a primary crack first forms. Furthermore, this damage pattern was not necessarily a function of the loading rate, as it also developed in specimen LD-2-S and did not develop in specimen LD-2-R.

- More heavily reinforced specimens may not respond to dynamic loading in the same manner, as increases in concrete tensile strength would have a lesser impact on crack pattern. Increased levels of transverse reinforcement may also induce more distributed damage, as cracks tend to coincide with stirrup locations. Similar suppositions were put forward by the Canterbury Earthquakes Royal Commission [11].

### 3.2.3 Summary

A literature review showed a tendency for the strength of ductile reinforced concrete components to increase with loading rate, with the highest relative increases at yield. Similar results were obtained for the beam specimens of this study. The earthquake displacement histories resulted in average dynamic increase factors relative to quasi-static specimens of 1.41, 1.17, and 1.11 at cracking, yield, and ultimate, respectively. In all cases, the peak strengths of the dynamic specimens corresponded with the yield strengths. The peak strengths of static specimens tended to occur well beyond the yield point, at drifts of 2.0% and above. Because the dynamic increase factors are associated with yielding rather than post-yielding performance, the change in strength is not likely to alter the desired weak-beam-strong-column behaviour of a capacity designed moment frame system that is designed without consideration of dynamic effects.

Experimental programs in the literature also showed a tentative tendency for reduced deformation capacities with increased loading rates. The three beam specimens that exhibited the lowest deformation capacities were those that were tested dynamically to failure (CYC-DYN) and dynamically to 2.17% drift (P-2 and LD-2). These specimens achieved drift capacities approximately 75-85% of equivalent specimens loaded at static loading rates. This indicates that dynamic loading negatively affects deformation capacity, but further research is required to isolate the variables influencing this reduction in deformation capacity. All specimens exhibited deformation capacities in excess of current code prescriptions, regardless of the applied loading rates. The experimental data in the literature are too varied to draw conclusions about the effects of loading rate on the damage pattern of reinforced concrete plastic hinges.
3.3 Axial elongation and restraint

3.3.1 Background

Axial elongation of reinforced concrete plastic hinges has been acknowledged in the literature for several decades, and a comprehensive early paper on the subject was written by Fenwick and Megget [55]. They describe the causes of what are here termed ‘geometrical elongation’ and ‘residual elongation’. Geometrical elongation is a function of the higher tension strains than compression strains that develop in reinforced concrete members subject to flexure, thereby giving rise to movement of the neutral axis and longitudinal displacement of the centroid. Yielding of the tensile steel in plastic hinges causes geometrical elongation to sharply increase to significant values. Residual elongation is present in plastic hinges under reversed cyclic loading, where compression steel that previously yielded in tension is unable to fully yield back under compression loading. This results in some remaining elongation after the hinge rotation returns to zero.

Geometric elongation is well understood, and is necessary for ductile behaviour of reinforced concrete. Fenwick and Megget proposed a method of calculating geometrical elongation in beams [Equation (3-7)], a modified version of which has now been incorporated into Amendment 3 of NZS 3101:2006 [Equation (3-8)]. For unidirectional hinges, residual elongation does not occur and this equation can therefore be used to predict the total hinge elongation.

\[ e_g = \frac{\theta_p}{2}(d - d') \]  \hspace{1cm} (3-7)

\[ e_g = \frac{\theta_m}{2}(d - d') \]  \hspace{1cm} (3-8)

where \( e_g \) is the geometric hinge elongation, \( \theta_p \) is the plastic rotation of the hinge, \( \theta_m \) is the total rotation across the hinge, \( d \) is the distance from extreme compression fiber to tension steel centroid, and \( d' \) is the distance from extreme compression fiber to compression steel centroid

Fenwick and Megget also listed two reasons for longitudinal steel being unable to fully yield back in compression, thus giving rise to residual elongation in reversing plastic hinges. The first is loose concrete particles filling the tension cracks in the concrete, and the second is the axial component of the diagonal compression struts in the truss model of reinforced concrete under combined shear-flexure actions (Figure 3-24). These actions are here termed the ‘contact stress effect’ and ‘diagonal compression strut effect’, respectively. Both of these factors cause a reduction in the force experienced by the compression steel. In longitudinally unrestrained conditions, residual elongation can make up the majority of total elongation. This is reflected by the equation for total elongation in NZS 3101:2006, which, in situations of zero axial load, is taken as 2.6 times the geometric elongation \( e_g \) calculated using Equation (3-8).
Axial load can force the longitudinal steel to yield back in compression upon load reversals. For this reason, tests on reinforced concrete columns with applied axial load often only exhibit geometric elongation. NZS 3101:2006 recognizes this by limiting the total elongation in members with axial load greater than $0.08A_g f'_c$ to $e_g$. In situations of axial load between 0-0.08$A_g f'_c$, the equations call for linear interpolation between 2.6-1.0$e_g$.

Certain damage states or plastic hinge mechanisms, such as longitudinal bar buckling or sliding shear deformations (both of which partially arise due to elongation), can prevent further elongation from occurring. Therefore, cyclically-loaded unrestrained reinforced concrete beams do not usually continue to elongate at large drift demands, but instead reach a maximum longitudinal strain prior to failure. In NZS 3101:2006, it is recommended that the maximum elongation of reversing plastic hinges be taken as 3.6% of the beam depth. The complete set of NZS 3101:2006 equations for the elongation of reversing plastic hinges is therefore as given by Equation (3-9).

$$e_{rev} = \begin{cases} 
\frac{\theta_m}{2}(d - d'), & P \geq 0.08A_g f'_c \\
2.6\frac{\theta_m}{2}(d - d') \leq 0.036h_b, & P = 0
\end{cases}$$

(3-9)

where $e_{rev}$ is the elongation of a reversing plastic hinge, $P$ is the axial compression, $h_b$ is the beam depth, and $\theta_m$, $d$, and $d'$ are as defined in Equation (3-8).

Modelling of the combined effects of geometrical and residual elongation is an area of active research. Eom and Park [93], Lee and Watanabe [61], and Matthews [35] proposed step-by-step methods of determining the progression of total elongation in reinforced concrete beams subject to cyclic loading. Visnjic [94] and Encina et al. [95] showed that properly conditioned fiber elements are able to capture the elongation behaviour of reinforced concrete beams and walls, respectively. Finally, methods for modelling elongation that could be incorporated into lumped-plasticity style analysis have been proposed [96, 97]. These models essentially mimic fiber elements, as they
involve meshes of axial spring members with material hysteretic behaviour. However, they have the advantage of being able to include elements to simulate the diagonal compression strut effect, as described by Peng et al. [97].

The interaction between the structural elements in a moment frame due to beam elongation is perhaps best understood through indeterminate frame tests. A selection of such frame tests that present results related to beam elongation are available in the literature [34, 35, 98-104]. In these tests, accurate emulation of in-situ boundary conditions is difficult, and some compromises must be made. A detailed description of the various configurations used in many of these test programs is provided by Peng [104].

Measurements of beam elongations from these test programs showed that the location of the beam within the structural system had a large impact on the magnitude of elongation that occurred. Bechtoula et al. [34] tested two single-bay two-storey moment frames (without a slab), and found that the total elongation of the second story beams was nearly double that of the first story beams. Matthews [35] tested a single-story 2x1 bay frame with a pre-stressed precast concrete flooring system. An elaborate loading system was employed to ensure that the columns neither restricted nor exaggerated beam elongation (i.e. no axial load was induced in the beams due to column restraint). In the direction with two bays, the precast flooring units spanned past the centre column. The beam plastic hinges adjacent to this centre column exhibited total elongation on the order of one-half that of the hinges adjacent to the exterior columns during loadings up to 2% drift. Reductions in beam elongation due to interactions with pre-stressed precast flooring systems were also reported by MacPherson [102], Lau et al. [103], and Peng [104].

These results indicate that, given certain conditions, both columns and slabs can serve to restrain beam elongation. The lateral stiffness of columns provides restraint, but this stiffness changes along the height of the building, and is dependent on the degree of yielding present in the columns and the degree to which beams in adjacent stories have elongated. The magnitude of restraint provided by floor systems is dependent on the type of floor system in question (i.e. cast-in-place or precast), as well as the connection details between the floors, beams, and columns.

No experiments have been conducted to the author’s knowledge that investigate the effects of varying axial load on the behaviour of ductile reinforced concrete beams. The restrained beam tests conducted as part of this experimental program provide data on the interaction between the axial compression that can be induced due to elongation, and the elongation and load-deformation characteristics of the plastic hinge itself. Furthermore, the earthquake loading histories provide the opportunity to compare the new NZS 3101:2006 elongation equations against realistic seismic loadings with variable cycle contents.
3.3.2 Experimental results – unrestrained specimens

This section focusses on the elongation behaviour of the unrestrained and unrepaired specimens, and specifically on evaluating the predictive capacity of the elongation equation for reversing plastic hinges of NZS 3101:2006 [Equation (3-9)].

In Figure 3-25, the elongation versus drift relationships exhibited by the test specimens during the initial earthquake-type loadings are compared against the NZS 3101:2006 equation for reversing plastic hinges with no axial load. The elongation versus drift response of specimen CYC-NOEQ is also provided, and the response of specimen CYC is included in each of the plots as a standard cyclic baseline. For specimens CYC and CYC-NOEQ, only data due to cycles at or below 2.44% drift are included, as shear sliding mechanisms tended to dominate beyond that point. Note that the specimens loaded with a static earthquake loading (P-2-S and LD-2-S) are not included as they had elongation versus drift responses nearly identical to their dynamic counterparts. The total rotation across the plastic hinge is approximated as the plastic rotation \( \theta_m = \theta_p \) to avoid a more complex bi-linear elongation versus rotation relationship [i.e. a geometrical elongation calculated by Equation (3-7) is used instead of Equation (3-8)]. This was deemed justified as the maximum elastic rotation across the plastic hinge length of the test specimens is only 0.07rad, when calculated using a yield curvature of \( 2f_y/(E_s h) \) [1] and a plastic hinge length of \( 0.022d_b f_y + 0.08M/V \) [105]. However, it may be more common in practice to use the lateral drift or chord rotation instead of the total rotation across the plastic hinge, which would result in a higher predicted elongation for a given drift demand. The NZS 3101:2006 equation corresponding to the total lateral drift (\( \theta_m = \theta_{tot} \)) is therefore also included in Figure 3-25, although it is noted that the effects of using \( \theta_{tot} \) rather than \( \theta_m \) are dependent on the shear span length.

The NZS 3101:2006 equation accurately predicts the elongation of specimen CYC, modestly over-predicts the elongation of the long duration earthquake motion specimens and specimen CYC-NOEQ, and significantly over-predicts the elongation of the pulse-type earthquake motion specimens. This is unsurprising as the equation was developed using data from reversed-cyclic experimental test programs, and therefore contains two conservative assumptions for application to earthquake response histories: (i) the maximum rotation is the same in both loading directions, and (ii) a cycle content comparable to that of typical cyclic protocols (it was previously shown in Section 3.1 that the number of inelastic loading cycles applied to the beam specimens was correlated with the magnitude of elongation).

If a more accurate prediction of elongation is desired for a given loading history, consideration should be given to the maximum rotation in each direction and to some measure of the cycle content. Such a model was developed by Lee and Watanabe [61], and although their model employs a calculation procedure that is not conducive to arbitrary loadings, its general form is found to be
applicable here. The model consists of one term that accounts for the maximum rotation in each direction, and one term that accounts for increased elongation due to cyclic loading. The cyclic loading term approaches an asymptotic maximum, in order to simulate the diminishing increase in elongation with each cycle that has been observed in specimens cycled at a single drift demand (see Lee and Watanabe [61] for plots of such specimen behaviour).

Equation (3-10) is here proposed as a method for determining the maximum elongation due to a given arbitrary loading history. The values computed for the cases of both $\theta_m = \theta_p$ and $\theta_m = \theta_{tot}$ are included in each of the plots of Figure 3-25, with $\theta_m = \theta_p$ providing a better match to the data. The equation follows the form of the Lee and Watanabe model [61], but is relatively simple to compute and converges to the same maximum value as the NZS 3101:2006 equation in cases of high cycle contents and identical peak rotations in both directions. The term $0.6/N_{eff}$ governs the importance of cyclic loading. The denominator of this term is here taken as the number of effective cycles at the peak drift ($N_{eff}$) for simplicity, as it was previously defined in Section 3.1.2.2 and fits the experimental data well without the need for additional manipulation. For upper-bound estimates of $N_{eff}$ due to earthquake loadings, such as the $N_{eff} = 7T_n^{-1/3}$ relationship proposed by Chang and Mander [106], this term tends towards zero except for structures with high natural periods $T_n$. However, any metric that represents the importance of cumulative loading could easily be substituted, as long as the denominator is modified using a constant or other means so that similar degree of impact is obtained.

$$e_{rev} = \frac{(\theta_{pos}/2 + |\theta_{neg}|/2)}{2} (d - d') \left(2.6 - \frac{0.6}{N_{eff}}\right) \quad (N_{eff} \geq 1)$$ (3-10)

where $\theta_{pos}$ is the maximum rotation reached in the positive direction, $\theta_{neg}$ is the minimum rotation reached in the negative direction, $N_{eff}$ is the number of effective cycles at the peak drift, and $d$ and $d'$ are as defined in Equation (3-7).

3.3.3 Experimental results – restrained specimens

This section discusses results from all restrained specimens, except specimen LD-2-LER-R, which was repaired following an initial dynamic earthquake loading and is instead discussed in Chapter 6. For comparative purposes, results from equivalent unrestrained specimens are also considered here.
The influence of loading characteristics and boundary conditions on plastic hinges

Figure 3-25: Elongation versus drift relationships for earthquake/variable loading specimens and comparison with the NZS 3101:2006 Equation (3-9), the proposed Equation (3-10), and standard cyclic specimen CYC

3.3.3.1 Quantifying the induced axial load

The axial load ratio at cycle peaks during cyclic testing of the restrained specimens is shown in Figure 3-26(a). The axial load peaked at approximately 0.05$A_gf'_c$ in the heavily restrained (ER) specimens and 0.025$A_gf'_c$ in the lightly restrained (LER) specimens, where $f'_c$ here corresponds...
to the measured compressive strength of concrete. The maximum axial load ratios induced during the initial earthquake loadings in specimens LD-2-ER and LD-2-LER were approximately $0.02A_g f'_c$ and $0.015A_g f'_c$, respectively. The constant axial stiffness of the restraint system meant that the axial compression was reduced between peaks, as the residual elongation was less than the peak elongation. An example of the complete axial load variation with drift is shown in Figure 3-26(b) for specimen CYC-ER.

![Graph showing axial load ratio at cycle peaks during cyclic testing of restrained specimens.](Figure 3-26)

**Figure 3-26:** (a) Axial load ratio at cycle peaks during cyclic testing of restrained specimens, and (b) Axial compression versus lateral drift data from specimen CYC-ER

### 3.3.3.2 Effect of restraint on strength

Cyclic envelopes for specimens CYC-ER, CYC-LER & CYC, and LD-2-ER, LD-2-LER, & LD-2 are compared in Figure 3-27. Unsurprisingly, given the tension-controlled nature of the beam
specimens, higher levels of restraint led to increased strengths. The strengths of the restrained specimens continued to increase due to progressively higher compressive forces until cyclic strength degradation began to occur during cycles above 3% drift. Shear forces corresponding to the nominal and overstrength moments as per NZS 3101:2006 are included in Figure 3-27 to provide context for the strength increases relative to relevant values. However, the exact magnitudes of the strength increases should not be focussed on, as they are dependent on the stiffness of the axial restraint system which is specific to this test program.

Figure 3-27: Cyclic envelopes for equivalent specimen sets with varying levels of axial restraint. For the LD-2 specimens, the range of the curves corresponding to the dynamic earthquake loading is marked by a dashed line.

In all cases, the strength at yield was increased by 7-10% due to the axial restraint, but this increase was largely due to the initial axial load of 50kN applied to each restrained specimen. Not enough elongation occurred prior to yield to induce significant additional axial load in the beams. The
dynamic increase factors at yield in the LD-2 specimens were found to be very similar (on the order of 1.11-1.13) regardless of the level of axial restraint.

If the axial load is known, the strength of reinforced concrete members under varying axial load can be predicted with reasonable accuracy using first principles. Figure 3-28 shows that standard calculations (based on the provisions of NZS 3101:2006) can account for the strength increases due to axial load exhibited by the restrained beam specimens in this experimental program.

An important finding from this study is that dynamic loading rates and induced axial compression do not necessarily synergize to further increase the strength of beams. This is because the largest increase due to dynamic loading rates occurs at first yield (see Section 3.2), a point at which the elongation is negligible. At higher ductility levels, where elongation becomes appreciable, the dynamic strengths tend to be similar to the static strengths. Therefore, these two effects can be considered separately when determining maximum strengths.

3.3.3.3 Effect of restraint on elongation

In Figure 3-29, the elongation at cycle peaks for the three static cyclic specimens with varying levels of axial restraint are compared against the NZS 3101:2006 Equation (3-9) (using $\theta_m = \theta_p$). The equations effectively bound the total elongation of the various specimens. As previously discussed, the equation corresponding to no axial load predicts with high accuracy the total elongation of specimen CYC up to greater than 2% drift, after which shear degradation precluded further elongation. However, the equations are slightly conservative in accounting for axial load. The lines corresponding to axial load ratios of 0.05$A_g f_c'$ and 0.025$A_g f_c'$, the maximum axial compression induced in specimens CYC-ER and CYC-LER, respectively, over-predict the elongation for a given drift demand. Similar results were obtained for the three specimens with varying levels of axial restraint that were subjected to initial earthquake loading LD-2 (Figure 3-30). The peak axial load ratios induced during the initial earthquake loading (0.02$A_g f_c'$ and 0.015$A_g f_c'$
for specimens LD-2-ER and LD-2-LER, respectively) were enough to cause reductions in the maximum elongation on the order of 50%, relative to unrestrained specimen LD-2.

Figure 3-29: Elongation at peak drift for static cyclic specimens with various levels of axial restraint

Figure 3-30: Elongation versus lateral drift responses during the initial earthquake loading for the LD-2 specimens with various levels of axial restraint
Plotting the axial force induced at both peak and residual (where residual here refers to a state of zero drift) for specimen CYC-ER provides an interesting insight into the nature of residual elongation (Figure 3-31). While the total elongation at peak drift continues to increase until severe degradation occurs near the end of the test, the residual elongation approaches an asymptotic maximum. The magnitude of compressive force corresponding to this asymptote is approximately 0.025\(A_g f'_c\). This indicates that 0.025\(A_g f'_c\) is the axial load required to prevent additional residual elongation in these beams, a significantly lower value than the 0.08\(A_g f'_c\) used in NZS 3101:2006.

If the compressive force required to overcome the diagonal compression effect is taken as \(V \cos \theta\), and the contact stress effect is assumed to be proportional to the concrete strength and gross sectional area, the total compressive force to prevent residual elongation can be calculated using Equation (3-11).

\[
P_{er} = V \cos \theta + kA_g f'_c
\]  

(3-11)

where \(V\) is the shear force, \(\theta\) is the angle of the diagonal compression struts, \(k\) is a constant, and \(A_g\) is the area of the concrete section.

If Equation (3-11) represents the maximum compressive force required to prevent residual elongation, the upper bound of axial compression that could be induced in a beam due to elongation is given by Equation (3-12). For beams with two plastic hinges forming at either end, as is typical in earthquake-resisting moment frames, the term dealing with geometrical elongation should be multiplied by a factor of two, but the axial compression force required to prevent residual elongation should not (assuming that axial force is constant along the beam length).
\[ P_e = V \cos \theta + kA_g f'c + P_{eg} = V \cos \theta + kA_g f'c + Ke_g \]  

where \( P_{eg} \) is the axial load induced due to geometric elongation, \( K \) represents the stiffness of the surrounding structure, and \( e_g \) can be calculated by Equation (3-7).

Fenwick and Megget [55] used test data from four reinforced concrete beams with varying levels of constant axial load to derive an approximate value of 0.05 for \( k \). Figure 3-31 shows that, in the beam specimens used in this test program, a total compressive force of 0.025\( A_g f'c \) was sufficient to prevent further residual elongation. In these beams, the low levels of shear stress caused the compression struts to form at a very small angle, and if the \( V \cos \theta \) term is therefore deemed to be negligible, the appropriate value of \( k \) would be in the range of 0.025. The lower value of \( k \) obtained from these experiments, relative to the Fenwick and Megget experiments, may be due to the significantly lower reinforcement ratios in these beam specimens (0.6% versus 1.8% for Fenwick and Megget). An axial load ratio of 0.025\( A_g f'c \) corresponds to approximately 50% of the strength of the longitudinal reinforcement on one side of the beam specimens (i.e. 0.5\( A_s f_y \)). Since residual elongation is a result of longitudinal reinforcement that previously yielded in tension being unable to yield back in compression, it may be more appropriate to use \( A_s f_y \) instead of \( A_g f'c \) as a metric for determining a critical axial load, above which residual elongation is prevented. Additional experimental data is required to test this hypothesis.

Unfortunately, even if the parameters \( k \) and \( \theta \) are able to be accurately defined, knowledge about the interaction of the elongating hinge with the surrounding structure (i.e. the ‘\( K \’) stiffness parameter) is required to predict the induced axial load. Given the current state of knowledge, \( K \) is not able to be computed analytically, and therefore the maximum axial compression force that could be induced due to beam elongation is not readily calculable. This is an important area requiring further research, as axial compression induced in beams can cause increases in both the beam flexural strength and the column shear force, both of which have the potential to alter the intended failure mode of a moment frame.

### 3.3.3.4 Effect of restraint on shear deformation

In all unrestrained tests, shear sliding was a major contributing factor to the degradation and failure of the beams. Part of the motivation for conducting the restrained tests was to investigate the relationship between axial restraint and shear deformation. As expected, sliding behaviour and shear deformation was reduced in the restrained specimens. Figure 3-32 shows that, at 2.5% drift, 5% of total displacement was due to shear deformation for specimen CYC-ER, compared to ~10% for CYC-LER and ~18% for CYC. Similar results were observed in the LD-2 specimens.
3.3.3.5 Effect of restraint on deformation capacity and damage pattern

Photographs of the plastic hinges for the two sets (CYC and LD-2) of otherwise identical specimens with varying levels of restraint are shown in Figure 3-33 and Figure 3-34, respectively. The lack of shear deformation resulted in the damage following a more flexure-dominant progression for the restrained specimens. The large sliding plane cracks that played an important role in the degradation of the unrestrained specimens were completely eliminated in the –ER specimens, but were still evident in the –LER specimens, albeit with reduced widths and impacts. Despite higher axial loads, longitudinal bar buckling first occurred at higher drift levels in the restrained specimens than the unrestrained specimens (>3% compared to <2.5%), due to a lack of dowel action. The curvatures induced in the buckled bars of the restrained specimens were generally higher than in the unrestrained specimens due to the bars tending to buckle between a single stirrup spacing, as opposed to across multiple stirrups (Figure 3-35). This difference in buckling mode was due to the restrained specimens having reduced dowel action and reduced degradation of the core concrete near the beam ends. The solid core concrete of the restrained specimens was able to prevent the returns on the stirrups from pulling out and also provide inward and lateral support to the reinforcement, forcing the bars to buckle outwards between stirrups. As a result of the increased curvatures exhibited in this buckling mode, longitudinal bar fracture occurred on previously buckled bars in all restrained specimens, but only in three out of eleven unrestrained specimens.
Figure 3-33: Observable damage at 2.17% drift and test completion for cyclic specimens with varying levels of axial restraint
Figure 3-34: Observable damage at 2.17% drift and test completion for LD-2 specimens with varying levels of axial restraint
The influence of loading characteristics and boundary conditions on plastic hinges

Figure 3-35: Photographs of longitudinal bar buckling during first cycle to 4.34% drift for cyclic specimens with varying levels of axial restraint

Figure 3-36 shows the load-deformation envelopes of the cyclic part of the test for all non-repaired specimens. (Note that the cyclic envelopes of Figure 3-36 connect only the first loading cycle to each increment, so curves that appear to end before reaching a 20% drop in strength would have exhibited such a drop during the second loading cycle at the maximum applied drift.) The restraint mitigated the variability in deformation capacity (drift at 20% drop in strength) that was observed in the unrestrained specimens. The deformation capacity of all restrained specimens was 4.34% drift, while the unrestrained specimens had deformation capacities ranging from 3.26-4.88% drift. Useful comparisons of normalized envelope curves are not possible due to the changing axial load in the restrained specimens, but it is evident that the variability in terms of the slope of the post-peak portion of the envelope curve was also mitigated. This reduced variability was due to the consistent failure mechanism (longitudinal reinforcement rupture after bar buckling) exhibited by the restrained specimens, resulting in a sharp in-cycle strength drop. Conversely, unrestrained specimens could either exhibit in-cycle deterioration due to bar rupture (e.g. specimen P-2-S), gradual cyclic deterioration (e.g. specimen LD-1), or rapid cyclic deterioration (e.g. specimen P-2) (see Figure 2-19 for hysteretic plots of these specimens). The variation in the deformation capacities and rate of cyclic strength degradation between the unrestrained and restrained specimens was predominantly due to the differences in damage patterns previously discussed.
Summary

A literature review found that the modelling of axial elongation is an area of active research. The first explicit consideration of elongation in a codified format has been incorporated into the latest amendment of NZS 3101:2006. The elongation of the beam specimens without axial restraint was conservatively bounded by the new elongation equation of NZS 3101:2006 \[ \text{Equation (3-9)} \]. However, for accurate prediction of the maximum elongation due to the earthquake-type displacement histories, consideration had to be given to the maximum deformation reached in each direction, as well as the number of inelastic loading cycles. A simple equation was proposed that captures these metrics and converges to a maximum value equal to that given by the NZS 3101:2006 equation.

The NZS 3101:2006 \( 0.08A_g f'_c \) axial compression limit at which residual elongation is assumed to not occur was found to be conservative for the axially restrained beam specimens. It was found that an axial compression of \( 0.025A_g f'_c \) was sufficient to prevent any further accumulation of residual elongation. In this regard, the NZS 3101:2006 method of interpolating between \( 0-0.08A_g f'_c \) to determine the elongation in cases of low axial load is deemed conservative.

The increase in flexural strength of the restrained beam specimens due to the increasing axial compression was readily predicted using first principles. However, knowledge about the induced axial load is required for such calculations to be performed for the purpose of design or assessment. The induced axial load is dependent on both the elongation versus axial load relationship of the hinge itself and the stiffness of the surrounding structure, and is not readily calculable.
The large residual elongations in the unrestrained beam specimens were found to be responsible for the large sliding shear deformations and high degree of variability in damage patterns and deformation capacities. The restrained beam specimens exhibited significantly less shear sliding deformations and a lower degree of variability at high deformation demands.

3.4 Conclusions and Recommendations

The following list presents the key findings with regards to the effects that the number of loading cycles, the loading rate, and the restraint to axial elongation have on ductile plastic hinges in beams and columns. More detailed summaries for each of these three topics are given in Section 3.1.4, Section 3.2.3, and Section 3.3.4, respectively.

- The number of moderate level loading cycles, which is approximately defined as those at or below 2.0% drift, has a limited effect on the cyclic strength degradation or deformation capacity of plastic hinges in reinforced concrete beams and columns with ductile seismic detailing.
- Common cumulative damage models can overweight the importance of moderate level loading cycles on the degradation of ductile plastic hinges. Calibration of cumulative damage model parameters can therefore be affected by the loading protocols of the test specimens they are calibrated against.
- Loading rate has the largest effect on the flexural strength of ductile reinforced concrete components at first yield. This indicates that it may be possible to safely neglect loading rates when assessing overstrengths in moment frames, as other sources of overstrength, including strength increases due to restraint of axial elongation, tend to reach a maximum at higher deformation demands.
- Ductile reinforced concrete components subjected to high loading rates have exhibited an inconsistent tendency to have lower deformation capacities relative to static equivalents. However, this effect is relatively minor, highly variable, and does not preclude a high level of ductility in well-detailed plastic hinges.
- The experimental data in the literature are too varied and inconsistent to make conclusions about the effects of loading rate on the damage pattern of reinforced concrete plastic hinges. Further research is recommended.
- The NZS 3101:2006 elongation equations [Equation (3-9)] were found to be conservative predictors of the maximum elongation in the beam specimens of the experimental program, both with and without axial restraint.
• Accurate prediction of the maximum axial elongation for an arbitrary displacement history requires consideration of additional parameters not commonly used in design or assessment [Equation (3-10)].
• Considerable variability can exist in the deformation capacity of nominally identical ductile reinforced concrete beams. This variability is partially due to unrestrained axial elongation; the degree of variability is reduced when axial restraint is present.
Chapter 4

Accounting for moderate plastic hinging damage in post-earthquake assessments

This chapter investigates how moderate damage affects the post-earthquake residual capacity of plastic hinges in reinforced concrete moment frames. This includes the effects of concrete cracking and cover spalling, but does not explicitly include the effects of yielded longitudinal reinforcement, which is specifically addressed in Chapter 5.

4.1 The use of residual crack widths as a damage metric

In post-earthquake situations, flexural cracking may be the only visual damage indicator available on lightly- or moderately-damaged reinforced concrete components. Residual crack widths have therefore been heavily utilized as a tool for assessing the post-earthquake residual capacity of damaged components, as evidenced by their prominent role in FEMA 306 [16] and the JBDPA Post-Earthquake Guideline [20]. The degree of cracking has also been used as an important indicator in studies attempting to correlate accumulated damage (as measured by damage indices) to visual damage states and reparability [107].

This section investigates the adequacy of residual crack width metrics for informing estimates of a variety of parameters of interest in post-earthquake situations. These parameters include: the peak deformation demands placed on the plastic hinge during the earthquake that caused the damage, the in-situ plastic hinge length, and the plastic hinge residual capacity (i.e. the residual stiffness, strength, and deformation capacity). Several different residual crack width metrics are evaluated, including (i) total crack width, where total refers to the sum of the widths of all non-hairline cracks (taken as cracks greater than 0.2mm), (ii) maximum crack width, (iii) the ratio of maximum to total crack width, and (iv) the number of non-hairline cracks. Maximum crack width is the only one of these metrics that has typically been reported in past testing programs, and the data used here is therefore primarily from the experimental program previously described in Chapter 2.

It is noted that the use of 0.2mm as the definition of a ‘non-hairline’ residual crack width is not necessarily applicable to reinforced concrete components in general. The intention of this metric is to capture cracks at which yielding of the longitudinal reinforcement is likely to have occurred. However, cracks due to shrinkage, temperature effects, or gravity loading may exceed 0.2mm in reinforced concrete buildings. Visual identification of cracks as narrow as 0.2mm may also be
impractical in post-earthquake situations (the beam specimens of the experimental program were painted white to facilitate identification of cracks). While a non-hairline residual crack is defined using a width of 0.2mm in the analysis presented herein, a definition on the order of 0.5-1.0mm may be more appropriate for practical purposes. Further research is recommended to determine the most appropriate definition of ‘non-hairline’ residual crack width for use in post-earthquake assessments.

It is emphasised that this section is focussed on flexural cracking behaviour typical of moderately-damaged ductile plastic hinges. The analysis and conclusions are not applicable in cases of wide diagonal cracks, which can indicate that yielding of transverse reinforcement has occurred. Sliding plane cracks that may permit significant sliding shear deformations are also outside the scope of this section; they are instead addressed in Section 4.2.2.

4.1.1 Experimental program crack width data

During the experimental program previously described in Chapter 2, widths of all cracks over 0.2mm were measured at both peak and residual drift states for each loading cycle during cyclic loading of the test specimens (except for specimens P-2-S and CYC-NOEQ, where measurements were only taken at peak drift states of the first cycle of each loading increment). The peak and residual crack width measurements during cyclic loading were taken on the tension sides of the beams. Residual crack widths were measured on both sides of the beams after initial damaging earthquake loadings. In all cases, crack width measurements were taken until delamination of cover concrete precluded further measurement. No crack width data are available for specimens LD-2 and LD-2-S as delamination occurred during the initial earthquake loading in those specimens. Crack widths could not be measured on specimen CYC-DYN due to the dynamic loading rate. Since the crack widths were measured on the beam sides, the widths do not correspond to the sliding plane cracks that opened up in many specimens. In specimens with large sliding plane cracks, multiple cracks would converge together as they propagated towards the beam centres, resulting in wider cracks through the core of the beam than on the sides. All crack width measurement data are available as part of the complete set of test data at DOI 10.17603/DS2SQ2K.

4.1.2 Relationship between total residual crack width and residual deformations

The total residual crack width of a plastic hinge is theoretically dependent on the residual elongation and rotation of the hinge. Figure 4-1 illustrates the relationship between total residual crack width and elongation for four residual deformation states a plastic hinge can be in following an earthquake. When no residual rotation is present, the two sides have identical total residual crack widths equalling the elongation of the hinge centroid. When residual rotation is present, one side has a larger total residual crack width to account for the residual longitudinal strain gradient across
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the cross-section. In this case, the total residual crack width on the tension side is greater than the elongation of the hinge centroid. The ratio of total residual crack width on the tension side to elongation reduces if the cracks are unable to fully close in compression. For example, in Figure 4-1, the ratio of total residual crack width on Side ‘a’ to total elongation will be larger for case (c) than case (d).

![Diagram](image)

Figure 4-1: Relationship between total residual crack width and the residual deformations of a reinforced concrete plastic hinge

Figure 4-2 shows total and maximum residual crack widths on the tension side of the beams for specimens CYC, CYC-LER, CYC-ER, and MONO. These four specimens were chosen for inclusion due to their varying drift versus elongation responses, which are included in Figure 4-2. The relationship between total crack width and elongation is readily apparent. The total crack widths for specimen CYC were on the order of twice the values measured in the other three
specimens. In specimens CYC-ER & MONO, the total crack widths grew to exceed the elongation by factors of 1.5-2 due to the compression side having limited longitudinal strain [the case illustrated by Figure 4-1(c)]. Comparatively, in specimens CYC-LER, and CYC, the ratio of total crack width to elongation was less than 1.5 due to residual elongation in the compression steel [the case illustrated by Figure 4-1(d)]. These data validate the interdependence of residual deformations and total crack width previously introduced in Figure 4-1.

Figure 4-2: Residual crack widths (total and maximum) on tension side of beams versus elongation for selected specimens (data is up to the point of cover concrete delamination)

4.1.3 Relationship between maximum and total residual crack width

Maximum residual crack width is a simpler and more commonly reported metric than total residual crack width, but is not as readily related to hinge-level deformations. The data presented in Figure 4-2 demonstrate that the maximum residual crack width also tends to increase with elongation, but with a more unpredictable, non-linear progression than the total residual crack width. The non-
linear relationship between maximum crack width and total crack width arises as a result of the formation of additional primary cracks (where primary crack here means a crack at which the longitudinal reinforcement has yielded). The formation of primary cracks in flexural members occurs when the bending moment is large enough to cause the tension force transferred across a crack to exceed the yield strength of the longitudinal reinforcement. After the first primary crack forms, additional primary cracks will develop if the bending moment becomes large enough to exceed the yield strength of the longitudinal reinforcement at another location. Formation of additional primary cracks is dependent on the gradient of the bending moment diagram and the strain hardening properties of the longitudinal reinforcement at the first primary crack.

4.1.4 Residual crack widths versus peak deformation demands

The dependence of total crack width on elongation resulted in specimens with similar residual elongations having similar residual crack widths, regardless of the load history of the specimens. Figure 4-3 shows the total residual crack widths, maximum residual crack widths, and elongation versus drift responses for specimens LD-1 and P-2. The residual cracks measured following the initial earthquake loading are shown in red. When these post-earthquake cracks were measured, LD-1 had been pushed to a peak drift of 1.36%, and had a residual elongation of 6.5mm and a residual drift of 0.18%; P-2 had been pushed to a peak drift of 2.17%, and had a residual elongation of 5.9mm and a residual drift of 0.67%. Despite the differences in peak drift demand during the initial earthquake loading, very similar residual crack measurements (both total and maximum) were obtained for the two specimens on the tension side (6.4mm total, 3.0mm maximum for LD-1 versus 6.0mm total, 3.0mm maximum for P-2). The total crack width on the compression side was smaller in specimen P-2 than LD-1 (2.2mm versus 3.9mm) due to the larger residual drift causing a steeper gradient in the longitudinal strain across the cross-section.

It is therefore concluded that the total residual crack width can at best inform estimates of the peak demands placed on a plastic hinge during a damaging earthquake to the same degree that the residual deformations can. In plastic hinges that are free to elongate, the magnitude of residual elongation is dependent on both the amplitude between the peak positive and negative drifts and the number of loading cycles, and is not an accurate metric for estimating the peak drift demands previously incurred (as previously demonstrated in Section 3.3). Figure 4-3 shows how an earthquake with a larger number of cycles but a lower peak drift (LD-1) can cause larger residual elongation and therefore a larger total residual crack width than an earthquake with a smaller number of cycles but a larger peak drift (P-2). It is important to note that this is a different result than would be obtained if only analysing data from reversed-cyclic or monotonic tests (as is common in most simulated seismic loading studies), where an approximately linear relationship between total residual crack width and peak drift demand exists (see Figure 4-2).
Figure 4-3: Residual crack widths (total and maximum) versus elongation for specimens LD-1 and P-2 (data is up to the point of cover concrete delamination). Measurements taken after the initial earthquake loading (on both sides of beam) are shown in red; all other measurements taken during cyclic loading (on tension side of beam) are shown in black.

In cases where some restraint to elongation is present, the usefulness of the total residual crack width to provide information about the prior demands on the hinge is further convoluted. In columns with substantial axial load, only hairline residual cracks might be present even if a plastic hinge has formed, as the axial load can force the cracks to fully close if the column returns to its original position. This is supported by the significant scatter between the residual crack width and the
maximum measured strain at the crack location reported in the bridge columns tested by Lehman et al. [30]. In beams subjected to identical drift histories, different magnitudes of residual elongation (and therefore residual crack widths) can be present depending on the location of the beam within the structure due to varying interaction with floor systems or columns, as previously discussed in Section 3.3. Figure 4-4 schematically illustrates the variance in drift versus elongation behaviours for plastic hinges in (a) beams with negligible restraint to elongation, (b) beams that are partially restrained by floor systems, and (c) columns with sufficient axial compression to prevent residual elongation.

The discussion thus far has focussed primarily on total residual crack width. However, the non-linear relationship between maximum and total crack width, previously discussed in Section 4.1.3, indicates that the identified limitations with regards to the ability of total residual crack width to inform estimates of the prior peak deformation demands apply equally or more so to maximum residual crack width.

Figure 4-5 shows plots of (a) the number of residual cracks wider than 0.2mm versus drift demand, and (b) the ratio of maximum to total residual crack width versus drift demand (note that in specimens P-2-S and CYC-NOEQ residual crack widths were not measured and the data instead correspond to peak crack widths). The data corresponding to crack measurements following initial earthquake loadings are shown in red. The number of cracks trends upwards with drift demand, reaching as high as 9 but mostly peaking at 5 to 8. The ratio of maximum to total crack width trends downwards, ranging from 1.0 (indicating a single crack wider than 0.2mm) at drifts less than 0.81% to lower than 0.3 at drifts upwards of 2.0%. These trends indicate that the formation of distributed non-hairline cracks was primarily a function of the imposed drift demand. Substantial scatter was observed between the different specimens, but limited trends due to other test variables (e.g. axial restraint, loading rate, load history) could be identified. The number of cracks that developed during
initial earthquake loadings tended to be lower than average for a given drift demand, possibly due to loading rate effects. However, this lower number of cracks did not necessarily result in an increased maximum to total crack width ratio. The largest residual crack width measured prior to formation of a second primary crack was 2.5mm (measured on specimen MONO at 0.81% drift).

Figure 4.5: (a) Number of residual cracks on beam side wider than 0.2mm versus peak drift, and (b) ratio of maximum to total residual crack width versus peak drift.
The ability of these ‘distribution of cracking’ metrics to inform estimates of the peak drift demand is explored in Figure 4-6. Least-squares linear regression models, and the corresponding 5th and 95th percentile prediction intervals (assuming the standard error of the prediction is normally distributed), are overlaid on the data previously presented in Figure 4-5. Figure 4-6(a) shows the absolute value of the drift demand versus the number of residual cracks wider than 0.2mm and Figure 4-6(b) shows the absolute value of the drift demand versus the ratio of maximum to total residual crack width. Only the data corresponding to multiple cracks (i.e. number of cracks > 1 and ratio < 1.0) were input to the regression model to prevent the non-linearity of the data in the single-crack region from overly affecting the linear model. This resulted in best-fit lines corresponding to an increase in the number of cracks of approximately 2 and a reduction in the ratio of maximum to total residual crack width of approximately 0.2 for every additional 1% drift demand. The predictive abilities of the best-fit lines are relatively poor, with coefficient of determination values of 0.56 and 0.47 for the number of cracks and ratio of maximum to total crack width metrics, respectively.

The high degree of dispersion in the data, and the fact that increased dispersion would be expected if test specimens with other detailing or configuration were included, means that these metrics capturing the distribution of non-hairline residual cracks cannot be expected to give a reliable estimate of the peak deformation demand placed on beams during a damaging earthquake. However, these metrics may be useful for roughly approximating an upper-bound limit on the peak drift demands incurred (e.g. based on the 95th percentile prediction interval). The number of non-hairline cracks is the preferred metric due to its simplicity and higher coefficient of determination, based on the data of Figure 4-6.

Given the uncertainty, it may be more appropriate to discuss the information that can be obtained from the number of non-hairline residual cracks in qualitative terms. The occurrence of only a single non-hairline residual crack can indicate that a beam plastic hinge was only subjected to a modest drift demand. In the test specimens here discussed, a drift demand of 0.8% was sufficient to induce a secondary non-hairline crack in all cases (Figure 4-5). Conversely, a number of non-hairline cracks distributed along a plastic hinge, such as the example from the 2011 Christchurch earthquake shown in Figure 4-7, can indicate that a beam has been subjected to a more significant inelastic excursion (as demonstrated by the trends shown in Figure 4-6).

It should be noted that this relationship between the distribution of non-hairline residual cracks and the deformation demand is not applicable for columns, where cracks are likely to close entirely if the hinge returns to a state of zero rotation. In beams, the residual elongation is normally comparable to the peak elongation and therefore cracks are unlikely to completely close (Figure 4-4). It may be possible to infer the demands incurred in the columns of moment frames indirectly, based on the estimated demands in the beams and an assumed lateral mechanism of the frame. Further system-level research investigating this potential relationship is required.
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Figure 4-6: Linear regression models and prediction intervals overlaid on (a) number of residual cracks on beam side wider than 0.2mm versus absolute value of peak drift, and (b) ratio of maximum to total residual crack width versus absolute value of peak drift.
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Concerns were raised following the Canterbury Earthquakes of 2010-2011 due to repeated observations of a limited number of wide cracks in reinforced concrete plastic hinges [10], such as that shown in Figure 4-8. The concerns were primarily focussed on the longitudinal reinforcement spanning across the cracks being subjected to large local strains, which could cause the rotation capacity of the hinge to be less than the design value. However, the Canterbury Earthquakes Royal Commission [11] concluded that the occurrence of only a single wide residual crack in a plastic hinge zone does not in and of itself indicate a shorter than expected plastic hinge length and an associated reduction in deformation capacity. They stated that sufficient rotation demands must be applied to a hinge to induce strain hardening in the longitudinal reinforcement and allow spreading of the yield zone to occur, and it is likely that many of the hinges observed to have only a single large crack did not experience large enough rotations to force additional cracks to develop. The experimental results presented in Figure 4-5 and Figure 4-6 support that conclusion.

Low reinforcement ratios were identified as an important parameter in causing limited primary cracks to form in reinforced concrete walls during the Canterbury earthquakes [33, 108]. The beam specimens tested in this experimental program had a longitudinal reinforcement ratio of 0.6%, which is approximately 1.25 times the minimum requirement in NZS 3101:2006 and relatively low compared to common practice. These results therefore demonstrate that an increase in distributed cracking with drift demand occurs even in lightly reinforced beams, as long as the minimum reinforcement requirement is met.

4.1.5 Residual crack widths versus in-situ plastic hinge length

Concerns were raised following the Canterbury Earthquakes of 2010-2011 due to repeated observations of a limited number of wide cracks in reinforced concrete plastic hinges [10], such as that shown in Figure 4-8. The concerns were primarily focussed on the longitudinal reinforcement spanning across the cracks being subjected to large local strains, which could cause the rotation capacity of the hinge to be less than the design value. However, the Canterbury Earthquakes Royal Commission [11] concluded that the occurrence of only a single wide residual crack in a plastic hinge zone does not in and of itself indicate a shorter than expected plastic hinge length and an associated reduction in deformation capacity. They stated that sufficient rotation demands must be applied to a hinge to induce strain hardening in the longitudinal reinforcement and allow spreading of the yield zone to occur, and it is likely that many of the hinges observed to have only a single large crack did not experience large enough rotations to force additional cracks to develop. The experimental results presented in Figure 4-5 and Figure 4-6 support that conclusion.

Low reinforcement ratios were identified as an important parameter in causing limited primary cracks to form in reinforced concrete walls during the Canterbury earthquakes [33, 108]. The beam specimens tested in this experimental program had a longitudinal reinforcement ratio of 0.6%, which is approximately 1.25 times the minimum requirement in NZS 3101:2006 and relatively low compared to common practice. These results therefore demonstrate that an increase in distributed cracking with drift demand occurs even in lightly reinforced beams, as long as the minimum reinforcement requirement is met.
It is here recommended that the observation of a single non-hairline residual crack, or similar limited cracking, only be treated as evidence of a short in-situ plastic hinge length in situations where such behaviour would otherwise be expected (e.g. when the cracking moment is higher than the yield moment, or due to curtailing of reinforcement, as discussed in Opabola et al. [109]). For typical plastic hinge zones with ductile detailing, it is recommended that the residual capacity of members be assessed assuming standard equivalent plastic hinge lengths (e.g. $0.08L + 0.022d_fy$, [105]), regardless of the observed cracking pattern.

### 4.1.6 Residual crack widths versus residual stiffness

In Figure 4-9, the total and maximum residual crack width data are compared against the reloading stiffness of the test specimens. An algorithm was used to extract the reloading stiffness values for each half cycle during the standard cyclic loading portion of the tests. The reloading stiffness is taken as the secant line between the residual deformation (the deformation at which the crack widths were measured) and the first instance of reaching the nominal moment strength $M_n$, which corresponds to a shear force of 95.3kN in the test specimens. The reloading stiffness is expressed in terms of both kN/mm and as a ratio of $E_cI_g$, where $E_c$ was calculated as $4700\sqrt{f'c}$ [1], and $f'c$ was taken as 30MPa. The data corresponding to the first half cycle following the initial earthquake loadings are shown in red.
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Figure 4-9: Reloading stiffness degradation during cyclic loading up to 2.5% drift versus (a) total residual crack width, and (b) maximum residual crack width
The residual crack width metrics, and particularly the total crack width, are highly correlated with the reloading stiffness of all cyclic specimens, regardless of the prior loading protocol or the level of axial restraint. The degraded reloading stiffness values decay exponentially with increasing total residual crack width, and approach an asymptote of approximately 1.3\(kN/mm\), or around 10\% of the initial secant stiffness to yield. However, this relationship does not hold for the reloading stiffness immediately after initial earthquake loadings, where higher stiffness values for a given crack width were observed. The specimens subjected to initial earthquake loadings immediately began exhibiting reloading stiffness versus residual crack width relationships similar to those of the cyclic specimens upon the subsequent load reversal (i.e. on the second half cycle after the onset of cyclic loading).

The implications of the data of Figure 4-9 with regards to post-earthquake assessment of residual stiffness is further explored in Section 4.3.

4.1.7 Residual crack widths versus residual strength

Standard flexural theory for determining the moment capacity of ductile reinforced concrete components accounts for cracked section behaviour and the longitudinal reinforcement reaching its yield strength. Wide residual cracks are merely evidence that the section has exhibited its intended behaviour, via yielding of the longitudinal reinforcement. Residual crack width metrics are therefore here considered to have no effect on the residual design strength (i.e. the nominal strength) of a ductile plastic hinge.

Residual crack width metrics are also here considered to be largely unrelated to the residual plastic hinge overstrength. However, the presence of non-hairline residual crack widths can be a useful indicator that yielding has occurred in the longitudinal reinforcement, which has the potential to affect the overstrength due to strain ageing effects. Further analysis on the effects of prior longitudinal reinforcement strain on the overstrength of plastic hinges is given in Chapter 5.

4.1.8 Residual crack widths versus residual deformation capacity

The analysis of Section 3.1 showed that the deformation capacities of the beam specimens were unrelated with the number of moderate level (<2.17\% drift) loading cycles. This implies that residual crack width metrics, which were only measurable up to moderate drift demands, were also unrelated to the deformation capacity. This is explicitly shown in Figure 4-10, where it can be seen that no correlation existed between the four residual crack width metrics considered in this section, as measured after all loading cycles at or below 2.17\% drift, and the deformation capacities of the beam specimens. Note that data from the axially restrained specimens were not included, as all restrained specimens had identical displacement capacities.
It was shown in Section 2.7.7 that more concentrated damage was correlated with lower deformation capacities in the unrestrained test specimens. However, Figure 4-10 shows that this is not evident when using either the ratio of maximum to total crack width or the number of non-hairline cracks as metrics for ‘concentrated damage’. This is likely due to the residual crack width data corresponding to flexural cracks on the beam sides, while the ‘concentrated damage’ referred to in Section 2.7.7 is based on the number and location of the sliding plane cracks that developed. Figure 4-11 shows an example of multiple flexural cracks propagating together to form a single large sliding plane crack in specimen P-2. It is concluded that, with regards to flexural cracking, the observation of low numbers of wide residual cracks or high maximum to total residual crack width ratios cannot confidently be linked with a concentration in damage at the ultimate limit state and any associated reduction in deformation capacity. Such observations may instead simply be a
result of limited deformation demands during the damaging earthquake, as previously discussed in Section 4.1.5.

Figure 4-11: Example photograph of multiple flexural cracks on tension side propagating together to form a single sliding plane crack (specimen P-2)

4.1.9 Summary

Limitations with the use of residual crack widths as damage metrics were identified. Based on these limitations, it is here concluded that less importance should be placed on residual crack widths than in previous post-earthquake assessment guidelines [16, 20]. The primary issues with the efficacy of residual crack widths for informing post-earthquake assessments are as follows:

- Residual crack widths are dependent on the residual deformations of the component being considered, which are in turn dependent on the load history, axial load, and level of restraint against axial elongation.
- Residual elongation and residual drift (and by extension, residual crack widths) are not always indicative of the peak deformation demands imposed on a plastic hinge during a damaging earthquake.
- Crack width data taken from standard cyclic tests may show trends that are artefacts of the loading protocols rather than causal relationships.
- Residual crack widths are not necessarily indicative of the residual strength or residual deformation capacity of reinforced concrete plastic hinges.

Despite these limitations, it is not here suggested that residual cracks be entirely ignored as a damage indicator, but that the importance of the exact magnitudes of crack widths be treated with scepticism. It is recommended that the number of non-hairline residual cracks in beam plastic hinge zones be used as a metric to roughly estimate the deformation demands incurred during the damaging earthquake, with a single crack indicating a modest deformation demand and numerous cracks indicating a more severe deformation demand. Due to the considerable variability, a
conservative model, such as the 95th percentile prediction interval shown in Figure 4-6(a), is recommended for the purposes of attempting to quantify deformation demand based on the number of non-hairline residual cracks.

It is also here concluded that post-earthquake observations of limited numbers of wide cracks do not necessarily indicate a short plastic hinge length, but rather an insufficient deformation demand to cause additional cracks to open. A short plastic hinge length should be assumed only in cases where other factors that may cause limited cracking are present (e.g. a cracking moment above the yield moment, or curtailment of longitudinal reinforcement).

Total residual crack width was highly correlated with the reloading stiffness of the experimental program beam specimens during cyclic loading. However, this correlation was not as strong following initial earthquake-type loadings, and additional factors must be considered when assessing residual stiffness. The residual stiffness of earthquake-damaged reinforced concrete plastic hinges is further discussed in Section 4.3.

It is emphasised that these findings regarding residual crack width metrics are targeted at considerations of flexural hinging behaviour. Other deformation mechanisms can be affected by the maximum crack width. In the experimental program conducted as part of this work, large cracks through the depth of the beams allowed significant shear sliding deformations to occur. This type of cracking is discussed in further detail in Section 4.2.2.

### 4.2 The implications of other types of moderate damage

In addition to the flexural cracking discussed in Section 4.1, ductile reinforced concrete plastic hinges can exhibit a variety of other damage states that are here classified as ‘moderate’ (see Section 1.4). These include longitudinal cracking, sliding plane cracks, and cover concrete spalling. The following sections briefly discuss the implications of these types of damage.

#### 4.2.1 Longitudinal cracking

Longitudinal cracking can occur due to either high compression strains or bond slip of the longitudinal reinforcement [16]. Compression-induced longitudinal splitting cracks in the cover concrete can be considered a precursor to cover concrete spalling, which is instead discussed in Section 4.2.3. Longitudinal cracks that occur at the location of the longitudinal reinforcement, and are therefore indicative of bond degradation between the longitudinal reinforcement and surrounding concrete, are the focus of this discussion.

In well-detailed plastic hinge zones that are not controlled by lap splice or anchorage failure modes, longitudinal cracking is a relatively minor damage state. Degradation of bond in the plastic hinge
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zone is necessary for the spreading of yielding along the longitudinal reinforcement. In the beams
of the experimental program described in Chapter 2, longitudinal cracking developed in all
specimens after being subjected to peak drifts in the range of 1.0 to 1.5% (further discussed in
Appendix B).

The primary effect that bond degradation has on the load-deformation response of a ductile plastic
hinge with adequate anchorage is stiffness degradation, due to lower strains being induced in the
longitudinal reinforcement for a given deformation demand. Due to this limited impact, explicitly
accounting for longitudinal cracking when assessing the residual capacity of moderately damaged
plastic hinges is here considered unnecessary. The effects of longitudinal cracking on stiffness
degradation are instead indirectly included in the analysis of Section 4.3.

Older reinforced concrete moment frames may have beam longitudinal reinforcement that passes
through interior beam-column joints, without sufficient joint depths to ensure adequate bond
transfer under repeated seismic loadings (e.g. joint depths less than 20 times the diameter of the
largest longitudinal beam bar, the minimum depth stipulated in ACI 318-14 [2]). In these cases,
bond degradation may have more serious implications for the residual capacity of the beam-column
assembly, including severe stiffness degradation to the point where extreme deformation demands
would be required for the beam to reach its design strength [44]. However, plastic hinge visual
damage indicators such as longitudinal cracking are not necessarily indicative of the degree of bond
degradation that has occurred within a beam-column joint. Further research is recommended to
investigate how to identify whether significant bond degradation throughout the depth of a beam-
column joint has occurred, and how to assess the residual capacity in such situations.

The focus of this discussion was primarily on hairline longitudinal cracks. Wide longitudinal cracks
may intersect with flexural cracks and cause delamination of large pieces of cover concrete, which
was one of the causes of spalling in the beams of the experimental program. The implications of
this type of spalling are discussed in Section 4.2.3.

4.2.2 Sliding plane cracks

In this work, a sliding plane crack is defined as any approximately transverse crack that passes
through the depth of a member, has sufficient width to limit shear transfer through aggregate
interlock, and does not cross any transverse reinforcement. Shear sliding deformations can cause
substantial stiffness degradation, and were the primary cause of the pinching behaviour observed
in the hysteretic load-deformation responses of the experimental program beam specimens (see
Figure 2-19). Shear sliding also necessitates shear transfer to occur through dowel action of the
longitudinal reinforcement, which can initiate reinforcement buckling or cover concrete spalling.
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(further discussed in Section 4.2.3). The prevalence of shear sliding is significantly reduced in plastic hinges subjected to axial compression (see Section 3.3).

Sliding shear behaviour in elongating reinforced concrete plastic hinges is not readily predictable, given the current state of knowledge. NZS 3101:2006 (based on the work of Paulay and Priestley [105]) requires that the maximum plastic hinge shear force be compared against a critical value, and that diagonal reinforcement be used to resist sliding shear actions when the critical value is exceeded. However, the beams of the experimental program were subjected to maximum shear forces of less than 50% of this critical value, yet nonetheless exhibited substantial sliding shear behaviour. The low shear stresses may have actually contributed to early formation of transverse sliding plane cracks, as higher shear stresses would have caused the flexural cracks to become more inclined as they propagated towards the beam centroid, due to rotation of the direction of principal tensile stress.

In post-earthquake situations, there is the advantage of being able to visually observe whether or not a sliding plane crack has formed. In such cases, it may be possible to better predict the future likelihood of shear sliding based on measurements of sliding plane crack widths. The approach to calculating shear transfer due to aggregate interlock that is used in the modified compression field theory [110], which builds on work by Walraven [111], explicitly considers crack width (although it is noted that the shear resistance of elongating plastic hinges is outside the domain for which this equation was originally developed). In the case of no compressive force perpendicular to the crack, the shear stress across the crack is given by Equation (4-1).

\[
v = \frac{0.18\sqrt{f'_c}}{0.31 + \frac{24w}{a_g + 16}} \quad (4-1)
\]

where \(w\) is the average crack width along the length of the crack (mm), \(a_g\) is the maximum aggregate diameter (mm), and \(v\) is the shear stress across the crack (MPa).

In Figure 4-12, the shear stress versus crack width relationship of Equation (4-1) (using \(a_g = 20\) mm and \(f'_c = 30\) MPa) is compared against the maximum shear stress sustained by the beam specimens of the experimental program (calculated based on an approximate maximum shear force of 120kN and the gross sectional area). The point where Equation (4-1) intersects the shear stress demand (~2.5mm crack width) may be representative of the point at which aggregate interlock shear transfer was unable to prevent significant sliding shear deformations from occurring.

Sliding plane crack widths cannot directly be compared against shear deformation data from the experimental program, as the widths of sliding plane cracks were not measured during testing. However, assuming that the elongation at the beam centroids was primarily concentrated at the
sliding plane cracks, which is supported by the photographic evidence shown in Appendix B, the beam elongation data can be used as a proxy metric for the sliding plane crack widths. Figure 4-13 shows the beam elongation versus shear deformation data for two sets of specimens that were subjected to identical loading protocols (except for differences in loading rate). In all specimens, the magnitude of shear deformation increased with beam elongation. At higher beam elongations, the specimens which exhibited concentration of damage around a single sliding plane crack had higher shear deformations than equivalent specimens that exhibited more distributed cracking. The elongation at which the shear deformation starts to increase in the specimens with a single sliding plane crack, relative to those with multiple sliding plane cracks, approximately corresponds to the crack width of the intersection point from Figure 4-12. Damage states corresponding to this point are shown in Figure 4-14 for specimens CYC and CYC-DYN. It can be seen that the elongation in specimen CYC-DYN was predominantly concentrated in an approximately transverse crack, while the elongation in specimen CYC was spread across multiple cracks.

![Figure 4-12: Comparison of Equation (4-1) with beam specimen maximum shear stress demands](image)

The data of Figure 4-13 support the use of Equation (4-1) for determining the crack width at which shear transfer through aggregate interlock begins to significantly reduce. Equation (4-1) can therefore be considered to provide a simple and quantifiable method for calculating a critical sliding plane crack width in post-earthquake assessments. However, there are considerable limitations with this procedure:

- There is no guarantee that a sliding plane crack with a width below this critical value will not simply grow wider upon reloading, and have a shear sliding response develop.
- The location of the sliding plane crack may influence the degree of impact it has on the plastic hinge performance (see Section 2.7.7). In the experimental program, sliding plane cracks located at the beam-foundation interface had a limited impact on the performance
of the test specimens, possibly due to the confinement effect of the foundation (although this effect may not be as significant in beam-column joints).

- A lack of approximately transverse cracks does not necessarily preclude sliding shear deformations. A similar ‘sliding’ deformation mechanism can occur (in one loading direction) on diagonal cracks that do not cross transverse reinforcement, as shown in Figure 4-15.

Despite these limitations, it is believed that a method for identifying potential sliding plane cracks in post-earthquake situations is needed, as concentrated sliding shear deformations have been associated with relatively poor plastic hinge performance, both in this test program (Section 2.7.7), and others (e.g. [112]).

![Graph showing shear deformation versus elongation relationship for unrestrained beam specimen sets subjected to identical loading protocols (data up to 2.17% drift)](Image)

*Figure 4-13: Shear deformation versus elongation relationship for unrestrained beam specimen sets subjected to identical loading protocols (data up to 2.17% drift)*

The influence that sliding plane cracks have on the residual capacity of plastic hinges is difficult to quantify. In FEMA 306 [16], the occurrence of sliding plane cracks is considered to be a relatively severe damage state. Ductile reinforced concrete coupling beams that exhibit sliding plane cracks are considered to have residual stiffness, strength, and deformation capacity values that are 20%, 30%, and 70% of equivalent undamaged components, respectively. In ductile reinforced concrete walls, the corresponding values are 40%, 50%, and 80%. FEMA 306 defines sliding plane cracks...
as a major transverse cracks through the member depth, with some degradation of concrete along the crack and a possible small lateral offset at the crack.

![CYC and CYC-DYN images](image)

**Figure 4-14**: Damage states of specimens CYC and CYC-DYN after reaching a beam elongation corresponding to the intersection point from Figure 4-12

![Relative movement across a 'diagonal' crack in specimen P-1 images](image)

**Figure 4-15**: Relative movement across a 'diagonal' crack in specimen P-1

The residual strength reduction factor used in FEMA 306 is here considered to be overly conservative. Figure 4-16(a) shows the damage states of four of the beam specimens from the experimental program, immediately prior to being subjected to a loading cycle at a drift demand of 2.44%. Three of the specimens (LD-2, LD-2-S, and P-2) exhibit damage consistent with the definition of 'sliding plane crack' used here (with the crack at the location which was found to be most destructive, see Section 2.7.7), while the fourth specimen (CYC-NOEQ) is completely undamaged. The hysteretic behaviour of these four specimens during the subsequent loading cycle is shown in Figure 4-16(b). The only noteworthy difference between the load-deformation responses of the various specimens is a lower stiffness in the specimens exhibiting the sliding plane cracks. (Direct analysis with regards to the effects of sliding plane cracks on residual stiffness is
not recommended – see Section 4.3.) The peak strengths reached by all of the specimens are approximately equal. It is therefore concluded that the observation of a sliding plane crack alone, i.e. without being accompanied by more severe damage states such as longitudinal reinforcement buckling or degradation of core concrete, does not indicate a reduced flexural strength.

Figure 4-16: (a) Damage states of selected specimens immediately prior to 2.44% drift loading cycle, and (b) hysteretic behaviour during 2.44% drift loading cycle
It was previously shown in Table 2-9 that concentration of damage around a single sliding plane crack was correlated with a lower deformation capacity in the beam specimens of the experimental program. In order to derive an approximate post-earthquake deformation capacity reduction factor for specimens exhibiting sliding plane cracks, the average deformation capacity of the specimens exhibiting a single sliding plane crack (3.4% drift) is divided by the average deformation capacity of all other specimens (4.3% drift). This results in a reduction factor of 0.79, or approximately 0.8. It is therefore recommended that, if a transverse crack through the depth of a plastic hinge is observed in a post-earthquake situation, and it has a width greater than that given by the intersection of Equation (4-1) with the probable shear stress demand (see Figure 4-12), the residual deformation capacity of that plastic hinge be taken as 0.8 of what would be calculated for an equivalent undamaged component. While further research is required to validate this recommendation, it is expected to be a conservative methodology for the following reasons:

i. It assumes that the observation of a sliding plane crack in a post-earthquake situation means that future damage will concentrate around that crack, and

ii. All beams of the experimental program, even those exhibiting a single sliding plane crack, had deformation capacities exceeding acceptance criteria for the most ductile classification of reinforced concrete beams in modern standards (e.g. NZS 3101:2006, ASCE 41-17), and

iii. Specimens with axial restraint were not included in this analysis. Restraint of elongation, and possible direct restraint against sliding due to slab shear stresses, may limit shear sliding deformations in moment frame buildings.

4.2.3 Cover concrete spalling

Example photographs of cover spalling in reinforced concrete columns, both from post-earthquake field investigations and laboratory testing programs [113], are shown in Figure 4-17. Ductile plastic hinge zones in columns are often designed with the intent that the cover concrete crushes before the ultimate limit state is reached, and methods for direct consideration of the effects of cover concrete spalling are therefore well established. The New Zealand seismic assessment guidelines [29] permit probable curvature capacities to be calculated based on either the cover concrete crushing at an assumed maximum strain of 0.004, or the core concrete crushing at a maximum strain calculated as per a modified version of the Mander et al. [46] confined concrete model. The strain corresponding to crushing of the confined core concrete typically governs the assessed deformation capacity of ductile plastic hinge zones in columns. This indicates that the occurrence of cover concrete spalling does not necessarily cause any reduction in the residual deformation capacity of well-confined ductile reinforced concrete columns. Reductions in flexural strength and stiffness may occur after cover concrete spalling, due to a reduction in the effective depth of the section, but this effect is limited for typical sections with small cover concrete depths. Direct consideration of cover concrete
spalling when assessing the residual capacity of moderately damaged plastic hinges in reinforced concrete columns is therefore not considered necessary.

The observation of columns with spalled cover concrete may be indicative of a relatively high deformation demand during the damaging earthquake. Berry and Eberhard [114] used a database of 102 rectangular and 40 spiral-reinforced flexure-dominant reinforced column tests to investigate trends in the deformation demand at the onset of cover spalling. The mean drift ratios at the onset of spalling were 1.53% and 2.30% for rectangular and spiral columns, respectively. The mean displacement ductility values, calculated using a semi-empirical yield deformation defined by Berry and Eberhard, were 1.8 and 2.3, respectively. High coefficients of variation of nearly 0.5 were observed for both the drift demand and displacement ductility at the onset of spalling for both the rectangular and spiral datasets.

Despite this high variability, the data show that spalling generally occurs at displacement ductility values of greater than one. Figure 4-18 shows the normal and lognormal cumulative distribution functions of the displacement ductility at the onset of spalling for both the rectangular and spiral column datasets. The cumulative distribution function parameters were determined based on the sample mean and standard deviation provided by Berry and Eberhard. Assuming these data are representative of all ductile columns, Figure 4-18 shows that over 80% of rectangular columns and over 90% of spiral-reinforced columns do not exhibit spalling until after yielding occurs. It is acknowledged that the yield point in reinforced concrete columns is not readily identified and the results presented in Figure 4-18 are therefore partially dependent on the definition of yield used by Berry and Eberhard. Nonetheless, it is here recommended that the observation of column cover
concrete spalling in post-earthquake situations can generally be taken as an indication that ‘yielding’ has occurred. Such an indicator is needed for reinforced concrete columns, as flexural crack widths that could otherwise be used as indicators of yielding may close due to gravity-induced axial compression forces (Section 4.1). However, it is important to note that the reverse is not true, i.e. a lack of observed cover concrete spalling does not necessarily indicate a lack of yielding in the column.

![Figure 4-18: Cumulative distribution functions of displacement ductility at onset of spalling for reinforced concrete columns (data taken from Berry and Eberhard [114])](image)

Certain aspects related to the post-earthquake observation of cover concrete spalling in columns warrant further research. Spalling is an indicator that strains beyond the crushing strain occurred in the cover concrete, which may also mean that relatively high strain demands were induced in the core concrete. Any degradation of the stress-strain behaviour of the core concrete that may occur as a result of such strain demands is neglected in this work (provided that no crushed or loose core concrete is observed, which would indicate the column is outside of the definition of ‘moderate’ damage used here). High compressive strains in the core concrete may also induce substantial confinement-related stresses in the transverse reinforcement, but any effect this may have on residual capacity is here neglected.

Crushing of cover concrete may also initiate spalling in reinforced concrete beams. However, spalling of cover concrete in ductile beams may be more likely to occur as a result of delamination due to the intersection of longitudinal cracks with flexural cracks, or due to shear deformations and
the associated dowel action of longitudinal reinforcement. These were the causes of spalling in the beams of the experimental program (e.g. as shown in Figure 2-18). This type of spalling is correlated with the onset of buckling in longitudinal reinforcement. In the beam specimens of the experimental program, some degree of longitudinal reinforcement buckling was typically observed in the first cycle in which the reinforcement was visible. Minor buckling behind partially delaminated cover concrete may have been one of the contributing factors in initiating spalling, although this cannot be verified as the reinforcement could not be observed prior to spalling. A lack of prior longitudinal reinforcement buckling is a key distinction in the analysis of Chapter 5. It is recommended that careful inspection of the longitudinal reinforcement be undertaken in cases of spalled or delaminated cover concrete. Loose cover concrete material should be removed to facilitate such an inspection. However, previously buckled longitudinal reinforcement can straighten upon load reversal, and therefore a lack of observed buckling does not necessarily guarantee a lack of prior buckling. The findings of Chapter 5 that are conditional on there being no evidence of prior longitudinal reinforcement buckling may not be applicable in situations where delamination or spalling makes such a determination uncertain. Further research is recommended.

4.2.4 Summary

The following list summarizes the key recommendations on how to account for longitudinal cracking, sliding plane cracks, or cover concrete spalling when assessing the residual capacity of a ductile plastic hinge zone:

- These types of damage can all result in stiffness degradation, but explicit consideration of the damage is not recommended for the purposes of assessing residual stiffness. Further information is given in Section 4.3.
- Provided that there is adequate anchorage for longitudinal reinforcement, longitudinal cracking is a relatively minor damage state that need not be accounted for when assessing the residual strength or deformation capacity of ductile plastic hinge zones.
- Sliding plane cracks alone, i.e. without being accompanied by core concrete degradation or longitudinal reinforcement buckling, do not necessarily cause a reduction in flexural strength (Figure 4-16).
- If significant shear deformations concentrate at a single sliding plane crack in a plastic hinge, it can result in a lower deformation capacity than would be observed in an equivalent plastic hinge with more distributed damage. A critical sliding plane crack width can be approximated as a function of the beam shear stress, the concrete strength, and the aggregate size [Equation (4-1)]. It is recommended that, if a sliding plane crack over this critical width is observed in a post-earthquake situation, the residual deformation capacity of that plastic hinge be taken as no greater than 80% of what would be predicted for an
equivalent undamaged component. This recommended procedure has several limitations, and is based primarily on limited data from the experimental program described in Chapter 2. Further research is recommended to validate the appropriateness of this method.

- In ductile reinforced concrete columns, crushing of cover concrete, such as that shown in Figure 4-17, has a limited effect on the residual strength or deformation capacity. Cover concrete spalling can be taken as an indicator that yielding has likely occurred (Figure 4-18), but a lack of cover concrete spalling does not indicate that yielding has not occurred.
- In ductile reinforced concrete beams, spalled or delaminated cover concrete is correlated with the onset of buckling in the longitudinal reinforcement. The analysis of Chapter 5, which focusses on longitudinal reinforcement in which there is no evidence of prior buckling, may not be applicable to beams with severe spalling or delamination.

4.3 Assessing the residual stiffness of moderately damaged plastic hinges

A number of factors contribute to the degradation of stiffness that occurs in reinforced concrete plastic hinges, relative to the initial secant stiffness to yield. The factors previously identified in this chapter include (i) crack closure, (ii) bond degradation, (iii) inelastic shear deformations, and (iv) concrete degradation. The Bauschinger effect in previously yielded longitudinal reinforcement, further discussed in Chapter 5, can also affect the plastic hinge stiffness. Quantifying the effects of each of these factors on the residual stiffness is not feasible for the purposes of a post-earthquake assessment. This section instead discusses a simple method to approximate the residual stiffness of plastic hinges in reinforced concrete moment frames.

4.3.1 Background on modelling of stiffness in reinforced concrete

For the purposes of linear analysis, a single value that represents the secant stiffness to the yield point is often employed to model the lateral stiffness of reinforced concrete components. A variety of methods for determining this stiffness have been proposed, including simple percentages of the gross stiffness \( E_c I_g \) [26], and more complex formulations that account for the contributions of flexural, shear, and bond slip deformations [115].

In non-linear analyses that incorporate stiffness degradation, distinction is made between unloading and reloading stiffness, and degradation of these two stiffness values is often considered. Unidirectional hysteretic models for reinforced concrete that consider stiffness degradation have existed in the literature for several decades, with some particularly well-known examples being the Clough model [116], the Takeda model [117], and the Ibarra-Medina-Krawinkler model [63]. The Clough model assumes no degradation of unloading stiffness and a peak-oriented reloading stiffness. The Takeda model assumes unloading stiffness degradation as a function of the ratio of
yield displacement to peak displacement, a peak-oriented reloading stiffness, and additional rules for load reversals from within the unloading or reloading branches. The Ibarra-Medina-Krawinkler model includes functions for both unloading and reloading stiffness degradation based on the cumulative energy dissipation. A number of modified versions of these models have been proposed, which account for alternate unloading and reloading stiffness degradation rules, multi-linear ‘pinching’ reloading stiffness behaviour, smooth transitions between sections of the hysteretic curve, and many other variations. It is noted that these models combine the various deformation mechanisms of a reinforced concrete plastic hinge into a single hysteretic response. Another modelling technique is to decompose the behaviour into separate flexural, bond slip, and shear models that are then connected in series (e.g. [118, 119]).

A schematic showing the various parameters that are typically used in calculating the instantaneous unloading or reloading stiffness (neglecting pinching behaviour) is shown in Figure 4-19. Note that the parameters shown are not comprehensive, as certain models may make use of other related metrics (e.g. cumulative ductility). The backbone behaviour shown in Figure 4-19 corresponds to an elastic-perfectly-plastic response, but other backbone curves, which may or may not account for strength degradation, are also commonly used.

4.3.2 Special considerations in post-earthquake situations

Figure 4-20(a) shows that the reloading stiffness of the beam specimens was strongly correlated with the difference between the residual drift and the prior peak drift in the reloading direction (i.e. the deformation parameters that define a peak-oriented reloading stiffness model as shown in Figure 4-19). Figure 4-20(b) shows that the unloading stiffness of the beam specimens was strongly correlated with the peak drift immediately prior to unloading (the unloading stiffness was taken as the secant line between this peak value and the residual drift). These results support the use of typical hysteretic stiffness degradation rules, such as that of the Takeda model [117], for the purposes of non-linear analysis.

However, obtaining sufficient knowledge of the deformation history of a component in order to calculate the instantaneous unloading or reloading stiffness as a function of these deformation parameters is not feasible in post-earthquake situations. Instead, a single degraded stiffness parameter that is presented as a fraction of the initial secant stiffness to yield, such as the stiffness reduction factors recommended in FEMA 306 [16], is desirable. This stiffness reduction factor must be a function of simple parameters that can be measured or estimated in post-earthquake situations.
Total residual crack width is a quantifiable visual damage metric that was shown to be correlated with the reloading stiffness of the test specimens (Figure 4-9). However, it was also shown that a lack of non-hairline residual cracks does not necessarily indicate that the component in question was not pushed into the inelastic range. In these situations, using the residual crack widths alone as a metric for predicting the residual stiffness would be inappropriate, as a minimal total residual crack width would imply no reduction in the secant stiffness to yield. Such an assessment would be clearly inaccurate, as factors such as bond degradation, concrete degradation, and the Bauschinger effect can affect the residual stiffness regardless of the residual crack width. Furthermore, spalling of cover concrete can preclude the measurement of cracks, making residual crack widths an unusable metric in certain situations. Residual deformation, the other readily quantifiable damage metric, suffers from similar limitations with regards to its ability to inform predictions of residual stiffness, as a lack of residual drift does not necessarily mean that the component did not exhibit significant plastic deformation and any associated stiffness degradation.
Figure 4-20: Stiffness degradation during cyclic loading up to 2.5% drift versus deformation demand metrics commonly used in hysteretic models: (a) reloading stiffness, and (b) unloading stiffness.

It is therefore concluded that an estimate of the peak deformation demand is the best metric for quantifying residual stiffness in post-earthquake scenarios. This estimate could be either based on
damage observations, or be based on an analytical model of the building (as described in the draft assessment methodology of Appendix C). Using the deformation demand as the primary metric for determining the residual stiffness is distinct from the method employed in FEMA 306 [16], where stiffness reduction factors were primarily based on damage states.

4.3.3 Deriving a residual stiffness factor as a function of deformation demand

Figure 4-21 shows that, when the absolute maximum prior drift demand is used as a deformation metric, the reloading stiffness degradation of the beam specimens during cyclic loading followed a consistent pattern, but this pattern did not hold for the first half cycle immediately following the initial earthquake loadings (similar to the finding for total residual crack width – see Figure 4-9). Following the initial earthquake loadings, the beam specimens exhibited higher reloading stiffness values for a given peak drift than in the cyclic tests. These results indicate that residual stiffness models based on standard reversed cyclic test data may give a conservative lower-bound for the residual stiffness of a reinforced concrete plastic hinge after being subjected to an arbitrary load history with a given maximum deformation demand.

![Figure 4-21: Reloading stiffness degradation during cyclic loading up to 2.5% drift versus peak drift demand](image)
In order to investigate the variability in the residual stiffness of plastic hinges following arbitrary loadings, additional relevant experimental data was required. The following information was deemed necessary in each experiment: (i) an initial arbitrary earthquake-type loading, (ii) a subsequent loading to allow assessment of residual stiffness, and (iii) a limited residual drift (<1.0%) after the initial loading (to prevent the distinction between reloading and unloading stiffness from unduly affecting the measured residual stiffness). Shake table tests that involve the application of multiple earthquake ‘runs’ to a test specimen constitute the primary source of this type of experiment. A dataset of relevant shake table tests on ductile reinforced concrete columns was compiled, consisting of four test programs [120-123], eight specimens, and 34 runs. All specimens were circular or oblong with spiral transverse reinforcement, typical of bridge column construction in seismically active areas of the United States.

Figure 4-22(a) shows the normalized stiffness during a particular run, versus the absolute maximum drift demand reached in any preceding run, for all columns in the dataset. The corresponding data for the beam specimens of the experimental program following initial earthquake loadings are also included in Figure 4-22(a). The stiffness was defined as the secant line between the residual drift and the first instance of reaching 80% of the maximum base moment (i.e. the peak base moment reached during any of the runs for that specimen). The residual stiffness values were normalized against the initial secant stiffness to 80% of the maximum base moment, which is here assumed to approximately correspond to the secant stiffness to yield. If any specimen exhibited more than moderate damage (as defined in Section 1.4) or residual drifts of greater than 1% during a particular run, all future runs on that specimen were omitted from the analysis. Some approximation was required in cases where numerical data was not available, and the normalized stiffness values should therefore only be considered accurate to one significant figure.

The data of Figure 4-22(a) shows an approximately linear trend between the prior maximum drift demand and the normalized residual stiffness of the column specimens, but with significant scatter. However, the stiffness degradation in the beam specimens of the experimental program was considerably higher than in the columns for a given maximum prior drift demand. In Figure 4-22(b), the data of Figure 4-22(a) is transformed to use displacement ductility as a deformation metric. The displacement ductility of the test specimens was determined using a yield rotation calculated from Equation (4-2).

\[ \theta_y = \frac{M_y L}{3E_c I_{eff}} \]  

where \(\theta_y\) is the yield rotation, \(M_y\) is the yield moment (taken as 80% of the maximum base moment for the specimens of the dataset), \(L\) is the shear span length, \(E_c\) is the Young’s Modulus of the concrete (taken as 25,000 MPa in this analysis), and \(I_{eff}\) is the effective moment of inertia, taken as a fraction of the gross
section moment of inertia based on the recommendations of ASCE 41-17 (a 0.3 factor was used in this analysis as all specimens had axial load ratios of approximately 0.1 or below).

- Circular columns - after previous shake table tests
- Beam specimens - after initial earthquake loadings

Figure 4-22: Normalized secant stiffness degradation of reinforced concrete plastic hinges following arbitrary loadings versus (a) maximum prior drift demand, and (b) maximum prior displacement ductility demand
Figure 4-22(b) shows that, when using displacement ductility as a deformation metric, the rate of stiffness degradation in ductile reinforced concrete beams and columns is comparable. It is therefore concluded that, although displacement ductility is more complicated to calculate than the maximum drift demand, it is necessary to use displacement ductility as a metric in order to consistently assess residual stiffness across various components.

Di Ludovico et al. [124] previously used data from standard cyclic tests to derive a stiffness reduction factor for reinforced concrete plastic hinges as a function of the prior displacement ductility demand. The test data came from a database of 23 reinforced concrete columns with deformed longitudinal reinforcement. A hyperbolic trend between the residual stiffness \( (K_r) \) [normalized against the initial secant stiffness to yield \( (K_y/K_y) \)] and the displacement ductility demand \( (\mu) \) was observed, and regression analysis yielded an empirical expression \[ K_r/K_y = 1 - (1.07 - 1.15\mu^{-0.92}) \] that fit the test data with coefficient of determination of 0.92. However, Di Ludovico et al. defined stiffness as the secant line between the peaks of two subsequent half cycles (i.e. peak-to-peak stiffness). In the case of symmetric cyclic loading, the peak-to-peak stiffness is equivalent to the origin-to-peak stiffness. Figure 4-23 shows that the ratio of the origin-to-peak stiffness to the initial secant stiffness to yield in an elastic-perfectly plastic system is equal to the inverse of the ductility demand. The slight differences between the empirically-derived expression proposed by Di Ludovico et al. and this analytical inverse ductility relationship can only be due to variations in the maximum strength of individual cycles (i.e. real plastic hinges are not elastic-perfectly plastic).

\[
\mu = \frac{\Delta_i}{\Delta_y} \quad K_r = \frac{F_y}{\Delta_i} \quad K_y = \frac{F_y}{\Delta_y} \quad \frac{K_r}{K_y} = \frac{\Delta_y}{\Delta_i} = \frac{1}{\mu}
\]

*Figure 4-23: Relationship between origin-to-peak stiffness degradation and ductility demand*
In Figure 4-24(a), the Di Ludovico et al. expression and the $K_r/K_y = 1/\mu$ expression are plotted alongside the test data from Figure 4-22(b). The similarity between the two equations is evident. For the remainder of this section, $K_r/K_y = 1/\mu$ is considered to be the preferred expression due to its simplicity. The equations give conservative predictions of the residual stiffness of the test specimens following arbitrary earthquake-type loadings. The exception to this conservativism is at low displacement ductility demands (less than 2.0). However, this is likely due to the variability involved in accurately quantifying the yield deformation of reinforced concrete columns. Given the uncertainty involved in estimating the displacement ductility in post-earthquake situations, and the fact that the region between 1.0-2.0 displacement ductility contributes to 50% of the overall stiffness reduction in the proposed expression, it is recommended that the stepwise function shown in Figure 4-24 is more appropriate in this low ductility region. Any component that is estimated to have been subjected to a displacement ductility greater than 1.0 should be considered to have a residual stiffness equal to no greater than 50% of its initial secant stiffness to yield. Incorporating this stepwise function at low ductility demands, the complete recommended expression for assessing residual stiffness as a function of displacement ductility becomes as given by Equation (4-3).

$$\frac{K_r}{K_y} = \begin{cases} 1.0, & \mu < 1.0 \\ 0.5, & 1.0 \leq \mu \leq 2.0 \\ \frac{1}{\mu}, & \mu > 2.0 \end{cases} \quad (4-3)$$

where $K_r$ is the residual stiffness, $K_y$ is the initial secant stiffness to yield, and $\mu$ is the prior displacement ductility demand.

In practice, a measured initial secant stiffness to yield is not available to normalize against. Figure 4-24(b), shows the same data as Figure 4-24(a), normalized against an initial secant stiffness calculated as a ratio of the gross flexural stiffness, as per the provisions of ASCE 41-17 (note that $0.3E_cI_g$ was used in all cases, as all the test specimens had initial axial load ratios of approximately 0.1 or below). Although slightly more scatter is observed, the proposed Equation (4-3) still provides a conservative residual stiffness value in all cases.

It is important to note that the data in Figure 4-24(b) still does not account for the full variability that exists in post-earthquake situations. While variations in the prior loading history and the estimated versus actual initial stiffness are accounted for, the variation in the estimated versus actual displacement ductility are not. This is the basis for the considerable conservatism in the proposed expression.
Accounting for moderate plastic hinging damage in post-earthquake assessments

Figure 4-24: Comparison of proposed stiffness degradation relationship \((1/\mu)\) and Di Ludovico et al. [124] relationship versus experimental residual stiffness values normalized against (a) the initial measured stiffness, and (b) \(0.3EJ_s\)
The use of a displacement ductility of 1.0 as the onset of the stepwise function is useful, as non-hairline residual crack widths in beams can be used as indicators that yielding of the longitudinal reinforcement has occurred. Therefore, in cases where non-hairline residual crack widths are observed, the residual stiffness of a beam should be taken as no greater than 50% of the initial secant stiffness to yield, regardless of the estimated displacement ductility. However, it must be emphasized that this use of crack widths does not necessarily work for columns, or other members subjected to axial compression. If concrete cover spalling occurs in columns this can be taken as an indication that yielding has occurred (see Section 4.2.3), but a lack of spalling does not preclude the possibility of prior yielding. For example, Figure 4-25 shows a photograph of the bridge column specimen from the Schoettler et al. [123] test program, after being subjected to a shake table test that imposed a peak drift demand of 1.8% and caused yielding to occur in the longitudinal reinforcement. No damage other than hairline flexural cracking was observed on the column, yet the stiffness in the subsequent shake table test was reduced by approximately 30%, relative to the initial secant stiffness to yield.

Figure 4-25: Photograph of reinforced concrete column showing no signs of damage (other than hairline cracking) despite having previously yielded, after [123]

4.3.4 Summary of recommended steps for determining the residual stiffness of moderately damaged plastic hinges

The steps required in order to obtain a conservative estimate of residual stiffness for moderately damaged plastic hinges, according to the analysis of this section, are as follows:

- Obtain an estimate of the maximum deformation demand the component was subjected to during the damaging earthquake. This estimate could be either based on damage observations, or be based on a numerical model of the building (as recommended in the draft post-earthquake assessment methodology presented in Appendix C).
• Calculate the corresponding maximum displacement (or rotation) ductility the component was subjected to during the damaging earthquake, using a yield rotation calculated as per the provisions of standard seismic assessment guidelines.

• Calculate the initial secant stiffness to yield \( K_y \) of an equivalent undamaged component using the provisions of standard seismic assessment guidelines.

• Calculate the residual stiffness \( K_r \) as per Equation (4.3), as a ratio \( K_r/K_y \).

• If the following steps add conservatism, update the residual stiffness estimates using the visual damage:
  - If non-hairline residual cracks are present, \( K_r/K_y \) should be taken as no greater than 0.5.
  - If concrete cover spalling is present, \( K_r/K_y \) should be taken as no greater than 0.5.

4.4 Conclusions and Recommendations

The information that can be gleaned from various moderate visual damage indicators in reinforced concrete plastic hinges was investigated. The specific focus was on the relationships between visual damage and either (i) the deformation demands during the damaging earthquake, or (ii) the residual capacity of the plastic hinge. The following conclusions were drawn:

• There are significant limitations with the use of residual crack width metrics to inform post-earthquake assessments (see Section 4.1.9 for a summary of these limitations).

• Moderate plastic hinging damage (i.e. flexural cracks, longitudinal cracks, sliding plane cracks, or cover concrete spalling) does not typically result in a reduction of strength or deformation capacity. However, plastic hinges exhibiting sliding plane cracks over a critical width or severe spalling of cover concrete may be more likely to have lower deformation capacities than undamaged equivalents (see Section 4.2.4 for a summary of recommendations related to these damage states).

• In ductile plastic hinge zones not subjected to axial compression (i.e. in beams), a lack of non-hairline residual crack widths can indicate that yielding has not occurred, a single non-hairline residual crack can indicate that some yielding has occurred, and multiple non-hairline residual cracks can indicate that more significant plastic deformation demands have occurred. ‘Non-hairline’ was here defined as cracks with widths greater than 0.2mm, although this may be too narrow to be practical. Use of a wider definition of ‘non-hairline’, e.g. 0.5mm, may be more appropriate in post-earthquake situations.

• In ductile plastic hinge zones subjected to axial compression (i.e. in columns), both cover concrete spalling and non-hairline residual crack widths are indicators that yielding has
occurred. However, a lack of observable damage other than hairline residual cracking does not necessarily indicate that yielding has not occurred. In such situations, it may be possible to approximate the column deformation demands indirectly based on the demands estimated to have occurred in the beams. Further research is recommended.

- Visual damage indicators have limitations for the purposes of assessing residual stiffness. It is instead recommended that an estimate of the maximum deformation demand during the damaging earthquake be used as the basis for calculating the residual stiffness of a plastic hinge. A simple inverse relationship between the residual stiffness (normalized against the initial secant stiffness to yield) and the displacement ductility was found to yield conservative results, based on a dataset of reinforced concrete components where the residual stiffness was measured after application of arbitrary earthquake-type loadings. A complete description of the proposed methodology for assessing residual stiffness is given in Section 4.3.4.
Chapter 5

The effects of yielded longitudinal reinforcement

This chapter investigates the effects that yielded longitudinal reinforcement has on the post-earthquake residual capacity of plastic hinges in reinforced concrete moment frames. The results are also relevant for the reparability of such plastic hinges by epoxy injection. The work in this section is focussed on longitudinal reinforcement that has yielded but not buckled, consistent with the definition of ‘moderate’ damage previously given in Section 1.4. Three distinct phenomena (residual strain, low-cycle fatigue, and strain ageing) that affect the stress-strain behaviour of reinforcing steel that has previously been strained into the plastic range are here considered.

Residual strain occurs in reinforcing steel that has been strained beyond the yield point but has not subsequently been forced back to its original position. Residual strain reduces the available strain capacity until failure when reloaded monotonically in the same direction as the residual strain. The magnitude of residual strain can also affect the degree to which the Bauschinger effect contributes to the cyclic stress-strain behaviour upon reloading [43, 125, 126].

Low-cycle fatigue and strain ageing are more complex phenomena. The following sections draw on past material-level testing programs to make conclusions about the effects of low-cycle fatigue and strain ageing. An experimental program on the low-cycle fatigue behaviour of grade 300E reinforcement is also described.

5.1 Low-cycle fatigue

Repeated inelastic loading cycles can cause steel to fracture at a lower strain than it would under monotonic loading. This behaviour is termed low-cycle fatigue, and has been put forth as an important failure mode in reinforced concrete plastic hinges during seismic events [76]. The fatigue life of reinforcing steel is dependent on a number of factors, such as bar diameter, deformation geometry, metallurgical properties, and yield strength [127]. However, the most important variable governing the fatigue life is the amplitude of the applied strain reversals.

Table 5-1 shows a list of past material-level experiments that have been conducted to investigate the low-cycle fatigue behaviour of reinforcing steel [12, 127-132]. As this review is focussed on ductile plastic hinges, only studies that involved cyclic testing of deformed reinforcing steel with an unsupported length \(s/d_b \leq 6\) are included in Table 5-1. Data from the Ghannoum and Slavin [127] test program corresponding to Grade 100 (690MPa) steel are not included as high strength
reinforcing steel is currently not used in ductile plastic hinge regions. The steels in these studies were manufactured using a variety of methods, including micro-alloying and quenching and tempering. The bar diameters and reported monotonic properties from control benchmark tests are included in Table 5-1.

<table>
<thead>
<tr>
<th>Study</th>
<th>Steel grade</th>
<th>Bar diameter (mm)</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$\varepsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ghannoum and Slavin (2016)</td>
<td>ASTM A706-60</td>
<td>16.0-35.0</td>
<td>424-499</td>
<td>628-719</td>
<td>0.095-0.119</td>
</tr>
<tr>
<td></td>
<td>ASTM A615-80</td>
<td>16.0-25.0</td>
<td>554-577</td>
<td>724-759</td>
<td>0.095-0.100</td>
</tr>
<tr>
<td>Mander et al. (1994)</td>
<td>ASTM A615-40</td>
<td>15.9</td>
<td>331</td>
<td>565</td>
<td>0.144</td>
</tr>
<tr>
<td>Hawileh et al. (2010a)</td>
<td>BS 460B</td>
<td>9.7</td>
<td>514-519</td>
<td>611-619</td>
<td>Not reported</td>
</tr>
<tr>
<td></td>
<td>BS B500B</td>
<td>11.7</td>
<td>608-614</td>
<td>702-705</td>
<td>Not reported</td>
</tr>
<tr>
<td>Hawileh et al. (2010b)</td>
<td>ASTM A615-60</td>
<td>19.1</td>
<td>437-443</td>
<td>700-709</td>
<td>Not reported</td>
</tr>
<tr>
<td></td>
<td>ASTM A706-60</td>
<td>19.1</td>
<td>474-488</td>
<td>690-690</td>
<td>Not reported</td>
</tr>
<tr>
<td>Kashani et al. (2015)</td>
<td>BS B500B</td>
<td>12.0-16.0</td>
<td>536-544</td>
<td>634-641</td>
<td>0.104-0.143</td>
</tr>
<tr>
<td>Loporcaro (2017)</td>
<td>AS/NZS 4671 300E</td>
<td>12.0</td>
<td>305-330</td>
<td>442-452</td>
<td>0.186-0.200</td>
</tr>
</tbody>
</table>

The studies listed in Table 5-1 applied fully-reversed constant amplitude strain cycles to reinforcement coupons until the point of fracture (here termed ‘low-cycle fatigue testing’). Figure 5-1 shows twice the total strain amplitude ($2\varepsilon_a$), or the peak-to-trough strain reversal, versus the number of half cycles to failure ($2N_f$) data from these previous test programs. In some cases the data points in Figure 5-1 represent the average values of two or three identical tests. Mean strain values were varied between tests in some cases, but that is not considered here as researchers have reported that mean strain has a limited effect when considering the large strain amplitudes that induce low-cycle fatigue [128, 133] (i.e. the data in Figure 5-1 includes specimens with a variety of mean strain values).

The Ghannoum and Slavin [127] test program involved a large number of variables other than strain amplitude, and the number of reversals to failure is therefore widely scattered for a given applied strain. In the other test programs, a linear trend is evident within the dataset, but the trend (i.e. the fatigue resistance of the steel) varies widely between the different experimental programs. These differences are likely in part due to the various methods used for measuring the average strain across the clear length of the specimens, and the difficulty of accurately performing such a measurement over short gauge lengths. The apparent enhanced fatigue resistance of the Hawileh et al. (2010a)
The effects of yielded longitudinal reinforcement

[131] specimens may be explained by deformations being included in the strain amplitude results that did not actually induce strain in the specimens (e.g. slippage of the gripping system or flexibility of the loading rig). Differences between the initial stiffness and the unloading stiffness in the hysteretic stress-strain plots shown by Hawileh et al. (2010a) support this conclusion. Similar errors may be present in the Kashani et al. [132] strain amplitude data, as evidenced by significantly higher yield strains than would be expected based on an assumed elastic stiffness of 200GPa. A similar plot to Figure 5-1 is reproduced in Figure 5-2 but with data from the Hawileh et al. (2010a) and Kashani et al. test programs removed.

![Figure 5-1: Logarithm of the peak-to-trough strain reversal versus logarithm of number of half cycles for constant amplitude low-cycle fatigue tests on deformed reinforcing steel with unsupported length s/d<6](image)

A commonly used model to predict the low-cycle fatigue life of steel is the Coffin-Manson expression for plastic strain [58, 59], which relates the plastic strain amplitude (ε_{ap}) to the number of half cycles to failure, as shown in Equation (5-1). Equation (5-2), which was proposed by Koh and Stephens [133], modifies Equation (5-1) to use the total strain amplitude rather than the plastic strain amplitude.

\[ \varepsilon_{ap} = \varepsilon'_f (2N_f)^c \]  

\[ \varepsilon_a = M (2N_f)^m \]
where $\varepsilon_{ap}$ is the plastic strain amplitude, $\varepsilon_a$ is the total strain amplitude, $2N_f$ is the number of reversals to failure, and $\varepsilon'_f$, $c$, $M$, and $m$ are constants.

Low-cycle fatigue studies, including those listed in Table 5-1, typically conduct regression analysis on their test data to derive constants for these equations. Best fit lines for Equation (5-2) corresponding to (i) the Mander et al. test data, and (ii) all test data with an unsupported length $s/d_b \leq 4$, are included in Figure 5-2. Coefficients for these best fit lines are provided in Table 5-2.

![Figure 5-2: Logarithm of the peak-to-trough strain reversal versus logarithm of number of half cycles for selected experimental programs and fitted lines corresponding to Equation (5-2) for selected datasets](image)

<table>
<thead>
<tr>
<th>Data source</th>
<th>$M$ – constant</th>
<th>$m$ – constant</th>
<th>Coffin-Manson [Equation (5-2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mander et al. (1994)</td>
<td>0.080</td>
<td>-0.45</td>
<td>$\varepsilon_a = 0.080(2N_f)^{-0.45}$</td>
</tr>
<tr>
<td>All specimens from Figure 5-2 with clear spacing $4d_b$ or less</td>
<td>0.093</td>
<td>-0.41</td>
<td>$\varepsilon_a = 0.093(2N_f)^{-0.41}$</td>
</tr>
</tbody>
</table>

Most of the test programs in Figure 5-2 used unsupported length ratios $s/d_b$ representative of typical stirrup or tie spacing in ductile regions of reinforced concrete moment frames. Mander et al. [128], Brown and Kunnath [129], and Loporcaro [12] used $6d_b$ and Ghannoum and Slavin [127] used $s/d_b$ ratios ranging from 4 to 6. These unsupported lengths did not entirely inhibit buckling. Photographs showing the degree of lateral buckling for the three different $s/d_b$ ratios used by
Ghannoum and Slavin are shown in Figure 5-3. The varying degrees of buckling shown in Figure 5-3 had a significant effect on the number of reversals to failure. Ghannoum and Slavin reported an 89% increase in fatigue life (half cycles to failure) of bars that had an unsupported length of $4d_b$ as compared to equivalent specimens with an unsupported length of $6d_b$. The best fit line based on regression analysis to only those specimens with an unsupported length of $4d_b$ or less is therefore included in Figure 5-2 in order to estimate the fatigue resistance of completely unbuckled bars, although it should be noted that these data come entirely from the Ghannoum and Slavin ($4d_b$) and Hawileh et al. ($-2d_b$) [130] test programs.

![Figure 5-3: Maximum lateral buckling observed during low-cycle fatigue testing using three different unsupported lengths, after [127]](image)

It is noted that the methodology of allowing some buckling to occur in the test specimens with $s/d_b$ ratios greater than 4 was intentional. Brown and Kunnath [129] noted that the actual strains at the midsection (buckled area) of the specimens were undoubtedly larger than the reported average strains across the specimen lengths. However, allowing some buckling enabled the findings to be applicable for the purpose of estimating the point of low-cycle fatigue failure in reinforced concrete plastic hinges, where longitudinal reinforcement buckling is likely to occur at higher drift demands. Typical modelling strategies for reinforced concrete utilize the plane sections remain plane assumption, which dictates that only average strains can be extracted from analysis. Average strain is therefore a useful parameter for predicting the point of low-cycle fatigue failure of longitudinal
The effects of yielded longitudinal reinforcement in analysis of reinforced concrete structures, and these test programs were designed to reflect that.

The separation of buckled and completely unbuckled bars in this review is due to the focus on post-earthquake assessment, where additional information is available. Visual inspection can help inform the assessor whether or not buckling occurred. If buckling of the longitudinal reinforcement did not occur, the consumption of fatigue life for a given number of cycles would be less than that predicted based on test programs that permitted buckling to occur (e.g. the best fit line corresponding to the Mander et al. data). The best fit line corresponding to specimens with an unsupported length of $4d_b$ or less is expected to be more appropriate for estimating the consumed fatigue life in post-earthquake situations where there is no evidence of prior longitudinal reinforcement buckling.

### 5.1.1 Experimental investigation

One of the limitations of the available test data is the lack of application of variable loading histories to the specimens. Such data are required to assess the applicability of Miner’s rule for assessing the residual strain capacity following an intermediate number of cycles that do not result in failure. Miner’s rule, shown in Equation (5-3), is a simple linear accumulation of the number of applied cycles relative to the number of cycles to failure at each amplitude of strain demand.

$$D = \sum_{i=1}^{n} \frac{N_{\text{cyc},i}}{N_{f,i}} (\text{failure at } D = 1.0)$$  \hspace{1cm} (5-3)

where $N_{\text{cyc},i}$ is the number of applied loading cycles at strain amplitude $i$, $N_{f,i}$ is the number of cycles to failure at strain amplitude $i$, and $n$ is the number of different amplitudes.

An experimental program on grade 300E deformed reinforcing steel was conducted to better understand the effects of an intermediate number of inelastic cycles on the residual uniform strain. The test setup is shown in Figure 5-4. The diameter of the test specimens was chosen to be 12mm, the largest diameter that could be tested due to the force capacity of the hydraulic actuator. The specimens were not machined; the virgin reinforcement was directly held by hydraulic wedge grips, with a 50mm grip length at each end. The clear span between grips was 48mm, or $4d_b$. Force and displacement readings were obtained from the actuator load cell and LVDT. This method of measuring displacement meant that any strain penetration that occurred in the grip lengths was included in the LVDT output. This error was quantified by comparing measured displacements in the elastic range against the theoretical displacement, assuming an elastic modulus of 200GPa. A linear relationship between the error and the applied force was obtained, and a linear correction factor was therefore applied to all raw displacement measurements based on the concurrent force.
measurement. The strains reported in the following pages correspond to these corrected displacement values.

![Figure 5-4: Test setup for low-cycle fatigue testing program](image)

Three specimens were tested under constant amplitude low-cycle fatigue loading protocols, with one specimen at each of 0.015, 0.025, and 0.035 strain amplitudes (i.e. with peak-to-trough strain reversals, $2\varepsilon_a$, of 0.03, 0.05, and 0.07) [Figure 5-5(a-c)]. The strain was primarily tensile, with only modest (approximately 0.005) compressive strains being applied, in order to emulate the strain demands on longitudinal reinforcement in reinforced concrete plastic hinges subjected to flexure. The specimens failed after 83, 27, and 13 cycles, respectively. The specimen cycled at a $2\varepsilon_a$ of 0.03 did not exhibit brittle fracture until severe cyclic strength deterioration had already occurred, and the failure cycle was therefore defined as the cycle in which the peak stress first dropped below 80% of the maximum stress. As shown in Figure 5-5(d), the fatigue resistance of these specimens was somewhat higher than those of the Ghannoum and Slavin [127] and Hawileh et al. [130] test programs previously used to derive coefficients for the Coffin-Manson equation for reinforcement with a clear span of $4d_b$ or less (see Figure 5-2). This is perhaps due to the high level of ductility that is required in grade 300E steel (minimum uniform strain of 0.15 [37]). ASTM A615-16 [134] and ASTM A706-16 [135] require grade 60 reinforcing steel to have minimum uniform strains of 0.07 to 0.09 and 0.10 to 0.14, depending on bar diameter, respectively.
Three additional specimens were tested under a loading protocol that involved applying approximately \( N_f / 3 \) cycles (again with one specimen each at \( 2\varepsilon_a \) values of 0.03, 0.05, and 0.07), followed by a monotonic pull to failure (Figure 5-6). The uniform strain reached by these specimens is compared against (i) the uniform strain reached by a control monotonic specimen (stress-strain shown in grey), and (ii) the uniform strain that would be predicted using Miner’s rule, assuming the uniform strain corresponds to the failure point. In all cases, the measured uniform strain was less than that of the monotonic specimen, but greater than that predicted by Miner’s rule. The specimen cycled at a \( 2\varepsilon_a \) of 0.03 exhibited the largest reduction in uniform strain relative to the monotonic specimen (approximately 26%). It should be noted that this specimen also exhibited a lower displacement due to necking (i.e. post-ultimate), with deformation on the order of 2.0mm instead of...
of the 6.0-8.0mm of the other specimens. Nonetheless, the results indicate that (i) grade 300E steel can exhibit a high (>0.15) uniform strain even after being subjected to approximately $N_f/3$ cycles, and (ii) Miner’s rule tends to give a conservative value for the residual uniform strain after an intermediate number of inelastic loading cycles.

Two specimens were tested under variable strain histories representative of longitudinal reinforcement in elongating plastic hinges, followed by a monotonic pull to failure (Figure 5-7). The strain histories were derived using simplified versions of the longitudinal reinforcement strain

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**Figure 5-6: Residual strain capacity of test specimens after being subjected to approximately $N_f/3$ cycles at 2ε values of (a) 0.03, (b) 0.05, and (c) 0.07**
data measured on beam specimen LD-1 during the initial earthquake loading (note that the strains were applied at a quasi-static loading rate regardless of the loading rate measured on the beam specimen reinforcement). Simplification of the strain history was necessary as the software controlling the hydraulic actuator did not have the capability to control displacement based on an input data file. One of the specimens was tested with the maximum strain in the variable loading approximately corresponding to the measured maximum of 0.045 (i.e. the maximum strain measured on beam specimen LD-1), while the strain history of the other was scaled up to give a maximum of 0.065. As shown in Figure 5-7, neither of these strain histories were found to affect the residual uniform strain, relative to that of the control monotonic specimen. The number of effective cycles at a strain reversal of 0.03 \( (N_{eff,0.03}) \) was 2.6 and 8.3 for the 0.045 and 0.065 maximum strain specimens, respectively (the original, un-simplified strain histories had corresponding values of 5.4 and 11.3). The values of \( N_{eff,0.03} \) were calculated using Equation (3-1) with \( \varepsilon_{u,\text{max}} = 0.03, C = 2.0, \) and cycles with peak-to-trough reversals less than \( 2\varepsilon_y \) (taken as 0.003) omitted. These results indicate that a negligible change in the uniform strain of grade 300E longitudinal reinforcement can be expected in post-earthquake situations where there is no evidence of prior buckling, provided that the imposed number of effective cycles at a strain reversal of 0.03 was in the range of 10 or less. However, the limited number of tests must be highlighted.

![Figure 5-7](image-url)

**Figure 5-7:** Residual strain capacity of test specimens after variable strain histories to (a) 0.045 and (b) 0.065 maximum strain
5.1.2 Summary of findings on the effects of low-cycle fatigue

An exponential relationship, commonly referred to as the Coffin-Manson equation, exists between the fatigue life of steel and the applied strain amplitude. Based on data from four experimental test programs on ASTM A615 and A706 reinforcing steel of various grades and one test program on New Zealand grade 300E reinforcing steel, the coefficients for the total strain amplitude version of the Coffin-Manson equation [Equation (5-2)] proposed by Mander et al. [128] were found to yield a conservative value (Figure 5-2) for the number of half cycles to failure for a given strain amplitude. Given that some degree of lateral buckling was allowed to occur in the Mander et al. test program, less conservative coefficients, such as those determined from the data of specimens with clear spacing of less than $4d_b$ only, may be more appropriate for application to completely unbuckled reinforcing steel.

The previous test programs identified in the literature did not apply variable strain amplitudes to deformed reinforcing steel samples. Such test data are required to investigate the applicability of Miner’s rule [60] for assessing cumulative fatigue and to understand the effect that an intermediate number of cycles that does not cause failure has on the residual uniform strain. An experimental program was conducted on grade 300E deformed reinforcement to help fill this research need. It was found that Miner’s rule gave a conservative lower bound on the residual uniform strain following an intermediate number of inelastic cycles. It was also found that a variable strain history representative of one that might occur in an elongating reinforced concrete plastic hinge during an earthquake did not affect the residual uniform strain.

It is recommended that additional test programs be conducted on deformed reinforcing steel manufactured to standards other than ASTM, to ensure a wide applicability of the results. It is also recommended that additional variable strain amplitude tests be conducted on deformed reinforcing steel, to supplement the limited data of the test program conducted as part of this work. Finally, additional low-cycle fatigue test programs using clear spans of $4d_b$ or less are required to validate the proposed coefficients for the Coffin-Manson equation, in cases of completely unbuckled reinforcement.

5.2 Strain ageing

Strain ageing is a phenomenon that affects certain steels that have been subjected to plastic strains and then allowed to rest for a period of time. Upon reloading, the steel can exhibit altered properties relative to the behaviour that would have occurred had reloading commenced immediately. These changes can affect the yield strength, Bauschinger effect, strain hardening, ultimate strength, uniform strain (strain at ultimate strength) and fracture strain (Figure 5-8 [136]). An explanation of the metallurgical basis for strain ageing is available in [137]. A selection of past studies available
in the literature have investigated the strain ageing behaviour of reinforcing steels used in New Zealand [12, 137-141]. Early studies focussed predominantly on the strain ageing of reinforcing steel that was cold-bent during the construction process [137, 138], but recent research has targeted the influence of strain ageing on the post-earthquake behaviour of plastically strained reinforcement in ductile reinforced concrete structures [12, 139-141].

The presence of certain alloying metals in the steel, such as Titanium and Vanadium, can negate the effects of strain ageing [136]. Erasmus and Pussegoda [142] quantified the required Vanadium content to prevent changes in stress-strain behaviour due to strain ageing. Modern grade 500E reinforcing steel in New Zealand which is manufactured using the micro-alloying process is not susceptible to strain ageing as it contains an adequate Vanadium content [141]. Grade 430 steel, which was previously a commonly used reinforcing steel in New Zealand (NZS 3402:1989 [143]), may also be alloyed with Vanadium and be resistant to strain ageing effects [139].

For steels that are susceptible to strain ageing (such as grade 300E reinforcement manufactured to AS/NZS 4671:2001 [37]), the magnitude of plastic strain previously applied to the steel and the duration since the plastic straining occurred are key variables affecting the degree of influence strain ageing has on the reloading behaviour [139-141].
Restrepo-Posada et al. [139] tested samples machined from Grade 300 (NZS 3402:1989 [143]) reinforcing steel monotonically and cyclically (one load reversal only). They showed that, for specimens that were strain aged after an applied pre-strain that did not initiate strain hardening (i.e. a pre-strain in the yield plateau range), limited strength increases occurred, but the Bauschinger effect tended to disappear for cyclically loaded specimens. Monotonic specimens that were pre-strained into the strain hardening range prior to ageing exhibited reappearance of a yield plateau and an increase in strength (~15%) at the reloading strain (strength increase $\Delta Y$ in Figure 5-8). Cyclic specimens that were pre-strained into the strain hardening range prior to ageing did not exhibit a new yield plateau, but the Bauschinger effect was reduced upon reloading (i.e. the reloading stiffness after strain ageing was similar to the initial elastic stiffness), and an increase in strength (~10%) after ageing was recorded. Negligible increases in ultimate strength due to strain ageing were observed in all cases, but the strains at ultimate stress (uniform strain) were reduced to between 81-96% of the un-aged values. Relatively minor differences were observed after ageing for 37 versus 147 days between otherwise identical specimens.

Momtahan et al. [140] performed monotonic tension tests on samples of strain-aged grade 300E reinforcing steel, with pre-strain levels of 2-15$\varepsilon_y$ and ageing periods of 3-50 days. Increases in strength at the reloading strain were small (<4%) for specimens with pre-strains of 2$\varepsilon_y$ or 5$\varepsilon_y$, but were estimated to be as high as 15% for specimens with a pre-strain of 15$\varepsilon_y$ that were aged for 50 days. The strength increase at the reloading strain was not directly reported in the paper and had to be determined visually from the stress-strain plots. The ageing period was found to be a relatively important factor in determining the increase in strength, with an approximately linear relationship between the two parameters (for ageing in the range of 3-50 days). The ultimate strength was unaffected in all cases, regardless of the degree of pre-strain or the ageing period. The bars were not tested to rupture; only strains of 0.2 could be achieved with the testing apparatus. The strain aged bars appeared to be able to achieve this 0.2 strain in all cases, although limited discussion or information on uniform strain is provided in the paper.

Loporcaro et al. [141] performed monotonic tension tests on strain-aged samples machined from grade 300E reinforcing steel, with pre-strains ranging from 0.01-0.05 and ageing periods of 7-365 days (additional specimens were tested to validate the efficacy of ‘accelerated strain-ageing’ but those are not considered here). A pre-strain of 0.01, which was in the yield plateau range, had a limited effect on the stress-strain behaviour regardless of the ageing period. Pre-strains of 0.03 and 0.05 (both in the strain hardening range) caused a re-appearance of a yield plateau and an increase in strength at the reloading strain, the magnitude of which was somewhat dependent on the ageing period and peaked at around 15%. Increases in ultimate strength were less than 5% in all cases. Decreases in uniform strain were reported, but the authors appear to have subtracted the pre-strain from the uniform strain of the strain-aged specimens when calculating this decrease. Since residual
strain is being treated as a separate phenomenon in this review, the decreases in uniform strain calculated by Loporcaro et al. are not applicable. Values of total uniform strain (including pre-strain) were therefore approximated from the data provided by Loporcaro et al. Strain ageing was observed to have a detrimental effect on uniform strain, with values estimated to be as low as 70% of the un-aged control tests. However, in all specimens a minimum uniform strain of 0.135 was reached. Ageing beyond 90 days had little effect on further reducing the uniform strain.

In all three test programs here reviewed, strength increases due to strain ageing were found to be appreciable, given the right conditions. Specimens that were pre-strained into the strain hardening range and aged for at least 30 days exhibited strengths upon reloading that were generally on the order of 10-15% higher than that of un-aged equivalents, with a few outliers at around 5%. In some specimens, this strength increase was accompanied by the re-appearance of a yield plateau, which resulted in the difference in strength between the aged and un-aged specimens quickly dropping as the strain demand was increased. In all cases, strain ageing had a limited influence on ultimate strength (< 5% increase).

One of the most concerning aspects of strain ageing is the potential for reduced strain capacities. Figure 5-9 shows the relationship between the magnitude of applied pre-strain and the post-ageing uniform strain, using data from the Restrepo-Posada et al. [139] and Loporcaro et al. [141] test programs (data from Momtahan et al. [140] were not included due to there being a lack of sufficient information on uniform strain in that paper). The data were split into two broad bins, those aged less than 90 days (10 specimens) and those aged for greater than 90 days (13 specimens). Comparisons are drawn against (i) the uniform strain capacity of the non-aged monotonic control specimens (approximately 0.2 in both test programs), (ii) the uniform strain requirement of 0.15 (90% probability of exceedance) stipulated by AS/NZS 4671:2001 [37] for 300E grade reinforcement, and (iii) the probable uniform strain of 0.1 used in the New Zealand seismic assessment guideline [29]. The reduction in uniform strain relative to the monotonic control tests ranged from 0-32%. However, the AS/NZS 4671:2001 requirement was still met by all specimens except those subjected to a pre-strain of 0.05 and aged longer than 90 days; these specimens had uniform strains ranging from approximately 0.13-0.15. In all cases, the probable uniform strain prescribed in the New Zealand seismic assessment guideline was exceeded.

For post-earthquake assessments, strain ageing periods of greater than 90 days are of most interest. In Figure 5-10, the data from Figure 5-9 is repeated using the relative uniform strain (i.e. the ratio of aged to un-aged uniform strain). A proposed conservative lower-bound model [Equation (5-4)] for estimating the effect of strain ageing after substantial periods of time (90 days or more) on uniform strain is overlaid on the data.
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\[
\frac{\varepsilon_{u, aged}}{\varepsilon_{u, un-aged}} = \begin{cases} 
1.0, & \text{pre-strain} < 0.01 \\
0.8 - 3(\varepsilon_{\text{pre-strain}} - 0.01), & \text{pre-strain} \geq 0.01 
\end{cases} 
\]

(5-4)

Figure 5-9: Uniform strain versus pre-strain for strain aged 300E grade reinforcement. Data taken from Restrepo-Posada et al. [139] and Loporcaro et al. [141].

Figure 5-10: Relative uniform strain versus pre-strain and proposed relationship [Equation (5-4)] for 300E grade reinforcement aged longer than 90 days.

Strain ageing can also affect the low-cycle fatigue behaviour of reinforcing steel. Loporcaro [12] is the only study to date that has experimentally investigated the effect of strain ageing on the low-cycle fatigue behaviour of reinforcement, to the author’s knowledge. The testing methodology
employed by Loporcaro involved applying either 33% or 66% of the number of constant amplitude cycles to failure to grade 300E deformed reinforcing steel samples. These samples were then subjected to ‘accelerated strain ageing’ by submerging them in boiling water for four hours, and then again cycled at the same amplitude until failure occurred. The strain aged samples were otherwise equivalent to the standard low-cycle fatigue tests conducted by Loporcaro (previously discussed in Section 5.1), enabling conclusions to be drawn regarding the effects of strain ageing on low-cycle fatigue behaviour. It was found that the strain ageing caused a reduction in the post-ageing number of cycles to failure of between 20-75%, relative to the remaining number of cycles to failure that would have been expected if testing had continued with no strain ageing.

It is noted that the testing program of Loporcaro [12] utilized an unsupported length of $6d_b$, which permitted some degree of buckling to occur in the reinforcement samples. The maximum strains applied to the reinforcement samples are therefore unknown, as is the degree to which the buckling may have influenced the effect of the strain ageing. Nonetheless, the results presented by Loporcaro show that low-cycle fatigue failure of strain-aged reinforcement is of concern. Testing programs that investigate the effect of strain ageing on the low-cycle fatigue response of completely unbuckled samples are recommended. Natural strain ageing tests are also recommended to supplement the accelerated strain ageing tests conducted by Loporcaro.

5.2.1 Summary of findings on the effects of strain ageing

Steels containing adequate contents of certain alloying metals such as Titanium or Vanadium are not susceptible to strain ageing. Test programs investigating strain ageing in reinforcing steel have been concentrated in New Zealand, and the results should therefore be applied with caution to steels manufactured to other standards. The following observations can be made regarding the effects of strain ageing on susceptible New Zealand steels, such as the current grade 300E reinforcing steel and past equivalents:

- For pre-strain levels that do not reach the strain hardening range, strain ageing has a limited effect.
- For pre-strain levels that reach the strain hardening range:
  - An ageing period of 90 days is sufficient to cause the majority of strain ageing effects to be realized.
  - A yield plateau re-appears and this new ‘yield strength’ can exceed the strength that the steel would have exhibited if immediately reloaded by up to 15%.
  - For cyclically loaded steel, the Bauschinger effect is reduced, and the reloading stiffness of the steel can become similar to the initial elastic stiffness.
A limited increase in ultimate strength of less than 5% can occur on 300E steel, although greater increases of over 10% were observed on Grade 275 steel (NZS 3402P:1973 [144]) tested by Pussegoda [136].

The uniform strain (strain at ultimate stress) reduces as a function of the applied pre-strain, but still exceeds 0.135 in all reported tests on strain aged 300E steel.

- Strain ageing can have a significant detrimental effect on the low-cycle fatigue life of deformed reinforcement. Additional testing is recommended to quantify this effect in completely unbuckled samples.

5.3 Estimating strain demands in the longitudinal reinforcement of plastic hinges

In order to estimate the effects of prior inelastic straining when assessing residual capacity or reparability, it is necessary to understand the strain demands placed on longitudinal reinforcement in plastic hinge regions. Strain hardening, which is a key factor in causing strains to distribute along a plastic hinge length, begins at strains of 0.025 and below in most reinforcing steel grades (e.g. as shown in [127-129, 139-141]). Strains of 0.03 are therefore typically sufficient to ensure a redistribution of strain in the longitudinal reinforcement and the formation of a well-distributed plastic hinge. Figure 5-11 illustrates the importance of strain hardening on the applicability of equivalent plastic hinge length equations.

In cases of strains greater than 0.03, it is here assumed that the strain induced in the longitudinal reinforcement of a plastic hinge due to member rotation can be approximated using a bilinear elastic–plastic deformation model and an equivalent plastic hinge length [Equation (5-5)]. This is the strain that corresponds to the ‘geometrical elongation’ previously discussed in Section 3.3. This equation is intended to approximate the maximum tension strains at the critical section of the plastic hinge, and is therefore a conservative estimate of the average tension strains across the hinge length. The conservatism is due to the nature of plastic hinge length models, which are intended to represent the length along which the maximum plastic curvature must act in order to give an equivalent member plastic rotation [105].

\[
\varepsilon_{rot} = \frac{(\theta - \theta_y)(d - c)}{L_p} + \varepsilon_y = \frac{\theta_p(d - c)}{L_p} + \varepsilon_y
\]

where \(\varepsilon_{rot}\) is the longitudinal reinforcement strain due to member rotation, \(\theta\) is the chord rotation of the member, \(\theta_y\) is the yield rotation, \(d\) is the depth from extreme compression fiber to centroid of
longitudinal reinforcement, \( c \) is the neutral axis depth, \( L_p \) is the equivalent plastic hinge length, \( \varepsilon_y \) is the yield strain, and \( \theta_p \) is the member plastic rotation.

If \( \varepsilon_y \) is neglected and \( c \) and \( L_p \) are approximated as functions of \( d \) with assumed values of \( c = d/10 \) and \( L_p = d/2 \), \( \varepsilon_{\text{rot}} \) becomes \( 1.8(\theta - \theta_y) \) or \( 1.8\theta_p \). This can be considered a rough but simple estimation of \( \varepsilon_{\text{rot}} \) in ductile members with well-distributed plastic hinges.

Figure 5-11: Importance of strain hardening on plastic hinge development
In Figure 5-12, the accuracy of this calculation method is investigated by comparing Equation (5-5) against longitudinal reinforcement strain data from beam specimen MONO (up to 5.0% drift), which had no reversed loading and therefore all strain was due to member rotation. Unsurprisingly, Equation (5-5) is a non-conservative predictor of reinforcement strain prior to the onset of strain hardening. However, the point of strain redistribution is clearly distinguishable, and at drifts of 2.0% and above, it provides a conservative estimate. Note that displacement gauge N1 was connected to the beam foundation, and therefore includes strain penetration deformations, but this was not accounted for when converting displacement to strain (i.e. the gauge length was taken as the distance from the connection point on the beam to the beam-foundation interface). The strains reported for gauge N1 in Figure 5-12 therefore overestimate the true strain in the longitudinal reinforcement.

Consideration of increased elongation due to cyclic loading is required for determining the absolute maximum strains in the longitudinal reinforcement of reversing plastic hinges with limited axial compression or restraint. As Equation (5-5) represents the strain due to geometrical elongation, it is only necessary to determine the additional strain, which arises due to accumulated tension strains in the compression end steel. The difference between the total elongation and the geometrical elongation is here termed the ‘accumulated elongation’ ($e_{\text{accum}}$), a parameter that is related to, but distinct from, residual elongation (the calculations here correspond to the peak response rather than residual). The corresponding magnitude of longitudinal reinforcement strain (i.e. the strain that is additional to $\epsilon_{\text{rot}}$) is here termed the ‘accumulated strain’ ($\epsilon_{\text{accum}}$). A simple model is used to calculate the accumulated strain, by assuming the accumulated elongation is uniformly spread along the equivalent plastic hinge length [Equation (5-6)]. The maximum expected accumulated elongation for a given rotation demand can be estimated by subtracting the geometrical elongation from the total elongation. The geometrical and total elongation can be calculated using the equations in NZS 3101:2006 for unidirectional [Equation (3-8)] and reversing [Equation (3-9)] hinges, respectively. In the case of zero axial load, the accumulated elongation can therefore be calculated as given by Equation (5-7).

$$\epsilon_{\text{accum}} = e_{\text{accum}}/L_p$$

$$e_{\text{accum}} = e_{\text{rev}} - e_g = 0.8 \theta_m (d - d')$$

where $\epsilon_{\text{accum}}$ the longitudinal reinforcement strain due to accumulated elongation, $e_{\text{accum}}$ is the accumulated elongation, $e_{\text{rev}}$ is the elongation of a reversing hinge [Equation (3-9)], $e_g$ is the elongation of a unidirectional hinge [Equation (3-8)], $L_p$ is the plastic hinge length, and $\theta_m$, $d$, and $d'$ are as defined in Equation (3-8).
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Figure 5-12: Comparison of $\varepsilon_{rot} = 1.8\theta_p$ with longitudinal reinforcement strain data from beam specimen MONO.
Assuming $\theta_m = \theta_p$, $(d - d') = 0.95d$, and $L_p$ again as $d/2$, $\varepsilon_{\text{accum}}$ becomes $1.5\theta_p$. This can be considered a rough but simple estimation of the accumulated strain expected in ductile members with well-distributed plastic hinges. This calculation method for accumulated strain is considered to be conservative as it uses the NZS 3101:2006 reversing plastic hinge equation, which can significantly overestimate the accumulated elongation in cases of random earthquake load histories (see Section 3.3). More accurate calculations, such as that proposed in Equation (3-10), may be more appropriate where the load history is known. Furthermore, the accumulated elongation is not necessarily localized in the equivalent plastic hinge length, as assumed in Equation (5-6), but instead spread across the entire length in which inelastic deformation occurs.

The absolute maximum strain in the longitudinal reinforcement of an elongating hinge is due to a combination of rotation demands and accumulated elongation, i.e. $\varepsilon_{\text{tot}} = \varepsilon_{\text{rot}} + \varepsilon_{\text{accum}}$. However, in terms of cyclic loading, the amplitude of strain reversals are largely a function of rotation demands only. Figure 5-13 illustrates this behaviour, using the hysteretic shear force versus average strain responses for gauges N1 and S1 on beam specimen CYC. Therefore, only $\varepsilon_{\text{rot}}$ need be considered when estimating the amplitude of strain reversals in longitudinal reinforcement. The absolute maximum strain is of limited importance for fatigue analysis purposes, as previous research has shown that the mean strain (i.e. the strain at the midpoint of the peak-to-trough strain reversal) has a limited effect on the fatigue life of steel [128, 133].

![Figure 5-13: Strain hysteresis for gauges N1 and S1 on specimen CYC (data up to 1.5% drift)](image-url)

The calculations relating longitudinal reinforcement strain to plastic rotation that were laid out in this section make use of a number of assumptions. Besides those already noted, they assume that plane sections remain plane, and neglect any tension lag or shear deformation. More information on the difficulty of calculating material strains in reinforced concrete plastic hinges is provided by Dhakal and Fenwick [145]. Nonetheless, these calculations are here used as a simple method of roughly approximating the plastic rotations at which critical longitudinal reinforcement strains are likely to be exceeded.
5.4 Accounting for the residual capacity of longitudinal reinforcing steel when assessing damaged plastic hinges

Figure 5-14 illustrates the cumulative effects that residual strain, low-cycle fatigue, and strain ageing have on the residual monotonic stress-strain behaviour of unbuckled reinforcing steel. The post-earthquake monotonic stress-strain properties are shown as the red line and the virgin stress-strain properties are shown as the dashed line. The effect that these changes in reinforcing steel properties would be expected to have on the residual stiffness, strength, and deformation capacity of reinforced concrete plastic hinges are discussed in the following sections. A discussion on the residual fatigue life following cyclic loading is also provided.

![Diagram showing residual strain, low-cycle fatigue, and strain ageing effects on residual stress-strain behaviour](image)

**Figure 5-14: Effect of residual strain, low-cycle fatigue, and strain ageing on the post-earthquake residual monotonic stress-strain response of reinforcing steel (not to scale)**

5.4.1 Residual stiffness

With regards to the residual stiffness of a reinforced concrete plastic hinge, the most important change in the stress-strain behaviour of yielded longitudinal reinforcement, relative to its virgin
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behaviour, is the Bauschinger effect. However, the Bauschinger effect is difficult to quantify, as it is dependent on the strain history [43, 125, 126] and can also be affected by strain ageing. It is also likely less important with regards to the residual stiffness than other hinge-level behaviours such as crack closure, bond degradation, and shear deformations. It is therefore recommended that no explicit consideration of the longitudinal reinforcement stress-strain behaviour need be accounted for when assessing residual stiffness. This is reflected in the residual stiffness assessment procedure previously proposed in Section 4.3.

5.4.2 Residual strength

Increase in the strength of longitudinal reinforcing steel can lead to similar percentage increases in the flexural strength of a reinforced concrete plastic hinge. Strength increases due to strain ageing should not be counted on as reliable when assessing probable strength, but should be accounted for when considering hinge overstrengths. The experimental work reviewed in Section 5.2 showed that strength increases due to strain ageing can be as high as 15%. However, using this upper bound strength increase due to strain ageing in addition to standard overstrength factors (e.g. the 1.25fy factor for 300E grade steel that is prescribed in the NZSEE seismic assessment guidelines [29]) may be overly conservative. Following an earthquake, localized fluctuations in the strain demands along the length of the longitudinal reinforcement would likely result in some areas exhibiting larger ageing-related strength increases than others. This would force any future yielding to first occur in the adjacent, weaker steel. It is here recommended that an additional overstrength factor of 1.1 be considered to account for strain ageing effects, except where special study shows that the reinforcing steel is not susceptible to strain ageing. Component-level testing of ductile reinforced concrete plastic hinges with strain aged longitudinal reinforcement is recommended to validate this proposed method.

5.4.3 Residual deformation capacity

Consideration of the post-earthquake monotonic stress-strain behaviour shown in Figure 5-14 in the context of a reinforced concrete plastic hinge yields a conclusion that it may be of limited importance in terms of the hinge deformation capacity. The monotonic tensile strain capacity of longitudinal reinforcing steel does not govern the deformation capacity of ductile plastic hinges in beams and columns subjected to earthquake loadings. This is evidenced by the extreme drifts (over 10%) that monotonically loaded reinforced concrete specimens are able to withstand prior to failure, as previously discussed in Chapter 3.

The material-level experimental program described in Section 5.1.1 showed that prior cyclic loading did not have a significant effect on the residual uniform strain of grade 300E reinforcement.
It is here assumed that, in the absence of buckling, the effects of cyclic loading on the residual uniform strain can typically be neglected, although further tests are required to validate this finding. More information on the effects of prior cyclic loading is provided in Section 5.4.4.

In Section 5.3, it was shown that the maximum strain in the longitudinal reinforcement of elongating plastic hinges can be conservatively estimated, through a simple calculation method, as \( \varepsilon_{\text{tot}} = \varepsilon_{\text{rot}} + \varepsilon_{\text{accum}} = 1.8\theta_p + 1.5\theta_p = 3.3\theta_p \), for a plastic hinge length \( d/2 \). Substituting this expression for maximum strain into the previously derived Equation (5-4), which represents a conservative model for the effects of strain ageing on the residual uniform strain of 300E reinforcement, gives Equation (5-8).

\[
\frac{\varepsilon_{u,\text{aged}}}{\varepsilon_{u,\text{un-aged}}} = \begin{cases} 
1.0, & \text{pre-strain} < 0.01 \\
0.8 - 3(3.3\theta_p - 0.01), & \text{pre-strain} \geq 0.01
\end{cases} \tag{5-8}
\]

Due to the strain localizations that can occur prior to reaching the strain hardening range in the longitudinal reinforcement, use of \( \varepsilon_{\text{tot}} = 3.3\theta_p \) as an approximate maximum strain is only recommended where the predicted strain is 0.03 or higher (i.e. where \( \theta_p \) is approximately 0.009 or higher). Figure 5-15 illustrates Equation (5-8) and shows the region where it can be non-conservative due to strain localizations. The \( \varepsilon_{u,\text{aged}}/\varepsilon_{u,\text{un-aged}} \) ratio corresponding to a longitudinal reinforcement strain of 0.03 is approximately 0.75 (dashed line in Figure 5-15).

Using Equation (5-8), a plastic rotation of 0.015 causes a relative reduction in uniform strain due to strain ageing of approximately \( 1/3 \)rd. Grade 300E reinforcement has a prescribed minimum uniform strain (\( \varepsilon_{u,\text{un-aged}} \)) of 0.15 [37], and therefore a \( 1/3 \)rd reduction results in a minimum
\( \varepsilon_{u,aged} \) in the range of 0.1. A uniform strain of 0.1 is still sufficient to ensure that exceeding the monotonic tensile strain capacity of longitudinal reinforcing steel is not a governing failure mode for reinforced concrete beams and columns that meet modern seismic design provisions. Reinforcing steels that have minimum uniform strains below 0.15 and are susceptible to strain ageing may be of more concern, but this is not applicable in New Zealand, where grade 500E (and the previously used Grade 430) have been shown to be resistant to strain ageing (see Section 5.2).

In cases where the tensile strain capacity of longitudinal reinforcing does not govern failure, residual strain in longitudinal reinforcement should not be considered to be a ‘consumed’ portion of strain capacity, in the sense that it reduces the deformation capacity of the plastic hinge. Residual strain in longitudinal reinforcement is similar to total residual crack width in that it is dependent on the residual deformations of the plastic hinge, as illustrated in Figure 4-1, and therefore is not always indicative of the peak strains incurred. A plastic hinge under a large enough axial load to prevent residual elongation could exhibit zero residual strain in the longitudinal reinforcement following an earthquake, regardless of the prior strain demands. However, that does not necessarily mean that the residual deformation capacity of that plastic hinge would be any higher than that of a hinge with appreciable residual strain in the reinforcement. This is supported by the finding in Section 3.1 that the variance in elongation (and the associated variance in residual strain in the longitudinal reinforcement) due to moderate-level loading cycles was not correlated with the deformation capacities of the beam specimens from the experimental program.

### 5.4.4 Residual fatigue life

Cyclic demands can consume some of the low-cycle fatigue life of reinforcing steel, as described in Section 5.1. In post-earthquake situations where the longitudinal reinforcement was previously subjected to either (i) a high number of cycles, or (ii) high strain amplitudes, the consumed fatigue life may lead to low-cycle fatigue of reinforcement being a more probable failure mode in a future seismic event. There is ongoing research to determine how to quantify the consumed portion of fatigue life in such situations [14]. However, it is here argued that, for typical earthquake loadings, moderate rotation demands, and longitudinal reinforcement without evidence of prior buckling, such calculations are often unnecessary. This is consistent with the experimental observations on reinforced concrete plastic hinges previously discussed in Section 3.1, which concluded that the number of loading cycles at or below 2.0% drift did not significantly affect the residual deformation capacity.

In Section 5.3, it was shown that the maximum strain in longitudinal reinforcement due to rotation can be roughly approximated as \( 1.8\theta_p \) for a plastic hinge length of \( d/2 \) and a neutral axis depth of \( d/10 \). The strain due to rotation (\( \varepsilon_{rot} \)) approximately corresponds to the strain amplitudes that
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occur during load reversals \((2\varepsilon_a)\). Note that this calculation procedure is only applicable after strain hardening has occurred, and can be non-conservative prior to the point at which strain hardening forces a redistribution of strain along the plastic hinge (assumed to be at \(\varepsilon_{\text{rot}} = 0.03\) or roughly \(\theta_p = 0.15\)).

If the \(\varepsilon_{\text{rot}} = 2\varepsilon_a = 1.8\theta_p\) expression is substituted into the total strain amplitude version of the Coffin-Manson expression [Equation (5-2)] using constants as derived based on low-cycle fatigue test data with a clear spacing of \(4d_b\) or less (Table 5-2), Equation (5-9) is obtained.

\[
1.8\theta_p = 0.19(2N_f)^{-0.41}
\]  

Figure 5-16 shows Equation (5-9) plotted on a logarithmic scale. In the shaded region, where the equation is non-conservative due to a potential lack of strain hardening (below \(\theta_p = 0.015\)), \(2N_f\) is equal to or greater than 85, or over 40 fully-reversed cycles. This number of cycles is many times higher than that which occurs during typical earthquake loadings. Chang and Mander [106] proposed a \(N_{\text{eff}} = 7T_n^{-1/3}\) relationship as a conservative estimate of the number of cycles likely to occur due to earthquake loading, and similar results were obtained by Malhotra [57]. It is therefore concluded that consideration of strain localization prior to strain hardening is unnecessary, as low-cycle fatigue failure due to such strains is improbable even under repeated earthquakes. Even at larger plastic rotations of 0.02, \(2N_f\) is equal to approximately 56, or 28 fully reversed cycles, which is still high enough to draw the same conclusion that it can typically be neglected as a potential failure mode.

At higher plastic rotations, in the range of 0.02 radians and above, consideration of the consumed portion of low-cycle fatigue life is of greater importance. However, large plastic rotations may either prevent the building under consideration from being repairable, or induce buckling in the longitudinal reinforcement. It is emphasised that the arguments of this section are not applicable if any degree of buckling has occurred, as it can result in significantly larger strain amplitudes than those given by Equation (5-5). In order to further validate the importance of prior buckling on the fracture of reinforcing steel, the PEER column database [66] was analysed. Of the 35 column specimens where longitudinal bar fracture was reported, all had previously exhibited longitudinal bar buckling.

It is here concluded that, for longitudinal reinforcement without evidence of prior buckling, there are two primary cases in which low-cycle fatigue of unbuckled reinforcement may be a governing failure mode in a future seismic event: (i) if the plastic hinge length is significantly shorter than is typical in ductile members (where typical is here taken as \(d/2\)), thereby allowing strain reversal amplitudes higher than 0.03 to occur even at low drift demands, or (ii) if the estimated maximum plastic rotation is greater than 0.02. Nonetheless, even in these cases, the damaging earthquake or
The effects of yielded longitudinal reinforcement

earthquake sequence must involve a significant number of cycles for low-cycle fatigue failure of unbuckled reinforcement to become probable.

![Coffin-Manson equation (plastic rotation)](image)

Figure 5-16: Coffin-Manson equation for plastic rotation based on Equation (5-9)

One important caveat to this section is that strain age embrittlement and low-cycle fatigue have been treated as mutually exclusive phenomena in the previous discussion. Test data from Loporcaro [12] have shown that strain ageing can reduce the residual fatigue life of reinforcing steel by upwards of 50%. In cases where strain aged reinforcement is subjected to a number of strain reversals, low-cycle fatigue of unbuckled reinforcement may be a more probable failure mode. However, these data come entirely from a test program on reinforcement that was strain aged after being cycled with an unsupported length $s/d_b = 6$ (i.e. allowing some degree of buckling to occur). Testing is required to investigate the residual fatigue life of strain aged reinforcement without prior buckling. Testing of reinforced concrete plastic hinges with strain-aged longitudinal reinforcement under simulated seismic loadings is also strongly recommended.

5.5 Conclusions and Recommendations

Three different factors that can alter the properties of reinforcing steel which has previously yielded, relative to its virgin properties, were identified: residual strain, low-cycle fatigue, and strain ageing. Data from material-level testing programs, including one conducted by the author on the low-cycle fatigue behaviour of grade 300E reinforcement, were used to draw conclusions about the effects of low-cycle fatigue and strain ageing. Strain ageing test programs have been concentrated in New
Zealand, and the results are therefore not necessarily applicable to reinforcing steels manufactured to standards other than AS/NZS 4671:2001 [37].

Simple methods of approximating the strain demands imposed on unbuckled longitudinal reinforcement in the plastic hinges of reinforced concrete moment frames were described. These methods relate the longitudinal reinforcement strain to the hinge plastic rotation, based on equivalent plastic hinge length assumptions (which are considered to be valid once strains of 0.03 or greater develop in the longitudinal reinforcement). Information from the material-level experimental data was combined with these relationships in order to estimate the effects of prior longitudinal reinforcement yielding on the residual stiffness, strength, deformation capacity, and fatigue life of reinforced concrete plastic hinges. The conclusions reached with regards to deformation capacity and fatigue life should be considered in the context of the assumptions on which they are based, and are only applicable in cases where the longitudinal reinforcement has not previously buckled. The following recommendations are made:

- Residual stiffness can be affected by the Bauschinger effect and strain ageing in the longitudinal reinforcement, but these effects are difficult to quantify and possibly less important than other factors affecting the residual stiffness of plastic hinges. It is recommended that there is no need for explicit consideration of prior longitudinal reinforcement strain when assessing residual stiffness.

- Residual strength was found to be primarily affected by increases in longitudinal reinforcement strength due to strain ageing. It is recommended that such strength increases be ignored when assessing the probable strength, but a 1.1 factor be used when assessing overstrength (in addition to standard overstrength factors). Such a factor need not be used in cases where the longitudinal reinforcement is not susceptible to strain ageing.

- Residual deformation capacity, as governed by the monotonic tensile strain capacity of the longitudinal reinforcement, was found to be largely unaffected by prior yielding. The uniform strain of reinforcing steel can be reduced by both prior cyclic loading and strain ageing. However, for plastic rotations of 0.015 radians and below, the reduction in uniform strain is unlikely to be significant enough to result in the monotonic tensile strain capacity of the longitudinal reinforcement becoming a potential failure mode. A possible exception to this finding is reinforcement that is susceptible to strain ageing and has a virgin uniform strain of less than 0.15, but tests on such steel have not been conducted, to the author’s knowledge. In cases where it is not expected that the tensile strain capacity of longitudinal reinforcement will govern failure, residual strain should not be considered to be a ‘consumed’ portion of the total strain capacity of longitudinal reinforcement.

- Low-cycle fatigue of unbuckled longitudinal reinforcement, in the absence of strain ageing, was found to be an unlikely failure mode, even under multiple earthquakes. For strain
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reversals of 0.03, which is comparable to a plastic rotation of 0.015 radians in typical plastic hinges, unbuckled reinforcement was found to withstand over 40 fully-reversed cycles prior to failure. However, low-cycle fatigue failure of unbuckled reinforcement could become a more probable failure mode in future earthquake events if strain ageing occurs in the longitudinal reinforcement. Further research is required.

The findings regarding the limited effects of residual strain and the unlikelihood of low-cycle fatigue failure in unbuckled and non-strain aged reinforcement are supported by the conclusion that the number of cycles below 2% drift does not significantly affect the deformation capacity of ductile reinforced concrete plastic hinges (see Section 3.1). However, the recommendations made with regards to the effects of strain ageing are based solely on simple and conservative analytical extensions of material-level studies. Component-level experiments are required to validate these recommendations.
Chapter 6

The effects of epoxy injection repair of moderate plastic hinging damage

This chapter uses experimental data to draw conclusions about the effects of epoxy injection repair of reinforced concrete plastic hinges. Previous relevant experimental work is reviewed, and detailed analysis is presented on the three repaired specimens tested as part of the experimental program described in Chapter 2. The work in this chapter is focussed on epoxy injection repair of moderate damage (as defined in Section 1.4), up to and including cracking, longitudinal reinforcement yielding, and spalling of cover concrete.

6.1 Previous experimental work

This section describes a review of previous experimental work that involved application of epoxy-injection repair methods to ductile reinforced concrete beam-column elements tested under cyclic loading. This review does not include experimental programs on epoxy-injection repair in which the specimens exhibited damage patterns inconsistent with ductile plastic hinging (e.g. joint shear cracking), such as those found in [146-148]. Also not considered are test specimens that were pushed to heavy damage states, beyond the point of significant strength loss, and complete reconstitution of the core concrete or replacement of the reinforcement was required as part of the repair [146, 147, 149].

Five experimental programs with a combined total of eleven specimens were identified in the literature. A summary of key information about these eleven specimens, and the three repaired beams tested by the author, is given later in this chapter (Table 6-5). The test specimens include beams [150, 151], exterior and interior beam-column assemblies [152, 153], and bridge columns [154]. A detailed review of each of these studies is provided in the following sections. The review focuses on the effects of epoxy-injection repair on the secant stiffness to yield, ultimate strength, deformation capacity, and energy dissipation characteristics. In most cases, numerical data were not available and these parameters had to be estimated from load-deformation plots. In some cases quantitative data could not be obtained with confidence, and qualitative measures are therefore also discussed.

The methodology used in most of these experiments was to test an undamaged specimen to an intermediate damage state, repair it, and then retest it using the same loading protocol. This allows
for comparison between repaired and original conditions, but does not allow for comparison
between the repaired and unrepaired (i.e. as-damaged) conditions. The bridge column specimen of
the Lehman et al. [154] test program was the only case where such a comparison is possible, as a
separate nominally identical specimen was tested, without being repaired, to failure, using the same
loading protocol as the repaired specimen.

6.1.1 Celebi and Penzien (1973) [150]

Four beam specimens were tested using a setup where each specimen consisted of two pin-ended
beams on either side of a centre column stub. It is noted that this setup would have restrained axial
elongation of the beams. The beams had a rectangular cross-section of 381x178mm, a longitudinal
 reinforcement ratio of 1.3%, and the shear span and transverse reinforcement content were varied
between the specimens. The loading protocol consisted of incrementally increasing cycles with four
cycles at each increment and five post-yield increments. In three of the four specimens the original
loading was applied dynamically, and those specimens exhibited the spike in yield strength due to
dynamic loading that was previously discussed in Section 3.2. The peak drift applied in the protocol
was not explicitly provided, but was estimated from the load-deformation plots to be approximately
3.0% for all specimens. The beams exhibited ductile plastic hinging as shown in Figure 6-1(a).

![Figure 6-1](image)

*Figure 6-1: Damage states after (a) testing of original specimen ‘5’, and (b) testing of repaired specimen ‘5’, after [150]*

The beams were then repaired by epoxy injection without correcting the residual drift present after
the initial loading. The same cyclic protocol was used in retesting, except all repaired beams were
retested at a quasi-static strain rate. The beams again exhibited ductile plastic hinging, but the plastic
hinge zones all shifted away from the repaired area, to various degrees [e.g. as shown in Figure
6-1(b)]. The ultimate strengths of the repaired beams were higher than the original specimens in all
cases. If the spike in yield strength of the original dynamically loaded specimens is neglected, the
strength increases were on the order of 10-20%. Note that the spike in yield strength also increased
the secant stiffness to yield of the dynamically loaded specimens, making direct comparison of
post-repair and pre-repair stiffness not appropriate. The specimen that was tested quasi-statically both before and after repair exhibited an increase in secant stiffness to yield of about 10% after repair, based on a visual estimate from the load-deformation plots. Fracture of longitudinal reinforcement occurred at drift ratios of approximately 2.5-3.0% in three of the four repaired specimens, before completion of the full loading protocol that was applied to the original specimens. The cycle-to-cycle energy dissipation of the repaired specimens up until the point of failure was similar to that of the original specimens.

6.1.2 Lee, Wight, and Hanson (1976) [152]

Two exterior beam-column assemblies were tested under reversed-cyclic loading (note that the other six specimens in this report are not considered here due to the repair method involving replacement of core concrete and straightening of buckled bars). The beams had a 254x210mm cross-section, asymmetrical longitudinal reinforcement, a shear span to depth ratio of 4.7, and two different transverse reinforcement layouts. The loading protocol consisted of three cycles at an estimated displacement ductility of 2.0, followed by three cycles at a ductility of 4.0, followed by another three cycles at a ductility of 2.0. The corresponding beam drift demands are not reported or readily calculable. The specimens both exhibited ductile plastic hinging in the beams, as shown in Figure 6-2(a).

![Figure 6-2: (a) damage state after testing of original (unrepaired) specimen ‘3’; (b) peak strengths in the first six loading cycles for the two specimens, both before and after repair, after [152]](image)

The specimens were then repaired by epoxy injection. It is unclear if residual drift was present at the time of repair. The specimens were retested using the same cyclic protocol and again exhibited ductile beam hinging, although photographs of the damage were not provided. The ultimate strengths of the repaired specimens were as much as 25% higher than those of the original specimens [Figure 6-2(b)]. In the two repaired specimens, the secant stiffness to 60% of the yield load was reported to be 0.93 and 1.03 times that of the original specimens, respectively. Observations from the load-deformation plots showed that these stiffness values are reasonable
approximations for the secant stiffness to yield. The specimens were not tested to a point of failure and therefore the effects of the repair on deformation capacity could not be assessed. The cycle-to-cycle energy dissipation was similar in both the repaired and original specimens.

6.1.3 French, Thorp, and Tsai (1990) [153]

Two nominally identical interior beam-column assemblies were tested cyclically. The beams had a 508x305mm cross section, a longitudinal reinforcement ratio of 0.7%, a shear span to depth ratio of 4.3, and stirrups spaced at 102mm. The column width was 381mm, or 15 times the longitudinal bar diameter ($d_b$). The column width was intentionally chosen to be less than the recommended minimum width of $20d_b$, to investigate the ability of epoxy resin to restore bond degradation in the beam-column joint. The loading protocol matched that used by Lee et al. [152], with a maximum applied displacement ductility of 4.0. Similar to the Lee et al. results, the corresponding beam drift demands are not reported or readily calculable. The induced damage was not discussed in detail, and photographs were not provided, but the general discussion and hysteretic behaviour indicate that plastic hinging occurred in the beams, with some cracking and bond degradation in the beam-column joint.

The specimens were then repaired using epoxy-injection on one specimen and epoxy vacuum impregnation, a similar procedure involving a plastic mesh being sealed around the damaged area and filled with epoxy resin, on the other. It is unclear if residual drift was present at the time of repair. The specimens were retested using the same cyclic protocol. In one of the specimens, a relocation of the plastic hinge zone away from the repaired area was reported. The ultimate strength of this repaired specimen was 5% higher than its original equivalent, while the ultimate strength of the other repaired specimen was the same as in the original test. The secant stiffness to approximately 60% of the ultimate strength was reported to be reduced in the two repaired specimens by 11% and 15%. These stiffness values are here taken as approximations for the secant stiffness to yield, in lieu of better data. The original specimens were not tested to a point of failure and therefore the effects of the repair on deformation capacity could not be assessed. The cycle-to-cycle energy dissipation was as much as 20% lower in the repaired specimens than in the originals, due to increased pinching behaviour. This was attributed by the authors to a degradation of anchorage bond in the joint, and an inability of the epoxy resin to penetrate far enough into the joint to restore the bond strength.

6.1.4 Lehman, Gookin, Nacamuli, and Moehle (2001) [154]

A circular bridge column was subjected to reversed-cyclic loading and then repaired after reaching a moderate damage level, after which it was retested until the point of failure (the other three repaired specimens of this paper are not here considered due to the severity of damage and invasive
repair techniques). A nominally identical column was tested to failure without any intermediate repair, allowing for comparison. The columns had a diameter of 610mm, a longitudinal reinforcement ratio of 1.5%, a shear span to depth ratio of 4.0, spiral reinforcement with a pitch of 32mm, and an axial load of approximately $0.07A_f f'_c$. The loading protocol consisted of incrementally increasing cycles with three cycles at each increment. The column that was repaired was subjected to a peak drift of 3.1% prior to the repair, and exhibited damage including spalling of cover concrete and yielding of longitudinal reinforcement, as shown in Figure 6-3(a). The measured peak strain in the longitudinal reinforcement prior to repair was 0.03 and the maximum residual crack width was 4.8mm.

The repair involved epoxy-injection of cracks and patching of spalled cover concrete. It is unclear if residual drift was present at the time of repair. The repaired specimen was then retested by applying the complete cyclic protocol to failure (i.e. the protocol that was applied to the unrepaired specimen). Similar damage progression was observed in both the repaired and unrepaired specimens. The ultimate strengths of the repaired and unrepaired columns were approximately the same. The secant stiffness to 80% of the ultimate strength, here taken as an approximation for the secant stiffness to yield, was estimated to be reduced by as much as 50% in the repaired specimen. The strength degradation, deformation capacity, and energy dissipation characteristics were similar in both the repaired and unrepaired columns, as shown in Figure 6-3(b).

6.1.5 Cuevas, and Pampanin (2017) [151]

Two beam-column assemblies were extracted from an earthquake-damaged building that was scheduled for demolition, and were tested in a laboratory setting. The specimens were nominally identical, with the beams having a cross section of 1100x575mm, a shear span to depth ratio of 2.3,
and stirrups spaced at 150mm. The beams had additional longitudinal reinforcement adjacent to the column, which was intended to relocate the plastic hinges away from the column face; however, much of the plastic hinging damage occurred adjacent to the column regardless. The earthquake damage that was present prior to extraction of the specimens was relatively minor, with no residual crack widths greater than 1.0mm. A higher level of damage was desired prior to repair, and so the specimens were tested cyclically, using a loading protocol with three cycles at each increment, until peak demands of approximately 0.6% (specimen ‘2’) and 1.1% (specimen ‘3’) beam drift had been imposed. As shown in Figure 6-4, the specimens exhibited plastic hinging in the beams, with maximum crack widths of 2.0mm in the specimen tested to 0.6% drift and 6.0mm in the specimen tested to 1.1% drift.

![Figure 6-4](image)

*Figure 6-4: Damage states immediately prior to repair for (a) specimen ‘2’, and (b) specimen ‘3’, after [151]*

The specimens were then repaired by epoxy injection and were retested by applying the same cyclic loading protocol to failure. It is not clear if the residual drift was corrected prior to repair. The post-repair damage progression of specimen ‘2’ was comparable to that of a separate non-repaired specimen (the non-repaired specimen was not taken to failure and therefore is not further discussed here), with plastic hinging adjacent to the column face. The post-repair damage progression of specimen ‘3’ involved plastic hinging, with some of the damage being relocated away from the column face (i.e. outside of the additional longitudinal reinforcement detail). The repaired specimens both exhibited strength increases of between 5-10% as compared to the original tests. The secant stiffness values to a visually estimated yield point were similar in the repaired and original tests of specimen ‘2’, but were reduced on the order of 10% in repaired specimen ‘3’. The repaired specimens began to exhibit significant cyclic strength degradation during the cycles corresponding to a beam drift of approximately 3.0%, with failure occurring during cycles to the subsequent loading increment of approximately 4.5% beam drift. Cyclic strength degradation was more severe in repaired specimen ‘2’ than ‘3’, despite having been repaired after a lower pre-repair drift demand. The energy dissipation characteristics were similar in the repaired and original tests.
6.2 Experimental results

Three beam specimens from the experimental program described in Chapter 2 were repaired by epoxy-injection and reconstitution of spalled cover concrete following an initial damaging long-duration earthquake loading (LD-1-R, LD-2-R, and LD-2-LER-R). Each of these specimens had an equivalent specimen that was not repaired following its initial damaging earthquake loading (LD-1, LD-2, and LD-2-LER, respectively). Comparison between the cyclic behaviours of these two sets of specimens is here used to evaluate the effects of epoxy-injection repair relative to a condition of no repair. Where possible, comparisons are also made with the behaviour of other non-repaired nominally identical specimens to evaluate the ability of epoxy-injection repair to restore the original structural performance. It should be noted that the residual drift of the repaired beams was not corrected prior to repair. Specimens LD-1-R, LD-2-R, and LD-2-LER-R had residual drifts at the time of repair of -0.26%, -0.70%, and -0.75%, respectively. For a detailed description of the repair process used, see Section 2.6.

6.2.1 Damage state prior to repair

Photographs showing the damage states of the three repaired specimens immediately prior to the repair are shown in Figure 6-5. In specimens LD-2-R and LD-2-LER-R, which were subjected to a peak drift of 2.17% during the initial earthquake loading, damage consisted of flexural cracking, longitudinal cracks along longitudinal the reinforcement, and minor delamination of cover concrete. Specimen LD-1-R also exhibited delamination of cover concrete, despite only being subjected to a peak drift of 1.36%, and damage was concentrated around a wide sliding plane crack through the depth of the member. The maximum residual crack widths ranged from 2.5-3.5mm.

The damage that developed during the initial earthquake loadings in the equivalent repaired and unrepaired specimen pair LD-2-LER-R and LD-2-LER was similar in both beams. However, as previously shown in Figure 3-23, there was significant variance in the damage that developed during the initial earthquake loadings in the unrestrained equivalent specimen pairs LD-1-R & LD-1 and LD-2-R & LD-2, despite each specimen having had identical loadings applied. Specimen LD-1 exhibited only flexural and longitudinal cracking during the initial earthquake loading, which was noticeably less severe than the damage to LD-1-R. Based on these damage states, if no repair had been performed, specimen LD-1-R would have been expected to exhibit poorer performance than LD-1. Conversely, specimen LD-2 exhibited concentrated damage which included a wide sliding plane crack and significant spalling of cover concrete, a noticeably more severe damage state than that of LD-2-R. Therefore, even without repair, specimen LD-2-R would have been expected to improved performance, relative to LD-2. The comparative results between these equivalent specimen pairs should be considered in the context of this expected behaviour.
The effects of epoxy injection repair of moderate plastic hinging damage

Figure 6-5: Damage states of the three repaired specimens after the initial earthquake loading and immediately prior to repair
6.2.2 Post-repair damage progression

Photographs of the three repaired specimens after completion of both loading cycles to 2.44% drift (the lowest post-repair drift increment applied to specimens LD-2-R and LD-2-LER-R) are shown in Figure 6-6. The repaired specimens generally exhibited more distributed damage (i.e. a longer plastic hinge zone) than the other cyclic specimens. In the repaired specimens, measurable cracks (cracks with widths greater than 0.2mm) developed up to an average distance of 540mm from the beam-foundation interface. In non-repaired cyclic specimens, the average distance from the beam-foundation interface along which measurable cracks appeared was 430mm. However, a complete shifting of the plastic hinge zone, as was observed in some of the specimens reviewed in Section 6.1, did not occur. Re-appearance of plastic hinging damage occurred within the repaired area. Furthermore, the more distributed damage that developed early in the repaired beam tests did not result in the formation of different ultimate failure modes than was observed in non-repaired specimens.

In all repaired beams, longitudinal cracks re-appeared at the interface between the mortar used to repair the delaminated concrete and the substrate concrete, indicating that the bond between these materials was weaker than their tensile strengths. In specimen LD-2-LER-R this bond was particularly weak, and delamination occurred in the first post-repair loading cycle. In the other two specimens, the point of delamination was consistent with tests on non-repaired specimens.

The epoxy-injection was generally effective in keeping injected cracks closed. However, as the drift demands increased and the damage progressed, some of the repaired cracks re-opened, particularly the large sliding plane crack at the second stirrup from the interface in specimen LD-1-R. Nonetheless, the epoxy-injection improved the damage distribution in LD-1-R, relative to what would have been expected without repair based on the damage pattern exhibited prior to repair [Figure 6-5(a)].

6.2.3 Stiffness

In Figure 6-7 (unrestrained specimens) and Figure 6-8 (-LER specimens), the first ¼ cycle of cyclic loading for the repaired and unrepaired equivalent specimen pairs are compared in order to assess the effect of epoxy-injection on the restoration of stiffness, relative to the residual stiffness in an as-damaged state. For the purposes of this analysis, the secant stiffness to the shear force corresponding to 0.8\(M_n\), where \(M_n\) is the nominal flexural strength of the beams calculated using NZS 3101:2006, is taken as a proxy metric for the secant stiffness to yield. When calculating \(M_n\) for the restrained LD-2-LER specimens, an axial compression force of 125kN was accounted for, which approximately corresponds to the measured compression forces at first yield of the repaired specimen.
The effects of epoxy injection repair of moderate plastic hinging damage

Figure 6-6: Photographs (rotated 90°) of the three repaired specimens after completion of both loading cycles to 2.44% drift. The length along which damage was distributed (other than hairline flexural cracking) is marked by red arrow.
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Figure 6-7: Secant stiffness to 80% of nominal strength after the initial earthquake loading in repaired and unrepaired equivalent specimen pairs (a) LD-1, and (b) LD-2
The secant stiffness to 0.8\(M_n\) was calculated using Equation (6-1) and is reported for the three specimen pairs in Table 6-1. Stiffness values are reported in kN/mm and as a ratio of \(E_c I_g\), where \(E_c\) is calculated as per the recommendations of NZS 3101:2006. The epoxy injection was effective in increasing stiffness by as much as 250% relative to the unrepaired equivalents. The relative stiffness increase was higher in the LD-2 and LD-2-LER specimen pairs than in the LD-1 specimen pair, due to the higher drift demand and corresponding increased stiffness degradation that occurred prior to repair in those specimens. However, the absolute post-repair stiffness of specimen LD-2-R was approximately 10% less than that of LD-1-R.

\[
K = \frac{0.8M_n}{(\Delta_{0.8M_n} - \Delta_{\text{resid}})} \tag{6-1}
\]

where \(K\) is the secant stiffness, \(\Delta_{0.8M_n}\) is the lateral displacement at the first instance of reaching 0.8\(M_n\), and \(\Delta_{\text{resid}}\) is the residual displacement after the earthquake loading.

Figure 6-8: Secant stiffness to 80% of nominal strength after the initial earthquake loading in repaired and unrepaired equivalent lightly restrained specimen pair LD-2-LER
The effects of epoxy injection repair of moderate plastic hinging damage

Table 6-1: Effect of epoxy-injection repair on secant stiffness to 0.8Mₙ in repaired and unrepaired equivalent specimen pairs

<table>
<thead>
<tr>
<th>Specimen pair</th>
<th>Repaired specimen stiffness (kN/mm / ratio EᵣIᵧ)</th>
<th>Equivalent unrepaired specimen stiffness (kN/mm / ratio EᵣIᵧ)</th>
<th>Percentage increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>LD-1</td>
<td>10.25 / 0.23</td>
<td>6.30 / 0.14</td>
<td>63%</td>
</tr>
<tr>
<td>LD-2</td>
<td>9.17 / 0.20</td>
<td>3.68 / 0.08</td>
<td>149%</td>
</tr>
<tr>
<td>LD-2-LER</td>
<td>10.26 / 0.23</td>
<td>4.25 / 0.09</td>
<td>141%</td>
</tr>
</tbody>
</table>

It is also of interest to compare the stiffness of the epoxy-repaired specimens with the initial stiffness of undamaged specimens. Comparison of the repaired specimen stiffness with its own initial undamaged stiffness is not appropriate due to the initial earthquake loading having been applied dynamically and the post-repair cyclic loading having been applied statically. The stiffness values of the two unrestrained repaired specimens are therefore instead compared against the initial stiffness of quasi-static undamaged specimens MONO, CYC, and CYC-NOEQ, as shown in Figure 6-9. The corresponding data are provided in Table 6-2. It should be noted that in Figure 6-9 the load-deformation responses of the repaired specimens have been translated to a state of zero residual drift, to facilitate visual comparison with the undamaged specimens.

Figure 6-9 shows that the repair by epoxy-injection was not able to restore the tension stiffening behaviour of the beams. This is due to the epoxy not being able to penetrate very small cracks (in the repair, cracks with a width less than 0.2mm were not injected). This resulted in secant stiffness values to 0.8Mₙ 27-35% lower than the average stiffness of the three undamaged specimens. However, due to the tension stiffening effect, the secant stiffness values of the undamaged specimens are heavily dependent on the 0.8Mₙ point selected. If the secant stiffness is instead measured to a force of 105kN, which was visually estimated to be the highest shear force prior to noticeable stiffness changes near the yield point, the repaired specimens exhibit stiffness values only 12-21% lower than the average stiffness of the undamaged specimens. This secant stiffness to a point visually estimated to be just below yield is considered to be more representative of the true secant stiffness to yield.

In Figure 6-10, the stiffness of the repaired restrained specimen LD-2-LER-R is compared against the stiffness of quasi-static undamaged specimen CYC-LER. It is here noted that these specimens had different axial compression forces, with approximately 125kN in LD-2-LER-R and 50kN in CYC-LER, and that increased axial compression causes an increase in the secant stiffness to yield [115]. The percentage of stiffness restoration due to the repair, reported in Table 6-3, is therefore somewhat higher than would be expected if the same axial load had been present in both specimens. Similar to the unrestrained specimens, secant stiffness comparisons are made at two points, corresponding to 0.8Mₙ (with Mₙ calculated using an axial compression force of 125kN) and a
force visually estimated to be just before noticeable yielding (115kN). The stiffness of the repaired specimen was 28% and 15% lower than the undamaged specimen for the two cases of secant to 0.8\(M_n\) and secant to 115kN, respectively. Again, the secant stiffness to the point visually estimated to be just below yield is here considered to be the better metric for approximating secant stiffness to yield.

\[\text{Table 6-2: Secant stiffness data for repaired and undamaged specimens without axial restraint (quasi-static loading only)}\]

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Secant stiffness to 0.8(M_n) (kN/mm / ratio (E_c I_g))</th>
<th>Secant stiffness to 105kN (kN/mm / ratio (E_c I_g))</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Undamaged specimens</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MONO</td>
<td>15.88 / 0.35</td>
<td>12.96 / 0.29</td>
</tr>
<tr>
<td>CYC</td>
<td>13.73 / 0.31</td>
<td>11.09 / 0.25</td>
</tr>
<tr>
<td>CYC-NOEQ</td>
<td>12.36 / 0.28</td>
<td>10.07 / 0.22</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td>13.99 / 0.31</td>
<td>11.37 / 0.25</td>
</tr>
<tr>
<td><strong>Repaired specimens</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LD-1-R</td>
<td>10.26 / 0.23</td>
<td>10.04 / 0.22</td>
</tr>
<tr>
<td>(73% of undamaged average)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LD-2-R</td>
<td>9.17 / 0.20</td>
<td>8.96 / 0.20</td>
</tr>
<tr>
<td>(66% of undamaged average)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Figure 6-9: Comparison of initial stiffness between repaired and undamaged specimens without axial restraint (repaired specimens shown in red; undamaged specimens shown in black). Only quasi-static undamaged specimens are included.*
The effects of epoxy injection repair of moderate plastic hinging damage

Figure 6-10: Comparison of initial stiffness between repaired (LD-2-LER-R) and quasi-static undamaged (CYC-LER) lightly restrained specimens

Table 6-3: Secant stiffness data for repaired and undamaged lightly restrained (-LER) specimens (quasi-static loading only)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Secant stiffness to 0.8M_n (kN/mm / ratio E_cI_g)</th>
<th>Secant stiffness to 115kN (kN/mm / ratio E_cI_g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CYC-LER</td>
<td>14.17 / 0.32</td>
<td>11.85 / 0.26</td>
</tr>
<tr>
<td>LD-2-LER-R</td>
<td>10.27 / 0.23</td>
<td>10.11 / 0.23</td>
</tr>
<tr>
<td></td>
<td>(72% of CYC-LER)</td>
<td>(85% of CYC-LER)</td>
</tr>
</tbody>
</table>

6.2.4 Strength

Table 6-4 compares the ultimate strengths reached by the repaired specimens with the strengths of (i) their equivalent unrepaired specimens, and (ii) the average of all comparable non-repaired specimens. Only peak strengths corresponding to the static loading portion of the tests are considered, so as to isolate the effects of repair from the effects of dynamic loading. The ultimate strengths reached by repaired specimens LD-1-R (125kN) and LD-2-R (129kN) were the highest strengths achieved during static cyclic loading for any of the unrestrained specimens, but still below the overstrength (131kN), calculated using a longitudinal reinforcement yield strength of 1.35f_y as per NZS 3101:2006. The strength increases due to the repair were modest, ranging from 4-7% relative to the average non-repaired specimen strength. A similar strength increase of 7% was
observed in repaired specimen LD-2-LER-R, relative to the average ultimate strength of the two non-repaired -LER specimens.

Table 6-4: Ultimate strengths of repaired specimens and comparisons with strengths of non-repaired specimens (quasi-static loading only – ultimate strengths reached during dynamic loading are neglected)

<table>
<thead>
<tr>
<th>Repaired specimen</th>
<th>Repaired specimen strength (kN)</th>
<th>Equivalent unrepaired specimen strength (kN)</th>
<th>Strength ratio</th>
<th>Average non-repaired specimen strength (kN)</th>
<th>Strength ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>LD-1-R</td>
<td>125</td>
<td>123</td>
<td>1.01</td>
<td>120.1¹</td>
<td>1.04</td>
</tr>
<tr>
<td>LD-2-R</td>
<td>129</td>
<td>115</td>
<td>1.12</td>
<td>120.1¹</td>
<td>1.07</td>
</tr>
<tr>
<td>LD-2-LER-R</td>
<td>158</td>
<td>148</td>
<td>1.07</td>
<td>147.5²</td>
<td>1.07</td>
</tr>
</tbody>
</table>

¹Taken as the average of eight cyclic unrestrained specimens that were partially or fully tested quasi-statically
²Taken as the average of the two unrepaired -LER specimens

6.2.5 Energy dissipation

Figure 6-11 shows the energy dissipation in each half-cycle for the equivalent repaired and unrepaired specimen pairs. The average cycle-to-cycle energy dissipation of all non-repaired specimens with the same restraint conditions is also included in Figure 6-11. Specimens LD-1-R and LD-2-R both had increased cycle-to-cycle energy dissipation in the cycles immediately following the repair, with the effect more pronounced in specimen LD-2-R. The energy dissipation of specimen LD-1-R subsequently degraded to be similar to that of the non-repaired specimens, while it remained higher in specimen LD-2-R. Specimen LD-2-LER-R had increased energy dissipation only in the first three half-cycles following repair, after which it was similar to that of the other –LER specimens.

6.2.6 Elongation

The repaired beams exhibited elongation characteristics comparable to undamaged beams. Figure 6-12 shows how the post-repair elongation versus drift response (shown for cycles up to 2.71% drift) of LD-2-R was similar to undamaged specimen CYC-NOEQ. The equivalent unrepaired specimen LD-2 exhibited a magnitude of elongation during those same loading cycles on the order of 1/3rd that of the repaired or undamaged specimens. The minor differences between the elongation behaviour of LD-2-R and CYC-NOEQ during the first loading cycles are likely due to the residual drift of LD-2-R at the time of repair.
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Figure 6-11: Cycle-to-cycle energy dissipation of the three repaired specimens (a) LD-1-R, (b) LD-2-R, and (c) LD-2-LER-R, and comparison with energy dissipation of non-repaired specimens
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Figure 6-12: Elongation versus drift relationships for loading cycles up to 2.71\% drift for specimens LD-2 (post-earthquake only), LD-2-R (post-repair only), and CYC-NOEQ (undamaged prior to these loading cycles)

In Figure 6-13, the elongation versus drift relationships of repaired specimens LD-1-R and LD-2-R are compared against the NZS 3101:2006 elongation equation for reversing plastic hinges [Equation (3-9)] (using $\theta_m = \theta_p$). When considering the pre-repair and post-repair elongations to be cumulative [Figure 6-13(a)], both specimens had larger elongations than would be predicted by the NZS 3101:2006 equation and both exceeded the 0.036$h_b$ limit. Specimen LD-1-R only marginally exceeded the limit, but LD-2-R substantially exceeded it, reaching a peak elongation of approximately 0.05$h_b$, the largest of any test specimen. The residual elongation following the initial earthquake loading for specimens LD-1-R and LD-2-R were 6.1mm and 11.6mm, respectively. If the post-repair elongation data are corrected to remove these residual elongations [Figure 6-13(b)], the NZS 3101:2006 equation conservatively predicts the elongation and the 0.036$h_b$ limit more accurately bounds the behaviour.

Lightly restrained specimen LD-2-LER-R was repaired with 4.6mm of residual elongation present after the initial earthquake loading, and the cumulative elongation (i.e. including the residual elongation at the time of repair) was the largest exhibited by any of the three –LER specimens. However, the differences between the maximum elongations of the repaired and non-repaired –LER specimens were relatively modest, likely due to (i) the relatively small residual elongation at the time of repair, and (ii) any additional elongation having resulted in additional restraining compressive forces.
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Figure 6-13: Elongation versus drift relationships of unrestrained repaired specimens, showing (a) cumulative pre-repair and post-repair elongation, and (b) post-repair elongation only

6.2.7 Deformation capacity

The three repaired specimens all first reached a 20% drop in strength during loading cycles to 4.34% drift. In specimen LD-2-R, this represented an increased deformation capacity over its equivalent unrepaired specimen LD-2, but in the other two repaired specimens, the deformation capacity was similar or reduced in the repaired specimens. However, it is important to note that it was previously shown in Section 3.1 that the number of loading cycles at or below 2.17% drift was not correlated to the ultimate deformation capacity. Attempting to draw conclusions about the effect of repair of specimens initially damaged to less than ~2.2% drift on the deformation capacity based only on comparisons between the equivalent repaired and unrepaired specimens would therefore be incompatible with this finding. Since the predominant damage mechanism was found to be correlated with deformation capacity (see Section 2.7.7), a more appropriate comparison would be
comparing the deformation capacity of the repaired beams in the context of their damage states at the point of repair.

Specimen LD-1-R was repaired after developing damage during the initial earthquake loading that was concentrated around a sliding plane crack located at the second stirrup from the beam column interface. This damage mechanism also developed in a number of other specimens which exhibited relatively low deformation capacities of 3.26-3.8% drift (see Section 2.7.7). The increased drift capacity of LD-1-R to 4.34%, relative to these specimens, implies that the repair was effective in restoring deformation capacity. In specimen LD-2-R the drift capacity of 4.34% is comparable to what would have been expected regardless of repair, based on comparisons with non-repaired specimens with similar damage patterns. The same is true for specimen LD-2-LER-R, as all restrained specimens exhibited drift capacities of 4.34%.

The fact that the drift capacities of the repaired specimens were comparable to the non-repaired specimens implies that the other effects of repair that were previously discussed (e.g. increased length along which damage was distributed, increased cumulative elongation) did not in turn affect the deformation capacity. The lack of reduction in deformation capacity is despite, in theory, a larger ‘effective drift’ having been applied to the repaired specimens, since the repairs were conducted on specimens with uncorrected residual drift. The two unrestrained specimens failed in both the positive and negative loading directions of the same cycle, further implying that the residual drift at the time of repair had little influence on deformation capacity.

6.3 Assessing the residual capacity of plastic hinges that have been repaired by epoxy injection

In this section, data from fourteen tests, consisting of the eleven tests that were identified in the literature (Section 6.1) and the three tests conducted as part of the experimental program on reinforced concrete beams (Section 6.2), are used to derive recommendations for the residual capacity of epoxy-repaired components. These recommendations are intended to be applicable to reinforced concrete beams that have formed plastic hinges with damage no more severe than cracking, yielding of longitudinal reinforcement, and spalling of cover concrete. The recommendations serve as a starting point for columns, but further research is required due to the limited test data on epoxy-repaired reinforced concrete columns available in the literature. It is also noted that lower quality repairs may be more likely to occur in practice than in laboratory settings, but no consideration is given to this issue here; further research is recommended.

A summary of the key data from the fourteen tests is given in Table 6-5. Visually estimated values of displacement ductility were used as a deformation metric due to the difficulty of extracting accurate beam drifts from the Lee et al. [152] and French et al. [153] test results. In all fourteen
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specimens, the ultimate post-repair strength met or exceeded the ultimate strength of the original tests. The secant stiffness to yield was in some cases difficult to quantify based on the provided data, but post-repair stiffness values were in most cases lower than those of the original tests, particularly in specimens where significant strength increases did not occur. Energy dissipation was similar in the original and repaired tests in all specimens except those of the French et al. [153] test program, in which the specimens exhibited lower energy dissipation and increased pinching of the hysteretic load-deformation response after repair. However, this was attributed to degradation of bond in the beam-column joint, which did not meet the column width requirements of modern concrete design codes (e.g. [1, 2]). The deformation capacities of the repaired specimens were similar to tests on equivalent unrepaired specimens in most cases, but the deformation capacity of the beams tested by Celebi and Penzien [150] were reduced due to earlier fracture of longitudinal reinforcement. The early fracture in the Celebi and Penzien tests may have been a result of low-cycle fatigue, as the loading protocol involved 20 post-yield cycles, both before and after repair.

Table 6-5: Summary of data from experiments that used epoxy injection repair techniques on ductile plastic hinge zones subjected to cyclic loading

<table>
<thead>
<tr>
<th>Study</th>
<th>Component type</th>
<th>Specimen name</th>
<th>Approx. ductility before repair</th>
<th>Secant stiffness ratio¹</th>
<th>Ultimate strength ratio¹</th>
<th>Energy dissipation</th>
<th>Deformation capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Celebi and Penzien (1973)</td>
<td>Beams</td>
<td>Celebi-5</td>
<td>3.5</td>
<td>N/A²</td>
<td>1.11³</td>
<td>Similar</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Celebi-7</td>
<td>4</td>
<td>N/A²</td>
<td>1.21³</td>
<td>Similar</td>
<td>Reduced</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Celebi-9</td>
<td>4</td>
<td>N/A²</td>
<td>1.19³</td>
<td>Similar</td>
<td>Reduced</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Celebi-10</td>
<td>5</td>
<td>~1.1</td>
<td>1.17</td>
<td>Similar</td>
<td>Reduced</td>
</tr>
<tr>
<td>Lee et al. (1976)</td>
<td>Exterior beam-column assemblies</td>
<td>Lee-1</td>
<td>4</td>
<td>0.93</td>
<td>1.28</td>
<td>Similar</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lee-3</td>
<td>4</td>
<td>1.03</td>
<td>1.24</td>
<td>Similar</td>
<td>N/A</td>
</tr>
<tr>
<td>French et al. (1990)</td>
<td>Interior beam-column assemblies</td>
<td>French-RVI</td>
<td>4</td>
<td>0.85</td>
<td>1.01</td>
<td>Reduced</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>French-RPI</td>
<td>4</td>
<td>0.89</td>
<td>1.05</td>
<td>Reduced</td>
<td>N/A</td>
</tr>
<tr>
<td>Lehman et al. (2001)</td>
<td>Bridge columns</td>
<td>Lehman-415MR</td>
<td>3.5</td>
<td>~0.5</td>
<td>1.00</td>
<td>Similar</td>
<td>Similar</td>
</tr>
<tr>
<td>Cuevas and Pampanin (2017)</td>
<td>Beams</td>
<td>Cuevas-2</td>
<td>1.2</td>
<td>~1.0</td>
<td>1.05</td>
<td>Similar</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Cuevas-3</td>
<td>2.2</td>
<td>~0.9</td>
<td>1.05</td>
<td>Similar</td>
<td>N/A</td>
</tr>
<tr>
<td>Marder (2018)</td>
<td>Beams</td>
<td>LD-1-R</td>
<td>3.4</td>
<td>0.88</td>
<td>1.04</td>
<td>Similar</td>
<td>Increased</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LD-2-R</td>
<td>5.4</td>
<td>0.79</td>
<td>1.07</td>
<td>Similar</td>
<td>Similar</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LD-2-LER-R</td>
<td>5.4</td>
<td>0.85</td>
<td>1.07</td>
<td>Similar</td>
<td>Similar</td>
</tr>
</tbody>
</table>

¹The ratio refers to the value of the parameter for the repaired specimen, divided by the value for the original, unrepaired specimen (except in the cases of Lehman et al. and Marder, where the comparisons are against nominally identical undamaged specimens).
²The stiffness values before and after repair are not comparable due to different loading rates.
³The strength ratio neglects the spike at yield that occurred in the original, unrepaired specimens due to dynamic loading rates.
6.3.1 **Effect of residual deformations prior to repair**

The data that form the basis of this discussion is predominantly from the three repaired beam specimens discussed in Section 6.2, as the other experiments identified in the literature did not typically state the residual deformations prior to repair.

The presence of residual elongation at the time of repair can cause an increase in the maximum cumulative elongation (i.e. the sum of the pre-repair residual elongation and the post-repair elongation) of a plastic hinge, relative to the maximum elongation that would be likely to occur if no repair was conducted (see Section 6.2.6 for more information). It is recommended that this can be accounted for by assuming that the maximum elongation of the repaired hinge (i.e. the elongation that is additional to the residual elongation prior to repair) is the same as the maximum elongation that would be calculated for an identical undamaged component. The maximum cumulative elongations of repaired reversing plastic hinges with no axial load can therefore be estimated by adding the residual elongation at the time of repair to the NZS 3101:2006 elongation equation, as shown in Equation (6-2). It is noted that this may be conservative, as greater cumulative elongations may induce higher axial compressive loads due to interactions with the surrounding structure. However, this effect cannot readily be quantified and is therefore here ignored. Plastic hinges in members with high levels of axial compression (i.e. with a compressive force greater than \(0.08A_f f'_c\) based on the NZS 3101:2006 equations) are likely to have minimal residual elongation and therefore can typically be assumed to have a negligible increase in maximum cumulative elongation after repair.

\[
e_{\text{cumulative}} = e_{\text{resid}} + 2.6 \frac{\theta_m}{2} (d - d') \leq e_{\text{resid}} + 0.036h_p
\]  

where \(e_{\text{cumulative}}\) is the cumulative maximum expected elongation of the repaired plastic hinge, \(e_{\text{resid}}\) is the measured residual elongation at the time of repair, \(\theta_m\) is the maximum expected rotation demand on the repaired plastic hinge, \(d\) and \(d'\) are the depth from extreme compression fiber to the tension steel and compression steel, respectively, and \(h_p\) is the member depth.

If damage in a repaired plastic hinge re-develops within the original plastic hinge zone, any increase in total elongation must be accompanied by an increase in longitudinal reinforcement strain. This could have effects on both the strength (due to higher levels of strain hardening) and deformation capacity of the repaired plastic hinge. The magnitude of additional strain that has the potential to occur in the longitudinal reinforcement of a repaired plastic hinge can be approximated by assuming the residual elongation at the time of repair is uniformly spread across an estimated plastic hinge length, as shown in Equation (6-3). The assumptions inherent in this calculation are illustrated in Figure 6-14.
\[ \varepsilon_{\text{avg}} = \frac{e_{\text{resid}}}{L_p} \]  

(6-3)

where \( \varepsilon_{\text{avg}} \) is the average residual strain across the equivalent plastic hinge length at the time of repair, \( e_{\text{resid}} \) is the residual elongation at the time of repair, and \( L_p \) is the equivalent plastic hinge length (e.g. \( L_p = 0.08L + 0.022d_{bf}y \) [105]).

As previously discussed in Section 5.3, use of equivalent plastic hinge length assumptions are not applicable prior to the development of strain hardening in the longitudinal reinforcement. However, the purpose of Equation (6-3) is to predict the maximum expected increase in strain that is likely to occur at the ultimate limit state of a repaired hinge, relative to the maximum expected strain at the ultimate limit state in an identical undamaged hinge. In ductile members, the ultimate limit state occurs after a well-distributed plastic hinge has developed, and it is therefore here assumed that localized fluctuations in the strain profile along the length of the longitudinal reinforcement at the time of repair are not of significance, as a more distributed strain profile would be expected to develop upon further loading. Equation (6-3) is therefore applicable in any situation where a well-distributed plastic hinge is expected to form, even if only a single crack is present at the time of repair. However, it must be emphasised that \( \varepsilon_{\text{avg}} \) is not a measure of the actual strain in the reinforcement at the time of repair.

The presence of residual drift at the time of repair (and the associated residual strain gradient) can similarly alter the strain demands in a repaired plastic hinge, relative to what would occur in an unrepaired equivalent. However, this effect is expected to be modest in practical situations, as plastic hinges with significant residual drift are only likely to occur in buildings with substantial out-of-plumb, and such buildings are unlikely to be economical to repair. In the repaired beam specimens discussed in Section 6.2, residual drifts between 0.26-0.75% were found to have no impact on the post-repair deformation capacity. It is here recommended that residual drifts (i.e. member drift or chord rotation) of 0.75% or below are ignored when assessing post-repair residual capacity of plastic hinges. Further experiments are required to determine the efficacy of epoxy injection repair on plastic hinges with larger residual drifts.

6.3.2 Post-repair damage development scenarios

The fourteen test specimens of Table 6-5 exhibited a variety of post-repair damage progressions. Figure 6-15 illustrates three possible scenarios with regards to the re-occurrence of plastic hinging damage in an area that has previously been repaired. Both plastic hinge relocation [Figure 6-15(a)] and re-formation of damage within the repaired plastic hinge [Figure 6-15(c)] can have consequences with regards to the post-repair residual capacity, with the underlying reasons being
largely mutually exclusive. An intermediate case, where damage partially occurs within the repaired area and partially relocates, is also possible [Figure 6-15(b)].

Plastic hinge relocation occurs if the strength of the repaired area increases by an amount sufficient to prevent further damage from developing in that region, as further discussed in Section 6.3.4. A potential complication of plastic hinge relocation is increased curvature demands for a given drift demand, due to a reduced shear span length. Re-formation of plastic hinge damage within the repaired area occurs if the strength of the repaired area does not increase enough to force hinge relocation. This can result in higher longitudinal reinforcement strains (due to additional cumulative elongation within the hinge, as previously discussed) and the re-loading of reinforcement that was previously subjected to inelastic demands. In either case (relocated or non-relocated), an increase in shear force occurs as a result of any increased moment capacity in the repaired region.

In some cases, it may be desirable to force relocation of the plastic hinge to occur, in order to protect the previously yielded longitudinal reinforcement. This may be achievable through retrofit with
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fibre-reinforced polymers or jacketing techniques (e.g. [147]), but is outside the scope of this research.

![Figure 6-15: Three possible cases of re-formation of plastic hinging following repair](image)

6.3.3 Stiffness

Eleven tests had data conducive to comparing the ratio of secant stiffness to yield between post-repair and equivalent undamaged cases. Of these eleven tests, ten had stiffness ratios ranging from approximately 0.8-1.1, with the Lehman et al. [154] specimen being an outlier with a stiffness ratio of approximately 0.5. The Lehman et al. specimen was a circular bridge column with axial compression while all other specimens involved plastic hinging occurring in a rectangular beam without axial load (except the compression induced in the one axially restrained specimen LD-2-LER-R). Lehman et al. attributed the reduced stiffness to degradation of the concrete stiffness and strength, and degradation of the bond capacity between the concrete and reinforcing steel. Another possible hypothesis for the lower stiffness is that the axial load may have caused closure of cracks at which the longitudinal reinforcement had previously yielded, limiting the ability of the epoxy resin to penetrate these cracks. It is recommended that additional experimental work be conducted
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on epoxy injection repair of ductile reinforced concrete columns subjected to axial compression to further investigate these hypotheses.

In Figure 6-16, the ratios of repaired to undamaged secant stiffness to yield are compared against the approximate maximum applied displacement ductility prior to repair. It can be seen that the level of displacement ductility that the specimens were subjected to prior to repair was not correlated with the ratio of repaired to undamaged stiffness. This indicates that the demands incurred prior to repair do not necessarily have to be considered when determining the post-repair stiffness. It is here recommended that epoxy injection repair of plastic hinges in beams can be conservatively assumed to restore a secant stiffness to yield that is 80% of that in an identical undamaged beam. This is consistent with the recommendations made by FEMA 306 for epoxy injection of moderately damaged reinforced concrete walls or coupling beams [16].

![Figure 6-16: Ratio of repaired to undamaged stiffness, versus the displacement ductility prior to repair](image)

6.3.4 Strength

The ratio of ultimate flexural strength between the repaired and undamaged specimens ranged from 1.0-1.3, with an average of 1.1. In Figure 6-17, these strength ratios are compared against the approximate maximum applied displacement ductility prior to repair. Similar to the finding for stiffness, the displacement ductility prior to repair was not correlated with the post-repair strength. These data indicate that epoxy-repaired plastic hinges can be expected to have moment capacities at least as high as the original reinforced concrete section, regardless of the demands incurred prior to repair.

The large increases in flexural strength that were observed in some specimens are of concern, as a higher than expected moment capacity could precipitate brittle failure in a structure that has not been capacity designed to accommodate such overstrengths. It is here assumed that strength
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Increases in repaired plastic hinges are largely due to increased longitudinal steel strength, which could be a result of strain ageing or higher levels of strain hardening due to increased cumulative elongation. These sources of strength increase result in the conclusion that higher deformation demands prior to repair are required in order for increased post-repair strength to occur. This is reflected in Figure 6-17 although limited data at low deformation demands are available. The epoxy itself is unlikely to cause a significant increase in member strength because cracks are able to re-form in the adjacent concrete, and the fact that epoxy is typically stronger than concrete in terms of compressive, tensile, and bond strengths is therefore of limited importance for the flexural strength.

Figure 6-17: Ratio of repaired to undamaged flexural strength, versus the displacement ductility prior to repair

The potential increase in strain hardening of the longitudinal reinforcement in a repaired plastic hinge with damage re-forming in the same region [Figure 6-15(c)] can be approximated by comparing the average strain along the equivalent plastic hinge length due to residual elongation at the time of repair [ε_avg, as defined in Equation (6-3)] against the strain hardening properties of the reinforcement. In New Zealand, AS/NZS 4671:2001 stipulates that grade 300E reinforcement must have an f_u/f_y ratio of between 1.15 to 1.5 (1.15 to 1.4 for 500E) and a minimum uniform strain of 0.15 (0.1 for 500E). Assuming a simple bi-linear stress-strain model, these requirements correspond to strain hardening slopes (rate_e(sh)), normalized with respect to the yield stress, as listed in Table 6-6.

The calculated increase in strain hardening of the longitudinal reinforcement in beam specimens LD-1-R and LD-2-R due to the residual elongation at the time of repair are listed in Table 6-7, and a comparison of the upper- and lower-bound AS/NZS 4671:2001 bi-linear models against the measured longitudinal reinforcement stress-strain response is given in Figure 6-18. The Paulay and Priestley [105] plastic hinge length model Lp = 0.08L + 0.022d bf_y is used to calculate ε_avg
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[Equation (6-3)]. The calculated percentage increases in longitudinal steel strength due to upper- and lower-bound \( \text{rate}_{sh} \) values from Table 6-6 effectively bound the measured increases in flexural strengths of the repaired beams, relative to the average strength of the unrepaired beam specimens (Table 6-7). As shown in Figure 6-18, the upper-bound \( \text{rate}_{sh} \) is conservative relative to the measured monotonic strain hardening slope at strains of approximately 0.07 and above. Data from specimen LD-2-LER-R are not included in Table 6-7 due to the impact of the elongation-dependent axial restraint, which resulted in different levels of axial load in the repaired and unrepaired specimens, making changes in longitudinal reinforcement strength not directly comparable to changes in beam strength. Data from other test programs are not included as residual elongation data were not available.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( \varepsilon_{\text{resid}} ) (mm)</th>
<th>( L_p ) (mm)</th>
<th>( \varepsilon_{\text{avg}} )</th>
<th>( \left[ \frac{f_u}{f_y} - 1 \right] / \varepsilon )</th>
<th>Increase in steel strength (%( f_y ))</th>
<th>Experimental beam strength increase (%)(^1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LD-1-R</td>
<td>6.1</td>
<td>338</td>
<td>0.018</td>
<td>1.00</td>
<td>1.8</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.33</td>
<td>6.0</td>
<td></td>
</tr>
<tr>
<td>LD-2-R</td>
<td>11.6</td>
<td>338</td>
<td>0.034</td>
<td>1.00</td>
<td>3.4</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.33</td>
<td>11.4</td>
<td></td>
</tr>
</tbody>
</table>

\(^1\)Based on comparison against the average of all non-repaired specimens (see Table 6-4)

Limitations of this calculation procedure include (i) the lack of consideration of the effects of cyclic loading on the rate of strain hardening, and (ii) the assumption that the equivalent plastic hinge length remains unchanged regardless of repair. It is nonetheless proposed as a simple method of estimating strength increases of a repaired plastic hinge due to residual elongation at the time of repair. Higher levels of strain hardening being a source of increased strength in repaired plastic hinges is somewhat corroborated by the lack of a strength increase in the Lehman et al. bridge column [154], which was subjected to a large enough axial compression force to have likely prevented any significant residual elongation at the time of repair. However, additional experimental research is recommended.
It is unknown what degree of strain ageing occurred in any of the repaired specimens here discussed, and therefore direct analysis regarding the effects of strain ageing on the post-repair strength is not possible. In the three repaired beam specimens of the experimental program, post-repair testing took place no later than 14 days after the initial damaging loading, and therefore strain ageing is not expected to have had a significant effect. As epoxy repair itself does not necessarily have any effect on the degree of strain ageing in the longitudinal reinforcement, the recommendations for considering strain ageing when assessing the overstrength of damaged hinges (see Section 5.4) are therefore considered to also be applicable for repaired hinges (use of a maximum 1.1 factor on the longitudinal steel strength in addition to standard overstrength factors).

If the increase in moment capacity of a repaired plastic hinge is large enough, a new plastic hinge can instead form in the adjacent area [as was shown in Figure 6-15(a)]. All specimens with repaired strength ratios higher than 1.07 belonged to one of two experimental programs (Celebi and Penzien [150] or Lee et al. [152]), and some degree of relocation of the plastic hinge zones away from the repaired area was reported in both of these test programs. A schematic shown in Figure 6-19 illustrates the theoretical relative increase in post-repair moment capacity required to completely relocate the plastic hinge. The effect of hinge relocation on the maximum shear force that can be carried by a component is also shown in Figure 6-19. Note that the schematic of Figure 6-19 is idealized, as the repaired moment capacity will fluctuate along the length of the repaired region, rather than be constant throughout.
The effects of epoxy injection repair of moderate plastic hinging damage

Figure 6-19: Theoretical increase in moment capacity required to completely relocate a plastic hinge and theoretical maximum shear force demand on elements with epoxy-repaired plastic hinges

It is recommended that the overstrength moment capacity of a repaired plastic hinge be calculated using a longitudinal reinforcement strength determined following the principles laid out in this section. If no residual elongation is present, strain ageing can be considered the only potential source of increased strength post-repair. If residual elongation is present, the potential increase in strength post-repair due to additional strain hardening can be calculated as described in Table 6-7. Note that both of these factors are in addition to normal overstrength factors, which are intended to account for variation in material properties and the degree of strain hardening that occurs in typical, non-repaired plastic hinges [29]. Once the overstrength moment capacity of a repaired plastic hinge has been determined, the procedure shown in Figure 6-19 can be followed to determine whether plastic hinge relocation is likely and how to calculate any associated reduction in the maximum shear force carried by the member. However, for the purpose of determining the shear force due to overstrength actions, the higher shear force, corresponding to the repaired moment capacity, should be taken.
6.3.5 Deformation capacity

Seven test specimens were tested in a manner that allowed conclusions to be drawn with regards to deformation capacity. The three beam specimens discussed in Section 6.2 and the Lehman et al. bridge column [154] had deformation capacities at least as high as those that occurred in equivalent non-repaired specimens. However, three of the four beam specimens tested by Celebi and Penzien failed due to longitudinal reinforcement fracture at a lower deformation demand than was applied prior to repair. Potential causes of reductions in deformation capacity of repaired plastic hinges, relative to undamaged hinges, are here discussed.

If plastic hinge relocation does take place [Figure 6-15(a)], the curvatures induced in the relocated hinge would be higher than those that would have been induced in the original hinge, for a given drift demand. In order to account for changes in curvature demand due to plastic hinge relocation, the geometrical relationships between drift and local curvature should be modified to account for the expected new location of the plastic hinge. The most conservative estimate of a completely relocated plastic hinge should be assumed, even where only partial relocation [i.e. a lengthening of the plastic hinge zone as shown in Figure 6-15(b)] is expected.

It is assumed that the use of standard seismic assessment calculations, unmodified for damage or repair, are appropriate for determining the curvature capacity of relocated plastic hinges. This is because the majority of the inelastic demand on the longitudinal reinforcement occurs on steel that has not previously been subjected to significant strains. However, special attention should be given to the transverse reinforcement detailing of the member, as relocated plastic hinges could cause yielding to spread outside of the region that is specially detailed for ductile behaviour.

If plastic hinge relocation does not take place [Figure 6-15(c)], residual elongation can cause an increase in cumulative elongation within the hinge and an associated increase in longitudinal reinforcement strain [as calculated by Equation (6-3)]. Reinforcement within the repaired hinge would also be subject to any reduction in strain capacity or fatigue life due to strain ageing and low-cycle fatigue phenomena (as discussed in Chapter 5). Epoxy injection may mitigate these issues to some degree, as epoxy is generally effective at keeping injected cracks closed, and cracks are the area where the highest longitudinal reinforcement strains are likely to have occurred. However, there is no way of quantifying this effect, and it is therefore recommended that it be neglected.

For calculating curvature capacities based on the New Zealand seismic assessment guideline [29], the potential increase in longitudinal reinforcement strain due to residual elongation at the time of repair \( \varepsilon_{avg} \) should not be directly subtracted from the 0.06 strain limit \( \varepsilon_s, max \). The 0.06 limit is based on reinforcement with a probable uniform strain equal to or greater than 0.10, as shown in Equation (6-4). (The strain limit is lower than the probable uniform strain in order to indirectly account for effects such as low-cycle fatigue and accumulated elongation, which are not directly
The effects of epoxy injection repair of moderate plastic hinging damage

considered in this calculation procedure.) The value of $\varepsilon_{avg}$ should be subtracted from the probable uniform strain for the appropriate grade of reinforcement, as shown in Equation (6-5).

$$
\varepsilon_{s, \text{max}} = 0.6\varepsilon_u \leq 0.06 
$$

(6-4)

$$
\varepsilon_{s, \text{max(\text{post-\text{repair}})}} = 0.6(\varepsilon_u - \varepsilon_{avg}) \leq 0.06 
$$

(6-5)

where $\varepsilon_{s, \text{max}}$ is the limiting reinforcing strain for calculating curvature capacities as per the New Zealand seismic assessment guidelines, $\varepsilon_u$ is the probable uniform strain of the longitudinal reinforcement, $\varepsilon_{avg}$ is the average strain across the equivalent plastic hinge length at the time of repair [as defined in Equation (6-3)], and $\varepsilon_{s, \text{max(\text{post-\text{repair}})}}$ is the recommended limiting reinforcing strain for calculating curvature capacities of epoxy-repaired plastic hinges.

Grade 300E reinforcement has a prescribed minimum uniform strain of 0.15 [37], which is also taken as the probable uniform strain in the New Zealand seismic assessment guidelines. A very high value of $\varepsilon_{avg}$ (>0.05) would therefore be required for residual elongation at the time of repair to affect the 0.06 strain limit. This is consistent with the finding that residual elongation at the time of repair did not affect the deformation capacity of the beam specimens analysed in in Section 6.2 (the beam specimens had grade 300E longitudinal reinforcement).

Grade 500E reinforcement has a prescribed minimum uniform strain of 0.1 [37], and many older reinforcement grades also have probable uniform strains in that range [29]. For such grades of longitudinal reinforcement, even small values of $\varepsilon_{avg}$ would affect the limiting strain $\varepsilon_{s, \text{max(\text{post-\text{repair}})}}$ for calculating the curvature capacity of an epoxy-repaired plastic hinge using Equation (6-5). Experiments on epoxy-repaired plastic hinges with grade 500E longitudinal reinforcement or older steel grades are recommended to validate the appropriateness of this proposed calculation method.

6.3.6 Summary of recommended steps for determining the residual capacity of epoxy-repaired plastic hinges

Figure 6-20 illustrates the recommendations given in this section for determining an idealized elastic-perfectly plastic moment-curvature response of a plastic hinge that has been repaired by epoxy injection. The complete set of recommendations for defining the performance of a repaired plastic hinge is summarized as follows:

- Assume the post-repair initial secant stiffness to yield to be 80% of what would be calculated for an identical undamaged component (Figure 6-16). Note that this may not be applicable for plastic hinges in columns.
The effects of epoxy injection repair of moderate plastic hinging damage

- Assume the post-repair design (i.e. nominal) moment capacity to be 100% of what would be calculated for an identical undamaged component (Figure 6-17). Calculate the post-repair overstrength moment capacity using a longitudinal steel strength that accounts for (i) standard overstrength factors (e.g. the 1.35 factor recommended in NZS 3101:2006), (ii) strain ageing overstrength, if applicable (as described in Chapter 5), and (iii) additional strain hardening due to residual elongation at the time of repair (as described in Section 6.3.4).

- Calculate the curvature capacity (e.g. based on the New Zealand seismic assessment guidelines [29]) for (i) a relocated plastic hinge (which can be taken as identical to that of an equivalent undamaged component in cases where the ductile detailing length is sufficient), and (ii) a non-relocated repaired plastic hinge (using a longitudinal reinforcement strain capacity that accounts for the residual elongation at the time of repair, as given by Equation (6-5), in addition to the strain ageing and low cycle fatigue considerations described in Chapter 5). These curvature capacities should be compared against curvature demands calculated based on the corresponding plastic hinge location.

- Assume the post-repair maximum elongation is 100% of what would be calculated for an identical undamaged component and cumulative to any residual elongation at the time of repair [Equation (6-2)].

![Diagram](image_url)

**Figure 6-20:** Recommended method for describing the theoretical elastic-perfectly plastic moment-curvature response of an epoxy-repaired plastic hinge, relative to an identical undamaged component
6.4 Conclusions and Recommendations

This chapter focussed on quantifying the effects of epoxy injection repair of moderate plastic hinging damage in reinforced concrete beams and columns, using data available in the literature as well as results from the three epoxy-repaired beams tested by the author. The key conclusions and recommendations are summarized as follows:

- The beam specimens of the experimental program described in Chapter 2 that were repaired by epoxy injection following an initial damaging loading were found to have:
  1. increased strengths (<10% higher),
  2. reduced secant stiffness to yield values (~10-20% lower),
  3. increased cumulative elongations,
  4. longer lengths along which inelastic deformation was spread, and
  5. comparable energy dissipation and deformation capacities, as compared with nominally identical test specimens that were not repaired at any point.

- A database was compiled, consisting of fourteen experiments that involved epoxy injection repair to moderately damaged ductile reinforced concrete components. Data from these tests were used as the basis for a proposed method of quantifying the residual capacity of a plastic hinge that has been repaired by epoxy injection (summarized in Section 6.3.6).

- The stiffness and strength of the repaired specimens were not correlated with the displacement ductility they were subjected to prior to repair.

- Within the range of damage states considered (up to and including cracking, longitudinal reinforcement yielding, and spalling of cover concrete), visual damage indicators prior to repair were not found to be correlated with post-repair behaviour.

- Only a single experimental test was identified that involved application of epoxy injection repair to a ductile reinforced concrete column with axial compression. This test exhibited less restoration of stiffness than comparable tests on ductile reinforced concrete beams. Additional testing of epoxy-repaired plastic hinges subjected to axial compression is recommended.

- The residual deformations immediately prior to repair should be reported in all experimental programs involving epoxy injection of damaged plastic hinges. Past test programs typically have not reported these data, and therefore all analysis regarding the effects of the residual deformations have come from the epoxy-repaired beams of the test program described in Chapter 2. Testing of beams with grade 500E or older longitudinal reinforcement is particularly recommended, to better understand any potential implications of increased strain due to residual elongation at the time of repair.
Due to a lack of data, no consideration was given to any variance in performance as a result of the repair quality, the extent to which the epoxy penetrated the crack system, or the properties of the repair materials. Testing is recommended to evaluate these issues.
Chapter 7

Conclusions and Recommendations

The motivation for the research presented in this thesis was the widespread demolition of reinforced concrete buildings that took place in the aftermath of the Canterbury and Kaikoura earthquakes in New Zealand. Many of these buildings were demolished despite being relatively modern, having exhibited only modest levels of damage, and having largely behaved as intended by the designers. While many topics related to post-earthquake assessment of reinforced concrete buildings require research, the specific focus of this thesis was improved understanding on the post-earthquake residual capacity and reparability of moderately damaged plastic hinges in reinforced concrete moment frames. Indirectly related topics, including the effects of the number of loading cycles, the loading rate, and the level of restraint to axial elongation, were also investigated. An experimental program on seventeen nominally identical reinforced concrete beams, described in Chapter 2, constituted a major part of this work. The repair method focussed on in this thesis was epoxy injection of cracks and patching of spalled concrete. Special attention was paid to the effects of prior yielding in the longitudinal reinforcement, as it has implications for both the pre- and post-repair response. The following sections present a concise summary of the main conclusions arising from this research, as well as a discussion of the limitations and recommended future work.

7.1 Conclusions – general plastic hinge behaviour

7.1.1 Effect of moderate-level loading cycles

Past experimental studies have shown that repeated cycling at large drift demands can have a major detrimental impact on the performance of reinforced concrete plastic hinges. This research focussed on isolating the effects of moderate-level loading cycles, which was taken as those cycles approximately at or below 2% drift. The available experimental data show that the number of moderate-level loading cycles has a limited effect on the cyclic strength degradation or deformation capacity of ductile plastic hinges in beams and columns. This conclusion supports the current practice of neglecting the number of loading cycles in design or assessment, where drift limits in the range of 2.5% are typically imposed. It also indicates that commonly used cumulative damage models can overweight the importance of moderate-level loading cycles.
7.1.2 Effect of loading rate

It has previously been shown that increased loading rates can cause increased flexural strength in ductile reinforced concrete components. This research used available experimental data to show that this increase in flexural strength tends to be largest at first yield. It may be appropriate to neglect this strength increase for the purposes of determining strength hierarchies in reinforced concrete moment frames, as other sources of beam overstrength tend to reach a maximum at higher deformation demands.

Concerns have been raised about whether seismic loading rates alter the damage pattern or deformation capacity of reinforced concrete components, relative to what occurs in typical static laboratory tests. This research found that ductile reinforced concrete beams and columns subjected to high loading rates have exhibited an inconsistent tendency to have lower deformation capacities relative to static equivalents. However, this effect is relatively minor, highly variable, and does not preclude a high level of ductility in well-detailed plastic hinges. The available experimental data are too varied and inconsistent to make conclusions about the effects of loading rate on the damage pattern of reinforced concrete plastic hinges.

7.1.3 Effect of axial elongation and restraint

Equations for estimating the magnitude of elongation in reinforced concrete plastic hinges have been incorporated into Amendment 3 of NZS 3101:2006. Comparison of these equations with data from the beam specimens of the experimental program showed that they gave a conservative prediction of elongation in all cases. In order to accurately predict the magnitude of beam elongation for an arbitrary load history, consideration had to be given to the peak positive and negative rotation and the number of loading cycles.

Significant displacement compatibility issues due to axial elongation of beam plastic hinges in reinforced concrete moment frames were observed following recent earthquakes in New Zealand. Previous frame tests have shown that the surrounding structure may or may not provide significant restraint to beam elongation, depending on factors including the location of the plastic hinge, the flooring system, and the connection details. An elastic axial restraint system was applied to five beam specimens of the experimental program to investigate the impacts of elongation-induced axial compression. The magnitude of elongation was found to be highly sensitive to the induced compression forces, and a relatively small axial load \((0.025A_gf'_c)\) was sufficient to prevent further accumulation of residual elongation. The restrained beam specimens also had higher flexural strengths, reduced shear deformations, more flexure-dominant damage patterns, and reduced variability in deformation capacity, relative to the unrestrained beam specimens.
7.2 Conclusions – residual capacity and reparability of plastic hinges

7.2.1 Implications of visual damage

Residual crack widths have typically been used as key damage metrics in post-earthquake situations. Data from the beam specimens of the experimental program were used to show that residual crack widths are dependent on the residual deformations, which are in turn dependent on the load history, axial load, and level of restraint to axial elongation. Residual crack widths are therefore not necessarily indicative of the peak rotation demands imposed on a plastic hinge, although such a relationship would be obtained if using data from standard cyclic experiments.

The number of non-hairline cracks that formed in the beam specimens was found to increase with the drift demand, albeit with significant scatter. Despite the scatter, qualitative conclusions were possible. In plastic hinge zones not subjected to axial compression (i.e. in beams), the presence of only hairline cracks can indicate that yielding has not occurred, a single non-hairline residual crack can indicate that some yielding has occurred, and well-distributed non-hairline residual cracks can indicate that more significant plastic deformation demands have occurred. Post-earthquake observations of limited numbers of wide cracks do not necessarily indicate a short plastic hinge length, but rather may indicate an insufficient deformation demand to cause additional cracks to open.

In plastic hinge zones subjected to axial compression (i.e. in columns), gravity loads can force cracks to close, and a column that has previously yielded can exhibit only hairline cracks. Cover concrete spalling in columns is an alternate visual damage metric that can indicate yielding has previously occurred.

Moderate visual damage, as defined in Section 1.4, was found to typically have no effect on the residual strength or deformation capacity of ductile reinforced concrete plastic hinges. However, two cases in which moderate visual damage may indicate a concern with regards to the residual deformation capacity were identified. If a transverse crack through the depth of a beam is observed to have a width sufficient to limit shear transfer through aggregate interlock, that beam may exhibit sliding shear deformations across that crack upon reloading. Based on limited data from the beams of the experimental program, it was conservatively recommended that the deformation capacity of beams exhibiting such cracks be taken as 80% of what would be calculated for an equivalent undamaged component. If cover concrete delamination or spalling that completely exposes the longitudinal reinforcement is observed, it may not be possible to confidently assess whether buckling has previously occurred in the longitudinal reinforcement. A lack of buckling is a key distinction in the recommendations made with regards to the effects of prior yielding in longitudinal
reinforcement, and those recommendations may therefore be inapplicable in cases of severe cover concrete spalling.

7.2.2 Post-earthquake residual stiffness

Considerable degradation in stiffness can occur in reinforced concrete components as a result of prior inelastic loading, but limitations were identified with the use of visual damage metrics for the purpose of assessing residual stiffness. A dataset consisting of six of the experimental program beam specimens and eight columns subjected to shake table loading was used to derive an approximate method for estimating residual stiffness as a function of the deformation demand. The specimens in the dataset were all subjected to earthquake-type loadings followed by at least one subsequent loading, which allowed the residual stiffness following the previous earthquake-type loading to be measured. It was found that the reduction in stiffness, relative to the initial secant stiffness to yield, could be conservatively estimated as the inverse of the maximum displacement ductility demand previously incurred. If using this model in post-earthquake assessments, it is recommended that any component believed to have yielded should be conservatively assumed to have a residual stiffness no greater than 50% of its initial secant stiffness to yield. Visual damage indicators can help determine if yielding has occurred, as previously discussed in Section 7.1.1.

7.2.3 Effects of yielded longitudinal reinforcement

The residual strain capacity and fatigue life of longitudinal reinforcement that has previously yielded was of considerable concern following recent earthquakes in New Zealand. Data from past experimental studies on the low-cycle fatigue behaviour of deformed reinforcement, including one study on New Zealand grade 300E reinforcement conducted by the author, were collated. It was found that reinforcement coupons tested using an unsupported length of four times the bar diameter, which was sufficient to prevent buckling, had higher fatigue resistance than coupons tested using an unsupported length of six times the bar diameter, which allowed some buckling to occur. Coefficients for the Coffin-Manson equation [Equation (5-2)] were proposed based on test data with an unsupported length of four times the bar diameter or less. These coefficients are recommended for the purpose of assessing the consumed fatigue life of longitudinal reinforcement in post-earthquake situations where there is no visual evidence of prior buckling.

Past test programs have shown that strain ageing of previously yielded steel can result in increased strengths and reduced strain capacities. The susceptibility of steel to the effects of strain ageing is dependent on its chemical composition; New Zealand grade 500E reinforcement is not susceptible. Data from three test programs investigating the effects of strain ageing on New Zealand grade 300E longitudinal reinforcement were analysed. Strength increases at the reloading strain were found to be as high as 15%, but limited increases in the ultimate tensile strength were observed. It was
recommended that an additional 1.1 overstrength factor be used when assessing the residual capacity of plastic hinges with strain aged longitudinal reinforcement. A lower-bound model was proposed to estimate the reduction in uniform strain due to strain ageing, as a function of the maximum ‘pre-strain’ induced prior to ageing.

Simple models were proposed for estimating the strain demands in the longitudinal reinforcement of reinforced concrete plastic hinges as a function of the plastic rotation demands. These models were based on equivalent plastic hinge length assumptions, which were assumed to be applicable once strains of 0.03 or larger are induced in the longitudinal reinforcement. The models were combined with the previously developed low-cycle fatigue and strain ageing models in order to estimate the impacts of low-cycle fatigue or strain ageing for a given prior plastic rotation demand. It was found that, for plastic hinges that were previously subjected to plastic rotations of 0.015 radians or below, low-cycle fatigue failure of unbuckled reinforcement or tensile fracture of strain aged reinforcement are unlikely to become governing failure modes in future earthquakes.

Only a single test program has investigated the effects of strain ageing on the low-cycle fatigue resistance of reinforcing steel, to the author’s knowledge. Reductions in the residual fatigue life of New Zealand grade 300E reinforcement on the order of 50% occurred as a result of strain ageing, based on tests using an unsupported length of six times the bar diameter. Tests using an unsupported length of four times the bar diameter are recommended to help quantify the effects of strain ageing on the residual fatigue life in post-earthquake situations where there is no visual evidence of prior buckling. Low-cycle fatigue failure of unbuckled strain aged reinforcement may be of concern and requires further research.

7.2.4 Effects of repair by epoxy injection

Epoxy injection is an established concrete repair method that is applicable to moderate plastic hinging damage, but its effects are not fully understood. A database of fourteen epoxy-repaired test specimens, including three of the beam specimens tested by the author, was used in an effort to quantify the effects of epoxy injection. It was found that the post-repair stiffness was not correlated with the displacement ductility the test specimens were subjected to prior to repair. Epoxy injection was effective in restoring the stiffness of beams to a minimum of 80% of their original secant stiffness to yield, but considerably less stiffness restoration was attained in the single epoxy-repaired column specimen that was identified in the literature.

All epoxy-repaired specimens exhibited strengths at least as high as their pre-repair strengths, and some specimens exhibited strength increases as high as 20%. It was hypothesised that these strength increases were due to strain ageing effects or higher levels of strain hardening in the longitudinal reinforcement. Higher levels of strain hardening in epoxy-repaired beams are likely to occur in any
beam that has residual elongation at the time of repair. Simple methods of accounting for increased longitudinal reinforcement strain due to residual elongation at the time of repair were proposed.

Post-repair strength increases can result in a partial or complete relocation of the plastic hinge zone. Beam plastic hinges that relocate away from the column face can be subjected to increased curvature demands for a given drift demand, but the inelastic demand occurs on reinforcement that has not previously been subjected to large strains. Plastic hinge relocation may therefore be beneficial from a capacity viewpoint, but detrimental from a demand viewpoint. Plastic hinge relocation is not readily predictable, based on the available test data, and it is therefore recommended that cases corresponding to both relocated and un-relocated plastic hinges be checked. In the case of un-relocated plastic hinge zones, consideration should be given to the previously discussed low-cycle fatigue and strain ageing phenomena, as well as the residual strain at the time of repair. However, these effects do not necessarily result in a reduction of deformation capacity in ductile plastic hinge zones. The repaired beam specimens tested by the author all exhibited deformation capacities comparable to undamaged specimens.

7.3 Research limitations and recommended future work

In conducting this research, effort was made to be as comprehensive as possible. Despite this, there are a number of limitations to the analysis, and several areas require further investigation.

i. Much of the analysis was based on data from the nominally identical beam specimens of the experimental program described in Chapter 2. Variations in detailing or geometry were not accounted for in this test program. The following limitations are particularly noteworthy:

- The beam specimens had a relatively low longitudinal reinforcement ratio (0.6%), and a relatively high shear span to depth ratio (3.6), resulting in low maximum shear stresses (approximately 0.5MPa). Further targeted research on ductile reinforced concrete beams with higher shear demands is recommended.

- The shear sliding behaviour of the beam specimens strongly influenced the performance at moderate-to-high deformation demands. This deformation mechanism may not be representative of beam plastic hinges in general.

- The analysis regarding residual crack width metrics was not supported by data from other test programs, due to a lack of availability. Flexural crack development can be affected by factors that were not varied in this experimental program, such as the shear span length, the strain hardening properties of the longitudinal reinforcement, and the section depth.
ii. A number of assumptions were made with regards to the definition of ‘moderate’ plastic hinging damage and its effects on plastic hinge residual capacity and reparability by epoxy injection:

- Wide diagonal cracks crossing transverse reinforcement were considered to be indicative of a ‘severe’ damage state, and therefore outside the scope of this work. However, members exhibiting such cracks may have substantial residual capacity or be able to achieve good performance after repair by epoxy injection. Further research into the residual capacity and reparability of plastic hinges with yielded transverse reinforcement is recommended.

- It was assumed that residual capacity or reparability is unaffected by any changes in the core concrete material-level stress-strain behaviour as a result of prior loading, provided that it remains visually intact. Further research is required to validate this assumption.

- The demands placed on transverse reinforcement as a result of confinement-related stresses were not considered in this study. It is possible that a plastic hinge meeting the criteria of ‘moderate’ visual damage could have previously been subjected to inelastic demands in the transverse reinforcement as a result of confinement stresses. Further research is recommended to determine if it is justified to neglect this effect when assessing the residual capacity and reparability of ductile plastic hinge zones in columns.

- It was assumed that plastic hinges exhibiting ‘moderate’ visual damage have sufficient anchorage of longitudinal reinforcement. However, visual damage indicators are not necessarily indicative of the degree of bond degradation that has occurred in a beam-column joint. The work in this thesis may not be applicable to buildings with joint depths insufficient to ensure adequate bond transfer under repeated inelastic loading. Further research is recommended.

iii. Other general limitations and recommendations include:

- Component-level experimental tests on ductile plastic hinges with strain-aged longitudinal reinforcement are recommended. The work in this thesis relied on simple analytical extensions of material-level data.

- Only a single experimental test was identified that involved application of epoxy injection repair to a ductile reinforced concrete column subjected to axial compression. This test exhibited less restoration of stiffness than tests on epoxy-repaired ductile reinforced concrete beams. Additional testing of epoxy-repaired plastic hinges subjected to axial compression is recommended.
Conclusions and Recommendations

- Past test programs on epoxy injection repair have typically not reported the residual deformations immediately prior to repair. All analysis regarding the effects of residual deformations prior to repair therefore relied on data from the epoxy-repaired beams of this experimental program. It is recommended that residual deformations be reported in all future test programs involving repair of plastic hinging damage.

- The work in this thesis was focussed on plastic hinges that form in reinforced concrete moment frames, and the conclusions are therefore not applicable to reinforced concrete wall buildings. Some recommendations on the residual capacity and reparability of earthquake-damaged walls are given in FEMA 306 [16], but further targeted research is recommended.

- The primary source of data in this thesis was reinforced concrete beams not subjected to axial compression. Although effort was made to extend the analysis to be applicable to plastic hinges that form in columns, targeted research programs on ductile reinforced concrete columns are also recommended.
Appendix A

Experimental program data processing

This section discusses the methodology used in processing the raw data collected during the experimental test program. A schematic of the instrumentation layout is repeated here for convenience in Figure A-1.

A.1 Introduction

Data were recorded using the University of Auckland Structures Testing Laboratory in-house data acquisition software, with a sampling rate of 100Hz for all dynamic tests and 5Hz for all static tests. The unprocessed data consisted of the raw or amplified voltage readings from the sensors, multiplied by an appropriate calibration factor to obtain a desired unit of measurement. Each sensor was calibrated immediately prior to the beginning of the test program.

The channels recorded during testing included all sensors shown in Figure A-1, as well as a time stamp and readings from the actuator’s internal LVDT and load cells (labeled ‘DISP’ and ‘LOAD’). DISP and LOAD both gave positive readings when the specimen was pushed in the south direction. For the specimens with axial restraint, data from the two additional load cells (‘LCE’ and ‘LCW’) were also recorded, which gave positive readings when undergoing compression.

All displacement sensors were positive when elongating, except for SP-2.580, SP-1.188, and SP-0.588, which were set as negative when elongating, so as to correspond to the positive direction of the actuator LVDT.

Due to the vertical orientation of the beam during testing, any reference herein to vertical displacements refers to displacements along the longitudinal axis of the beam, and any reference to lateral displacements refers to displacements perpendicular to the longitudinal axis in the plane of the beam.

A.2 Initial processing

A.2.1 Filtering

All channels of the raw data were filtered to reduce noise using a zero phase shift low-pass Butterworth filter. Selection of the cut-off frequencies was based on a trial-and-error approach, resulting in values of 7.5Hz for dynamic tests and 0.375Hz for static tests. These were the lowest
possible cut-off frequencies that eliminated the majority of electrical noise, but did not over-filter the data.

Figure A-1: Schematic and naming convention of instrumentation for the three different instrumentation layouts used

The majority of the data presented in this document are in their filtered state. However, in some cases, the unfiltered data were used to avoid any artificial change in readings due to the filtering process. One example of this was identification of the yield strength, which often occurred at the peak of a spike in the load response. The filtering process smoothed these spikes, which artificially
Experimental program data processing

reduced the yield strengths by up to 2%. The unfiltered data were therefore used to determine the exact yield strengths of all specimens.

A.2.2 Foundation movement gauges

In all specimens, the recorded displacements for the gauges measuring foundation movement (FU-N, FU-S, and FS) were typically less than ±0.1mm. The corresponding maximum lateral displacements at the actuator position are ±0.4mm due to foundation rocking (calculated using Equation (A-1)), and ±0.1mm due to foundation sliding. It was decided that these displacements represent negligible movement of the foundation, and the data from these gauges were therefore ignored in all future processing (i.e. all deformations were assumed to take place in the beam).

\[ \Delta_{\text{foundation rocking}} = \frac{(FU-N - FU-S)}{d_{\text{between sensors}}} \times H \]  

(A-1)

where \( H \) is the length from foundation base to actuator centreline (3.105m), and the distance between sensors is 1.550m.

In specimens P-2, LD-2, & LD-2-S, the readings from the foundation movement gauges increased to more significant values (±0.5mm) late in the test. However, this was deemed to be due to movement of the reference frame caused by the out-of-plane twisting action of the specimens, and these data were therefore ignored.

A.2.3 Longitudinal displacement gauges

The displacements recorded by the longitudinal gauges positioned along the first 1.188m of the beams were converted into average strains by dividing by the gauge length. In specimens with instrumentation layouts A and B the gauge length for the bottom sensors (N1 and S1) was 108mm, and the gauge length for all others (N2-N10 & S2-S10) was 120mm. In specimens with instrumentation layout C the gauge length for N1 & S2 was 25mm, the gauge length for N2 & S2 was 263mm, and the gauge length for all others (N3-N5 & S3-S5) was 300mm. In all cases, the displacements measured in bottom sensors N1 and S1 also included any deformation due to strain penetration in the foundation.

Any reference herein to longitudinal displacement gauges \( N_n \) or \( S_n \) refers to these calculated strains, rather than the raw measurements.

A.2.4 Vertical string potentiometers

String potentiometers SP-NV, SP-SV, SP-NVB, and SP-SVB were intended to measure purely vertical displacements. However, they were mounted with the wire running at a slight diagonal in
order to avoid spalling concrete from affecting the results or damaging the instruments. It was therefore required to use the lateral displacements of SP-0.588 and SP-1.188 to correct the raw measurements. The method of calculating vertical displacements $\Delta Y$ is given by Equation (A-2) with variables as shown in Figure A-2.

$$\Delta Y = \sqrt{(L_i + \Delta L)^2 - (X_i - \Delta X)^2} - Y_i$$

(A-2)

where $X_i, Y_i, L_i$ are constants measured prior to testing, $\Delta L$ is the raw measurement from the string potentiometer being corrected, and $\Delta X$ is measured by either SP-0.588 or SP-1.188, depending on the string potentiometer being corrected.

The vertical displacements $\Delta Y$ of each of the diagonal string potentiometers was further processed to obtain the vertical displacement at the location of the longitudinal displacement gauges. This was done to facilitate comparisons between the string potentiometer and displacement gauge measurements. The method used to correct the vertical displacements was based on similar triangles and the assumption of plane sections, as shown in Figure A-3.

The corrected values of string potentiometers SP-NV, SP-SV, SP-NVB, or SP-SVB refer to these vertical displacements corresponding to the location of the longitudinal steel, rather than the raw measurements. Any reference herein refers to these corrected values. The values of $X_i$ and $d_{measurement}$ for each specimen are given in Table A-1. The corrected values of string potentiometers SP-NV, SP-SV, SP-NVB, or SP-SVB refer to these vertical displacements corresponding to the location of the longitudinal steel, rather than the raw measurements. Any reference herein refers to these corrected values.
A.3 Calculation of lateral displacement components

A.3.1 Total displacements

Total lateral displacements along the length of the beam were determined in two ways. The first method was direct measurement using SP-0.588, SP-1.188, SP-2.580 and the actuator LVDT. The second method was by calculation from the diagonal displacement gauges. Total lateral displacement at the point 0.588m from the foundation top was calculated from gauges BSN and BNS using Equation (A-3) with variables as shown in Figure A-4. This equation was also used to
calculate the net lateral displacement due to deformations between 0.588m and 1.188m from the
foundation top, using data from gauges TSN and TNS.

\[
\Delta X = \frac{\sqrt{(L_i + \Delta L_{\text{north}})^2 - (Y_i + \Delta Y_{\text{north}})^2} - \sqrt{(L_i + \Delta L_{\text{south}})^2 - (Y_i + \Delta Y_{\text{south}})^2}}{2} \tag{A-3}
\]

where \(Y_i, L_i\) are constants measured prior to testing, \(\Delta L\) is measured by the diagonal displacement
gauges, and \(\Delta Y\) is measured by the vertical string potentiometers.

\[\Delta_0.588\text{--interface} = \frac{(S1 - N1)}{d_{\text{between sensors}}} \left( GL \left( \frac{588mm - \frac{GL}{2}}{2} \right) - BS \right) \tag{A-4}\]

where \(GL\) is the gauge length of the interface sensors in instrumentation layout C (25mm), S1,N1, & BS are displacement
gauge readings, and the distance between sensors is 0.630m (the distance between centroids of the longitudinal reinforcement).

**Figure A-4: Variables used to calculate total lateral displacement from diagonal displacement gauges**

In the specimens with instrumentation layout C, the diagonal displacement gauges were connected
to the beam at a point 25mm from the foundation. The displacement due to interface deformations,
measured using gauges N1, S1, & BS, therefore had to be added to the displacements calculated
from the diagonal gauges in order to obtain the total lateral displacement. The displacement at a
height of 0.588m above the foundation due to the interface deformation was calculated using
Equation (A-4).

**A.3.2 Flexural displacements**

Lateral displacements due to flexure were calculated using two methods. The first method used the
longitudinal displacement gauges \(N_n\) and \(S_n\), with curvatures \(\phi_n\) calculated for each pair of gauges
using Equation (A-5). Flexural displacements along the length of the beam covered by the
displacement gauges were then calculated from the average curvatures using a discretized version
of standard Euler-Bernoulli beam theory, with the centre of rotation assumed to be at the midpoint of each gauge length. The flexural displacements at locations of 0.588m and 1.188m above the foundation top were of most interest, as they could be compared with displacements calculated using the second method.

\[ \varphi_n = \frac{(N_n - S_n)}{d_{\text{between sensors}}} \]  

(A-5)

where the distance between sensors is 0.630m.

The second method used the vertical string potentiometers to calculate flexural displacements. With this method it was necessary to assume a centre of rotation \( L_{CR} \), which was estimated to be 200mm above the foundation top. This estimate was based on analysis of the curvature distributions calculated using Equation (A-5). While the actual centre of rotation varied somewhat during the duration of the tests, and between specimens, 200mm was chosen as a value close to the mean. Flexural displacements at the location of the string potentiometers were then calculated using Equation (A-6) with variables as shown in Figure A-5.

\[ \Delta_{\text{flexural}} = \frac{(\Delta Y_{\text{north}} - \Delta Y_{\text{south}})}{d_{\text{between sensors}}} \times L(L - L_{CR}) = \theta(L - L_{CR}) \]  

(A-6)

where \( \Delta Y \) is measured from the vertical string potentiometers, \( L \) is the distance from foundation top to the string potentiometer location, and the distance between sensors is 0.630m.

Figure A-5: Variables used to calculate flexural displacements from vertical string potentiometers
A.3.3 Shear displacements

Displacement due to shear deformation was calculated by subtraction of the flexural displacements from the total displacements. Given the multiple methods of calculating both flexural and total displacements, there were several different ways shear deformations could be extracted. For specimens with instrumentation layouts B & C, the reported shear deformations come from subtracting the flexural displacements calculated from the vertical string potentiometers from the total displacements calculated from the diagonal displacement gauges. For specimens with instrumentation layout A, the shear deformations come from subtracting the flexural displacements calculated from the vertical string potentiometers from the total displacements measured by the lateral string potentiometers.

Shear sliding on the beam-foundation interface was directly measured using displacement gauge BS (instrumentation layout B & C only). This shear sliding displacement is included in the total shear displacement calculations previously described.

A.4 Other calculations

A.4.1 Beam elongation

Beam elongation was taken to be the average value of vertical string potentiometers SP-NV and SP-SV. These string potentiometers were connected to the specimen at a height of 1.188m above the foundation top, and therefore any elongation occurring above that height was assumed to be negligible and was not included in the measurements. This assumption was validated by comparing the vertical displacements measured by SP-NV and SP-NVB or SP-SV and SP-SVB. In all cases, minimal difference was observed between the two, implying that the vast majority of elongation occurred within the range of SP-NVB and SP-SVB (i.e. within 0.588m of the foundation top).

A.4.2 Shear force

For the specimens with axial restraint, it was required to remove the lateral component of the force in the restraining rods from the raw shear force readings in the actuator load cell. This was approximated by the geometrical calculation in Equation (A-7) assuming rigid body rotation of the beam as shown in Figure A-6.
Figure A-6: Variables used to correct actuator load cell reading to determine shear force transferred to the specimen

\[ V = LOAD - P \times \left( \Delta \times \frac{L_{\text{beam}}}{L_{\text{rod}}} \right) \]  

where \( L_{\text{beam}}, L_{\text{rod}} \) are constants, \( \Delta \) is approximated as the beam drift, \( LOAD \) is measured by the actuator load cell, and \( P \) is the total axial force, taken as the sum of the two restraint system load cells (LCW+LCE).

A.5 Summary of measured parameters

A summary of key parameters measured and/or calculated from the experimental test program is presented in Table A-2.
Experimental program data processing

Table A-2: Summary of measured parameters during experimental test program

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Height above foundation top (m)</th>
<th>Method of measurement</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement due to foundation rocking</td>
<td>2.580</td>
<td>Calculated from foundation rocking displacement gauges</td>
<td>Negligible and therefore taken as zero</td>
</tr>
<tr>
<td>Displacement due to foundation sliding</td>
<td>Constant along length of beam</td>
<td>Foundation sliding displacement gauge</td>
<td>Negligible and therefore taken as zero</td>
</tr>
<tr>
<td>Longitudinal strain distribution</td>
<td>0.000-1.188</td>
<td>Calculated from longitudinal displacement gauges</td>
<td></td>
</tr>
<tr>
<td>Curvature distribution</td>
<td>0.000-1.188</td>
<td>Calculated from longitudinal displacement gauges</td>
<td></td>
</tr>
<tr>
<td>Total lateral displacement</td>
<td>2.580</td>
<td>a) Actuator LVDT</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>b) String potentiometer</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>a) String potentiometer</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>b) Calculated from diagonal displacement gauges</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>c) Calculated from diagonal displacement gauges and interface gauges</td>
<td></td>
</tr>
<tr>
<td>Lateral displacement due to flexure</td>
<td>0.588¹ &amp; 1.188</td>
<td>Calculated from vertical string potentiometers</td>
<td></td>
</tr>
<tr>
<td>Lateral displacement due to shear</td>
<td>0.588¹ &amp; 1.188</td>
<td>Calculated from longitudinal displacement gauges</td>
<td></td>
</tr>
<tr>
<td>Beam-foundation interface sliding¹</td>
<td>0.000</td>
<td>Beam sliding displacement gauge</td>
<td></td>
</tr>
<tr>
<td>Beam elongation</td>
<td>0.588¹ &amp; 1.188</td>
<td>Calculated from vertical string potentiometers</td>
<td></td>
</tr>
<tr>
<td>Force applied by actuator</td>
<td>2.580</td>
<td>Actuator load cell</td>
<td></td>
</tr>
<tr>
<td>Force induced in axial restraint rods</td>
<td>Constant along length of beam</td>
<td>Axial restraint load cells</td>
<td></td>
</tr>
</tbody>
</table>

¹These data were not collected for specimens with instrumentation layout A
²Instrumentation layout C only

Corrected to remove horizontal component of axial restraint

For specimens with axial restraint only

Actuator LVDT data are used in all cases. String pot. used only for verification

Negligible elongation occurred above 0.588m
Appendix B

Test specimen damage progression

This appendix presents a description of the damage progression for each specimen, along with a selection of photos at critical points during each test. The complete set of photos and videos are available at DOI 10.17603/DS2SQ2K.

For all cyclic tests, photographs are provided at three drift levels, 1.36, 2.17, and 2.71%. In all cases the photographs correspond to the damage state after both cycles to the peak drift in question. For tests with an initial earthquake loading, photographs are provided at the completion of the earthquake portion of the test, which corresponds to either 1.36% (1 specimen) or 2.17% (2 specimens) peak drift. Photographs are also included from the cyclic portion of these tests at any of the previously listed drift levels that are still applicable. For all specimens, photographs at test completion (i.e. the residual state after the final loading cycle) are also included.

B.1 Specimen CYC

Specimen CYC exhibited progressively increasing numbers of flexural cracks until the loading cycles to 1.09% drift, after which point few additional flexural cracks appeared. The number of non-hairline flexural cracks on each end of the beam continued to grow, until reaching a maximum of six (south end) and five (north end) during the cycles to 1.63% drift. Longitudinal bond splitting cracks first appeared in the cycles to 1.36% drift. A non-hairline crack through the depth of the member at a location other than the beam-foundation interface also first occurred at 1.36% drift. From this point onward, the width of this sliding plane crack grew progressively, resulting in increased shear deformations. The sliding plane crack occurred adjacent to a stirrup located at a distance of approximately 300mm from the beam-foundation interface. Concurrently with the growth of the sliding plane crack, the bond splitting cracks began to widen and intersect with large flexural cracks, leading to delamination of cover concrete during the cycles to 2.44% drift. Buckling of the longitudinal reinforcement quickly followed delamination of the cover concrete. Significant degradation of the core concrete near the beam ends occurred late in the test. Longitudinal bar rupture occurred on bars that had previously been subjected to a number of severe buckling cycles during the first cycle to 4.88% drift.
B.2 Specimen CYC-DYN

In specimen CYC-DYN, the point at which additional flexural cracks stopped appearing could not accurately be identified due to the high-speed nature of dynamic testing. Longitudinal bond splitting cracks first appeared in the cycles to 1.09% drift. A non-hairline crack through the depth of the member first occurred at 0.81% drift, at a stirrup located approximately 175mm from the beam-foundation interface. Damage became heavily concentrated around this sliding plane crack in subsequent cycles. Delamination of cover concrete first occurred during the cycles to 1.90% drift. Bar buckling and degradation of the core concrete near the beam ends quickly followed and became progressively worse in subsequent cycles. This damage pattern continued until the test was stopped after cycles to 3.80% drift due to significant strength loss. Longitudinal bar rupture did not occur.

B.3 Specimen P-1

Flexural cracking was the only damage to Specimen P-1 during dynamic earthquake loading. Additional flexural cracks appeared during the first static cycles (to 1.63% drift) only. Bond splitting cracks also first appeared during the cycles to 1.63% drift. The number of non-hairline flexural cracks continued to grow until reaching a maximum of eight (south end) and six (north end) during the cycles to 2.17% drift. A non-hairline crack through the depth of the member first occurred at 1.90% drift, at a stirrup located approximately 300mm from the beam-foundation interface. A second, smaller sliding plane crack also opened up at the next stirrup, a distance of 425mm from the interface. Delamination of cover concrete and longitudinal bar buckling first occurred during the cycles to 2.71% drift. Degradation of the core concrete near the beam ends was present but to a lesser degree than in most other unrestrained specimens. Longitudinal bar rupture occurred on bars that had previously been subjected to a number of severe buckling cycles during the first cycle to 4.88% drift.

B.4 Specimen LD-1

Damage to specimen LD-1 during dynamic earthquake loading included flexural cracking, some bond splitting cracking, and a beam-foundation interface crack wide enough to permit some shear sliding.

A limited number of additional flexural cracks and a significant increase in bond splitting cracks were observed during the first static cycles (to 1.63% drift). The number of non-hairline flexural cracks continued to grow until reaching a maximum of six (south end) and five (north end) during the cycles to 2.17% drift. A non-hairline crack through the depth of the member at a location other than the beam-foundation interface also appeared during the cycles to 1.63% drift, at a stirrup
located approximately 300mm from the interface. However, shear deformations did not heavily concentrate at this crack, as sliding continued to occur at the interface crack. Delamination of cover concrete and longitudinal bar buckling first occurred during the cycles to 2.71% drift. Progressively worsening bar buckling and degradation of core concrete near the beam ends led to significant strength degradation and the test was stopped following cycles to 4.88% drift. Longitudinal bar rupture did not occur.

### B.5 Specimen P-2

Damage to specimen P-2 during dynamic earthquake loading included flexural cracking, bond splitting cracking, and a non-hairline crack through the depth of the member that enabled shear sliding. The sliding plane crack occurred adjacent to a stirrup located approximately 175mm from the beam-foundation interface.

Additional flexural and bond splitting cracks opened up during the first static cycles (to 2.44% drift). The number of non-hairline flexural cracks increased from three to six (south end) and two to six (north end) during the first static cycle. Delamination of cover concrete also occurred at 2.44% drift, during the second loading cycle. Bar buckling was first noted during cycles to 2.71% drift. Damage was heavily concentrated around the sliding plane crack that opened during dynamic testing. Severe degradation of the core concrete near the beam ends first occurred during cycles to 3.26% drift. The test was stopped following cycles to 4.34% drift, at which point the core concrete was almost entirely disintegrated. Longitudinal bar rupture did not occur.

### B.6 Specimen P-2-S

Damage to specimen P-2-S during the static earthquake loading included flexural cracking and some minor bond splitting cracking.

Additional flexural cracks and a significant increase in bond splitting cracks were observed during the first cyclic-loading cycles (to 2.44% drift). The number of non-hairline flexural cracks reached a maximum of eight (south end) and six (north end) during the cycles to 2.44% drift. An obvious sliding plane crack at a location other than the beam-foundation interface did not develop until late in the test. This was due to the cracks through the depth of the member being somewhat inclined, rather than horizontal. Eventually, a horizontal crack through the depth of the member that allowed some modest sliding deformations developed during the cycles to 3.26% drift, at a stirrup located approximately 300mm from the interface. Delamination of cover concrete occurred during the first loading cycle to 2.71% drift, which was immediately followed by bar buckling in the subsequent cycle. The lack of sliding deformations limited the dowel action of the longitudinal reinforcing
Test specimen damage progression

steel, causing the bar buckling to be predominantly due to compression instability between stirrups. Longitudinal bar rupture occurred during the second cycle to 3.80% drift, which is the earliest point that bar rupture occurred in any specimen. The early fracture may have been due to the severity of the bar buckling. The buckled bars exhibited particularly large curvatures for that stage in the test, as compared with other specimens.

B.7 Specimen LD-2

Damage to specimen LD-2 during dynamic earthquake loading included flexural cracking, bond splitting cracking, a non-hairline crack through the depth of the member that enabled shear sliding, and delamination of cover concrete. The sliding plane crack occurred adjacent to a stirrup located approximately 175mm from the beam-foundation interface.

A limited number of additional flexural cracks occurred during the first static cycles (to 2.44% drift). Additional bond splitting cracks above the delaminated cover concrete also appeared. The onset of bar buckling occurred during the second cycle to 2.44% drift. Damage was heavily concentrated around the sliding plane crack that opened during dynamic testing. Significant degradation of the core concrete near the beam ends first occurred during cycles to 3.26% drift. The core concrete degradation was somewhat asymmetrical, with degradation on one end being markedly more destructive than the other end. The test was stopped following cycles to 3.80%. Longitudinal bar rupture did not occur.

B.8 Specimen LD-2-S

Damage to specimen LD-2-S during the static earthquake loading included flexural cracking, bond splitting cracking, a non-hairline crack through the depth of the member that enabled shear sliding, and delamination of cover concrete. The sliding plane crack occurred adjacent to a stirrup located approximately 175mm from the beam-foundation interface. Prior to delamination, the number of non-hairline flexural cracks reached a maximum of four (south end) and six (north end).

No additional cracks formed during the cyclic part of the test. Longitudinal bar buckling was observed during the first cycle to 2.44% drift. Damage was heavily concentrated around the sliding plane crack that opened during the static earthquake testing. Significant degradation of the core concrete near the beam ends was first occurred during cycles to 2.71 % drift. The beam continued to follow this damage pattern, with progressively worsening sliding deformation, bar buckling and core concrete degradation, until the test was stopped following cycles to 4.88% drift. At this point the core concrete was almost entirely disintegrated. Longitudinal bar rupture did not occur.
B.9 Specimen CYC-NOEQ

All of the flexural and bond splitting cracks that developed on specimen CYC-NOEQ appeared during the two loading cycles to the first displacement increment (2.44% drift). The number of non-hairline flexural cracks on each end of the beam reached their maximum values of six (south end) and seven (north end) during the cycles to 2.44% drift. A non-hairline crack through the depth of the member also occurred at 2.44% drift, at a stirrup located approximately 175mm from the beam-foundation interface. Damage did not entirely concentrate around this sliding plane crack, as additional large inclined cracks through the depth of the member were present. Delamination of cover concrete and longitudinal bar buckling first occurred during the cycles to 2.71% drift. Significant degradation of the core concrete near the beam ends first occurred during cycles to 3.26% drift. Longitudinal bar rupture occurred on bars that had previously been subjected to a number of severe buckling cycles during the first cycle to 4.88% drift.

B.10 Specimen LD-1-R

Damage to specimen LD-1-R during dynamic earthquake loading was significantly more destructive than that to LD-1, despite the two specimens having had identical loadings applied. LD-1-R exhibited flexural cracking, bond splitting cracking, a non-hairline crack through the depth of the member that enabled shear sliding, and delamination of cover concrete. The sliding plane crack occurred adjacent to a stirrup located approximately 175mm from the beam-foundation interface. The damage to specimen LD-1-R was repaired following dynamic earthquake loading, except for hairline cracks that could not be epoxy injected and so were left in place.

During the first cycles of the static test (to 1.63% drift) a larger number of flexural cracks opened up than had been present at the end of the dynamic test. Longitudinal bond splitting cracks also reappeared during these first cycles. The number of non-hairline flexural cracks continued to grow until reaching a maximum of six (both ends) during the cycles to 2.17% drift. The length along which the non-hairline flexural cracks were distributed was longer than at the end of the dynamic test. Some of the cracks opened up in entirely new locations, while others opened close to cracks that had previously been repaired. A beam-foundation interface crack wide enough to permit shear sliding opened during the cycles to 1.90% drift. The sliding plane crack located 175mm from the interface that had previously opened during dynamic earthquake testing and been repaired re-opened during the cycles to 2.71% drift. Damage later concentrated around this sliding plane crack. Delamination of cover concrete and onset of bar buckling both occurred during the cycles to 2.71% drift. Subsequent cycles led to degradation of the core concrete near the beam ends. The test was stopped following cycles to 4.34% drift. Longitudinal bar rupture did not occur.
B.11 Specimen LD-2-R

Damage to specimen LD-2-R during dynamic earthquake loading was somewhat less destructive than that to LD-2, despite the two specimens having had identical loadings applied. LD-2-R exhibited flexural cracking, bond splitting cracking, and two non-hairline cracks through the depth of the member that enabled shear sliding. The sliding plane cracks were located at the beam-foundation interface and adjacent to a stirrup located approximately 300mm from the interface. The damage to specimen LD-2-R was repaired following dynamic earthquake loading, except for hairline cracks that could not be epoxy injected and so were left in place. Although the end concrete had not fully delaminated, it had partially delaminated, and so it was removed and recast during repair.

During the first cycles of the static test (to 2.44% drift) a larger number of flexural cracks opened up than had been present at the end of the dynamic test. Longitudinal bond splitting cracks also re-appeared during these first cycles. The number of non-hairline flexural cracks reached the test maximums of six (south end) and seven (north end) during the first static cycle. The length along which the non-hairline flexural cracks were distributed was longer than at the end of the dynamic test. Some of the cracks opened up in entirely new locations, while others opened close to cracks that had previously been repaired. A beam-foundation interface crack wide enough to permit shear sliding opened during the cycles to 2.44% drift. The sliding plane crack located 300mm from the interface that had previously been repaired did not re-open. A second sliding plane crack at a location over 400mm from the beam-foundation interface opened during the cycles to 2.71% drift. Delamination of cover concrete and onset of bar buckling both occurred during the cycles to 2.71% drift. Degradation of the core concrete near the beam ends occurred subsequently, with the worst degradation occurring just above the beam-foundation interface. The test was stopped following cycles to 4.88% drift. Longitudinal bar rupture did not occur.

B.12 Specimen LD-2-ER

Damage to specimen LD-2-ER during dynamic earthquake loading consisted of flexural and bond splitting cracking.

Additional flexural and bond splitting cracks opened up during the first static cycles (to 2.44% drift). The number of non-hairline flexural cracks reached test maximums of seven (south end) and five (north end) during the first static cycle. Delamination of cover concrete first occurred during the cycles to 2.71% drift, and bar buckling was subsequently observed during cycles to 3.26% drift. The bar buckling became progressively more pronounced until failure occurred due to bar rupture in the cycles to 4.34% drift. Shear sliding did not occur, although cracks that could have developed into sliding planes were visible. Degradation of the core concrete was minimal throughout the test.
B.13 Specimen CYC-ER

Specimen CYC-ER exhibited progressively increasing numbers of flexural cracks until the loading cycles to 1.09% drift, after which point few additional flexural cracks appeared. The number of non-hairline flexural cracks on each end of the beam continued to grow, until reaching maximums of eight (south end) and five (north end) during the cycles to 1.90% drift. Longitudinal bond splitting cracks first appeared in the cycles to 1.09% drift. Delamination of cover concrete first occurred during the cycles to 2.71% drift, and bar buckling was subsequently observed during cycles to 3.26% drift. The bar buckling became progressively more pronounced until failure occurred due to bar rupture in the cycles to 4.34% drift. Degradation of the core concrete was minimal throughout the test.

B.14 Specimen LD-2-LER

Damage to specimen LD-2-LER during dynamic earthquake loading included flexural cracking, bond splitting cracking, a non-hairline crack through the depth of the member that enabled shear sliding, and minor delamination of cover concrete. The sliding plane crack occurred adjacent to a stirrup located approximately 175mm from the beam-foundation interface. Additional flexural and bond splitting cracks opened up during the first static cycles (to 2.44% drift). Bar buckling was first observed during the cycles to 3.26% drift. Some degradation of the core concrete near the beam ends occurred. Limited dowel action was observed and bar buckling was predominantly caused by compression instability between stirrups. The bar buckling became progressively more pronounced until failure occurred due to bar rupture in the cycles to 4.34% drift.

B.15 Specimen CYC-LER

Specimen CYC-LER exhibited progressively increasing numbers of flexural cracks until the loading cycles to 1.09% drift, after which point few additional flexural cracks appeared. The number of non-hairline flexural cracks on each end of the beam continued to grow, until reaching maximums of five (both ends) during the cycles to 1.90% drift. Longitudinal bond splitting cracks first appeared in the cycles to 1.09% drift. A non-hairline crack through the depth of the member first occurred at 2.71% drift, at a stirrup located approximately 300mm from the beam-foundation interface. This sliding plane crack was angled slightly more diagonally than in most other specimens, which may have helped in reducing shear deformations. Bar buckling first occurred during cycles to 3.26% drift, and became progressively more pronounced until failure occurred due to bar rupture in the cycles to 4.34% drift. The level of degradation of the core concrete near the beam ends was modest throughout the test.
B.16 Specimen LD-2-LER-R

Damage to specimen LD-2-LER-R during dynamic earthquake loading included flexural cracking, bond splitting cracking, and minor delamination of cover concrete. The damage to specimen LD-2-LER-R was repaired following dynamic earthquake loading, except for hairline cracks that could not be epoxy injected and so were left in place. Although the end concrete had not fully delaminated, it had partially delaminated, and so it was removed and recast during repair.

During the first cycles of the static test (to 2.44% drift) a limited number of additional flexural cracks opened up in addition to what had been present at the end of the dynamic test. Longitudinal bond splitting cracks also re-appeared during these first cycles. The number of non-hairline flexural cracks reached the test maximums of seven (north end) during the static cycles to 2.71% drift. Some of the cracks opened up in entirely new locations, while others opened close to cracks that had previously been repaired. The south end of the beam delaminated during the first static cycle, possibly due to poor bonding between the repair mortar and the concrete. Bar buckling was first observed during the cycles to 3.26% drift, and became progressively more pronounced until failure occurred due to bar rupture in the cycles to 4.34% drift. Development of a wide sliding plane crack was not observed. Degradation of the core concrete was minimal throughout the test.

B.17 Specimen MONO

For specimen MONO, damage was only observed at various points where the monotonic loading was stopped, and therefore the drift values listed here are approximate. MONO exhibited progressively increasing numbers of flexural cracks until 1.36% drift, after which point few additional flexural cracks appeared. The number of non-hairline flexural cracks on the tension end of the beam continued to grow throughout the test, with a final number of approximately ten at test completion. The length over which the non-hairline cracks were distributed was substantially longer than in any cyclic test. Bond splitting cracks first appeared at 1.36% drift and the first observations of crushing of compression end cover concrete were at 3.26% drift. This damage pattern continued until test completion, with increasing number and widths of tension cracks and increasing degree of cover concrete crushing. Failure was not reached during testing and core concrete did not crush.
Test specimen damage progression

Figure B-1: Damage progression - specimen CYC
Test specimen damage progression

![Figure B-2: Damage progression - specimen CYC-DYN](image)

1.36%

2.17%

2.71%

Test completion (3.80%)
Test specimen damage progression

Figure B-3: Damage progression - specimen P-1
Test specimen damage progression

Dynamic EQ test completion (1.36%)

2.17%

2.71%

Test completion (4.88%)

Figure B-4: Damage progression - specimen LD-1
Dynamic EQ test completion (2.17%) 2.71%

Test completion (4.34%)

Figure B-5: Damage progression - specimen P-2
*Figure B-6: Damage progression – specimen P-2-S*
Test specimen damage progression

Dynamic EQ test completion (2.17%)  2.71%

Test completion (3.80%)

Figure B-7: Damage progression - specimen LD-2
Figure B-8: Damage progression - specimen LD-2-S
Test specimen damage progression

Figure B-9: Damage progression - specimen CYC-NOEQ

2.71%  Test completion (4.88%)
Figure B-10: Damage progression - specimen LD-1-R
Figure B-11: Damage progression - specimen LD-2-R
Figure B-12: Damage progression - specimen LD-2-ER
Figure B-13: Damage progression - specimen CYC-ER
Figure B-14: Damage progression - specimen LD-2-LER
Test specimen damage progression

Figure B-15: Damage progression - specimen CYC-LER
Test specimen damage progression

Figure B-16: Damage progression - specimen LD-2-LER-R
Test specimen damage progression

Figure B-17: Damage progression - specimen MONO

Test completion (15.93%)
Appendix C

Draft post-earthquake assessment methodology

This appendix presents a draft methodology for the detailed post-earthquake assessment of reinforced concrete buildings. The proposed methodology is not intended to function as a finished product, but rather as a way of identifying the research needs that are required before such an assessment procedure is practically applicable. The research needs related to quantifying the residual capacity and reparability of moderately damaged plastic hinges in ductile moment frames were addressed in the body of this thesis.

A flowchart of the proposed detailed assessment methodology, which builds on the general framework used in FEMA 306 [16], is shown in Figure C-1. The assessment procedure is meant to be used in conjunction with a seismic assessment guideline such as the New Zealand seismic assessment guidelines [29] or ASCE 41-17. The following sections discuss each step of the proposed methodology, and a list of identified research needs is given at the end of this appendix. It is highlighted that this methodology is targeted at detailed assessments, which are generally conducted by engineers that have been engaged by the owners of damaged buildings, and is not intended to be used for rapid assessments during a state of emergency. Similar to FEMA 306 and the JBDPA Post-Earthquake Guideline [19], this assessment procedure is focussed on determining the residual structural capacity of the lateral load-resisting system. Methods for assessing the residual capacity of floors or other gravity load-resisting elements (e.g. the method used in Wellington for assessing the residual capacity of precast flooring systems following the Kaikoura earthquake [9]), need to be used in conjunction with this assessment procedure.

C.1 Step 1 – Gather available information

In this step, all information relevant to the pre-earthquake or post-earthquake condition of the structure should be obtained. This includes:

- As-built structural drawings and any alteration or retrofit drawings (if available);
- Engineering calculations (if available);
- Ground motion records from the damaging earthquake (if available); and
- Post-earthquake damage inspections
Figure C-1: Proposed methodology for the detailed assessment of earthquake-damaged reinforced concrete buildings
Information on the observable damage should be recorded at both the system-level and the component-level. The system-level information should focus on the distribution of damage and be primarily targeted at identifying the governing lateral mechanism. Consideration should also be given to damage patterns that may indicate complex system-level behaviours such as excitation of torsional modes or interactions with the gravity system.

At the component-level, damaged members should be classified into those exhibiting either (i) flexural, or (ii) non-ductile behaviour. Among those exhibiting flexural damage, they should be further classified into those exhibiting ‘moderate’ or ‘severe’ damage, as defined in Section 1.4. Members not exhibiting damage do not have to be classified at this stage. All available damage metrics should be recorded, with the minimum damage information obtained for each flexural component including:

- Residual deformations (i.e. residual drift and residual elongation);
- Residual crack widths and crack distributions;
- Degree of spalling, if any; and
- Any other noteworthy damage, including buckling or fracture of longitudinal reinforcement, crushing or incipient crushing of core concrete, damage to transverse reinforcement, or evidence of shear sliding.

If uncertainties remain regarding the degree of damage after a visual inspection, additional investigation may be warranted (note that this may not become evident until progressing further into the assessment). A list of various testing procedures for obtaining information about the in-situ state of reinforced concrete structures is given in FEMA 306. Other methods not listed in FEMA 306, such as the reinforcement hardness tests currently being researched in New Zealand [12], may also be of use.

C.2 Step 2 – Identify critical non-ductile or severe damage

This step is performed early in the assessment in order to immediately identify any damage that would clearly be a limiting factor in the residual capacity of the building, and to gauge whether repair is feasible. It is appreciated that ‘non-ductile’ damage is a very broad category, and could involve damage such as wide shear cracks in joints or shear-critical members (whether or not calculations show these members to be shear-critical), severe longitudinal cracking in lap splice zones, or others. Severe flexural damage and the magnitude of residual drift (i.e. the building out-of-plumb) should also be considered in this step.

If substantial damage of this nature is identified, a repair feasibility study should be conducted as per Step 10. Only if repair is deemed feasible should the assessment proceed to Step 3; otherwise,
demolition should be recommended. If repair is deemed feasible, all future analysis should focus on the repaired condition, rather than the damaged condition. Accurate quantification of the residual capacity of a building with severe damage is here considered to be not necessarily possible or useful, as repair will always be implemented in practice prior to re-occupation of the building, regardless of the assessed residual capacity.

C.3 Step 3 – Perform analytical assessment

The purpose of this step is to use established analysis methods to determine the lateral mechanism of the structure and obtain an estimate of the demands incurred during the damaging earthquake. The procedures laid out in the chosen seismic assessment guidelines (e.g. the New Zealand seismic assessment guidelines or ASCE 41-17) can be followed, with the exception of using the ground motion records from the damaging earthquake instead of the ultimate limit state hazard. Research is needed into whether linear assessment procedures (e.g. modal analysis) can provide a sufficient estimate of the maximum deformation demand for the purposes of a post-earthquake assessment. Research is also needed into how non-linear static procedures could be used with a ground motion-specific response spectrum to obtain an estimate of the maximum deformation demand.

Ground motion records will typically not be available at the building site, and therefore the records used in the analysis have to be extrapolated from nearby recording stations. This extrapolation should account for the distance to the recording station and any difference between site conditions between the building and the recording station. Observations following the 2010-2011 Canterbury earthquakes demonstrated the importance of local site effects [155]. Research is needed into the accuracy of spatially extrapolated ground motions across different soil mediums.

C.4 Step 4 – Compare expected behaviour and observed damage

In this step, the building model should be updated, if required, to account for differences between the expected lateral mechanism and the observed damage distribution. This is an important step as there is little rationale in conducting a detailed post-earthquake assessment if the structural behaviour of the building in its undamaged state cannot be reasonably predicted.

It is appreciated that significant differences may exist between the predicted mechanism and observed damage distribution. A study that compared the predicted response (using non-linear response history analysis) and observed damage of three reinforced concrete buildings following the 2011 Christchurch earthquake concluded that the models tended to substantially over-predict the deformation demands [156]. If such differences are noted, additional factors that may have been
ignored in the initial analysis should be given consideration, including soil-structure interaction, foundation rocking, stiff non-structural elements, or higher mode effects.

Research is needed on how best to use visual damage indicators to update building models and provide better predictions of the structural response.

C.5 Step 5 – Extract best estimate of deformation demands

In this step, demand parameters of interest are extracted from the ground motion-specific analysis conducted on the building model, which may or may not have been updated based on the observed damage (Step 4). These demand parameters represent the best analytical estimate of the deformation demands that the building was subjected to during the damaging earthquake. The maximum inter-story drift will generally be the parameter of most interest.

C.6 Step 6 – Determine residual capacity of damaged components

This step forms the basis for a model of the building in its damaged state. This step is generally only applicable in cases of light-to-moderate damage, where quantification of residual capacity may be necessary for the purposes of assessing whether occupancy can continue prior to repair (see Section 1.4). The residual capacity of damaged components must be calculated in sufficient detail for analysis to be conducted according to the provisions of the chosen seismic assessment guideline. This means, at a minimum, the residual stiffness, strength (and overstrength if the component is intended to exhibit ductility), and deformation capacity must all be determined (see Section 1.5). Both the observable damage (Step 1) and the analytical estimate of demands (Step 5) may contribute to determining component residual capacity.

This is a significant area of research need, as codes for seismic assessment or new building design do not give consideration to earthquake damage and therefore are of little use for deriving the residual capacity of damaged components. FEMA 306, the JBDPA Post-Earthquake Guideline, and the work presented in this thesis all provide guidance in this area, but further research is required.

C.7 Step 7 – Account for special post-earthquake considerations

Extra consideration must here be given to issues that can be important in post-earthquake situations, but are not typically considered in seismic assessment or design. The residual fatigue life is of particular concern, as cumulative damage is not accounted for in modern seismic assessment guidelines. The work presented in this thesis showed that it is often appropriate to neglect cumulative measures when assessing the residual capacity or reparability of plastic hinge zones in
moment frames. However, further research is required to validate these findings, and research is needed how to incorporate considerations of cumulative measures into post-earthquake assessments in situations where neglecting them is not appropriate.

C.8 Step 8 – Update building model and repeat analytical assessment

This step involves updating the model previously developed in Step 3 to account for the damaged component properties (if coming from Step 7) or the repaired component properties (if coming from Step 11). The assessment procedures of the chosen seismic assessment guideline can again be used, with the hazard now taken as the ultimate limit state (or other desired limit state), as the objective is now to assess the seismic capacity of the structure in its current state.

C.9 Step 9 – Check results against performance objectives

This step corresponds to assessing the %NBS (percent of new building standard) if using the New Zealand seismic assessment guidelines, or comparing the analysis results against the target limit state if using ASCE 41-17. These analyses are typically governed by when the most critical element within the structure reaches its ultimate limit state, although some latitude is given to the judgment of the assessor if that element does not represent a significant life safety risk. If the building is found to be governed by a damaged or repaired component with a residual capacity as determined in Step 6 or Step 11, extra caution should be exercised to ensure that the effects of damage or repair were adequately accounted for.

If the structure cannot meet its performance objectives, repair options should be considered (i.e. proceed to Step 10). The design of adequate repair methods may be an iterative procedure requiring multiple repair re-designs and multiple updates to the building model. If all repair options have been exhausted and the building still cannot meet its performance objectives, demolition should be recommended.

If the analysis is carried out through nonlinear static analysis, determination of the deformation demand for a given seismic hazard typically requires an estimate of the damping as a function of the system ductility. Research is required to determine how to assess the appropriate damping ratio for damaged or repaired reinforced concrete structures.

C.10 Step 10 – Assess if repair is feasible

In cases where severe damage indicates that repair may not be technically or economically feasible, this step is conducted immediately after Step 2 to avoid extra unnecessary work on a building that
is certain to be demolished. In scenarios of moderate damage, this step is likely to result in an assessment that repair is feasible, and the analysis will proceed to Step 11. It is noted that a precise economic assessment may be impractical at this stage, and demolition should therefore only be recommended based on economic considerations if high repair costs relative to the value of the building are readily apparent.

C.11 Step 11 – Determine residual capacity of repaired components

Similar to the assessment of the residual capacity of components in their damaged state (Step 6), this step must also provide component capacities sufficient for the purposes of a detailed analysis using the provisions of the chosen seismic assessment guideline. This includes determining the residual stiffness, strength (and overstrength if applicable), and deformation capacity for each repaired component.

Substantial research is required in order to quantify the effects of various common repair methods on earthquake-damaged components. The research presented in this thesis focussed on quantifying the effects of repair by epoxy injection. In the case of complex or bespoke repair methods, the engineer recommending the repair would be expected to quantify the effects of the repair in terms of the residual stiffness, strength, and deformation capacity.

C.12 Summary of research needs

A proposed detailed post-earthquake assessment procedure, which draws on the methodology of FEMA 306, was introduced. A discussion of the step-by-step requirements for the proposed procedure highlighted the following areas of research need:

i. Research is needed into the applicability of using ground motion-specific records to estimate the prior deformation demand, using a variety of linear or non-linear assessment procedures (Step 3).

ii. Research is needed on how best to use ground motions from non-site-specific recording stations to define the characteristics of the damaging earthquake at the building site (Step 3).

iii. Research is needed on how best to use post-earthquake damage inspections to modify a building-level numerical model and obtain a better prediction of the structural response (Step 4).

iv. Research is needed to quantify the residual capacity of reinforced concrete components with light-to-moderate damage (Step 6). This was partially addressed by the work in this thesis.
v. Research is needed to determine whether factors that are not typically considered in design or assessment require consideration in post-earthquake situations. The primary factor identified here is the effect of cumulative loading (Step 7). This was partially addressed by the work in this thesis.

vi. Research is needed on how to estimate the level of damping in damaged or repaired buildings, for use with non-linear static analysis procedures (Step 9).

vii. Research is needed to quantify the effects of a variety of common repair techniques, both for moderate and severe damage states (Step 11). This was partially addressed by the work in this thesis.
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