Impact of prior shaking on earthquake response and repair requirements for structures – studies from ATC-145:

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ABSTRACT

The Federal Emergency Management Agency (FEMA) is funding an ongoing project focussed on *Development of Guidance for Repair of Earthquake Damaged Buildings to Achieve Future Resilience*. To date the project has focussed on when simple repairs (e.g. epoxy injection) are sufficient to ensure adequate future performance of earthquake-damaged ductile concrete frames and developing a process for engineers to determine whether this is the case. The process entails checks on system, component, and low-cycle fatigue behaviour. Studies underpinning these checks are summarised in this paper, and show that simple (or no) repairs are sufficient provided that:

- Storey drift does not exceed 2.0%, or
- Component chord rotation does not exceed 0.02 radians and low-cycle fatigue demands are insignificant according to criteria defined in the study.

It can thus be concluded that ductile concrete frames (i.e. frames able to sustain a displacement ductility of at least $\mu = 3$ and expected to form a beam sway mechanism) are generally robust, and that future life safety performance is unlikely to be degraded except following severe prior shaking.

1 INTRODUCTION

The Federal Emergency Management Agency (FEMA) is funding an ongoing project focussed on *Development of Guidance for Repair of Earthquake Damaged Buildings to Achieve Future Resilience*, which is managed by the Applied Technology Council (ATC). The ATC project reference, ATC-145, is used throughout this paper as shorthand for the project. To date the project has focussed on identifying when simple repairs (i.e. epoxy injection, patch repair of spalling, and similar repairs) are sufficient to ensure adequate future performance of earthquake-damaged ductile concrete frames (i.e. frames able to sustain a displacement ductility of at least $\mu = 3$ and expected to form a beam sway mechanism). The inspection and assessment framework developed to determine whether simple repairs are sufficient is summarised in Figure 1a. This framework includes an inspection and analysis phase aimed at estimating the peak drift demands imposed during a damaging earthquake. These drift estimates are then used as the basis for assessing the impact of the damaging earthquake on the future performance of structural components of the frame. Both future life safety and (optionally) serviceability performance is considered. The outcomes of these assessments determine whether adequate performance of the building during a future earthquake would be expected following only simple (or no) repair, or whether complex repairs are required.

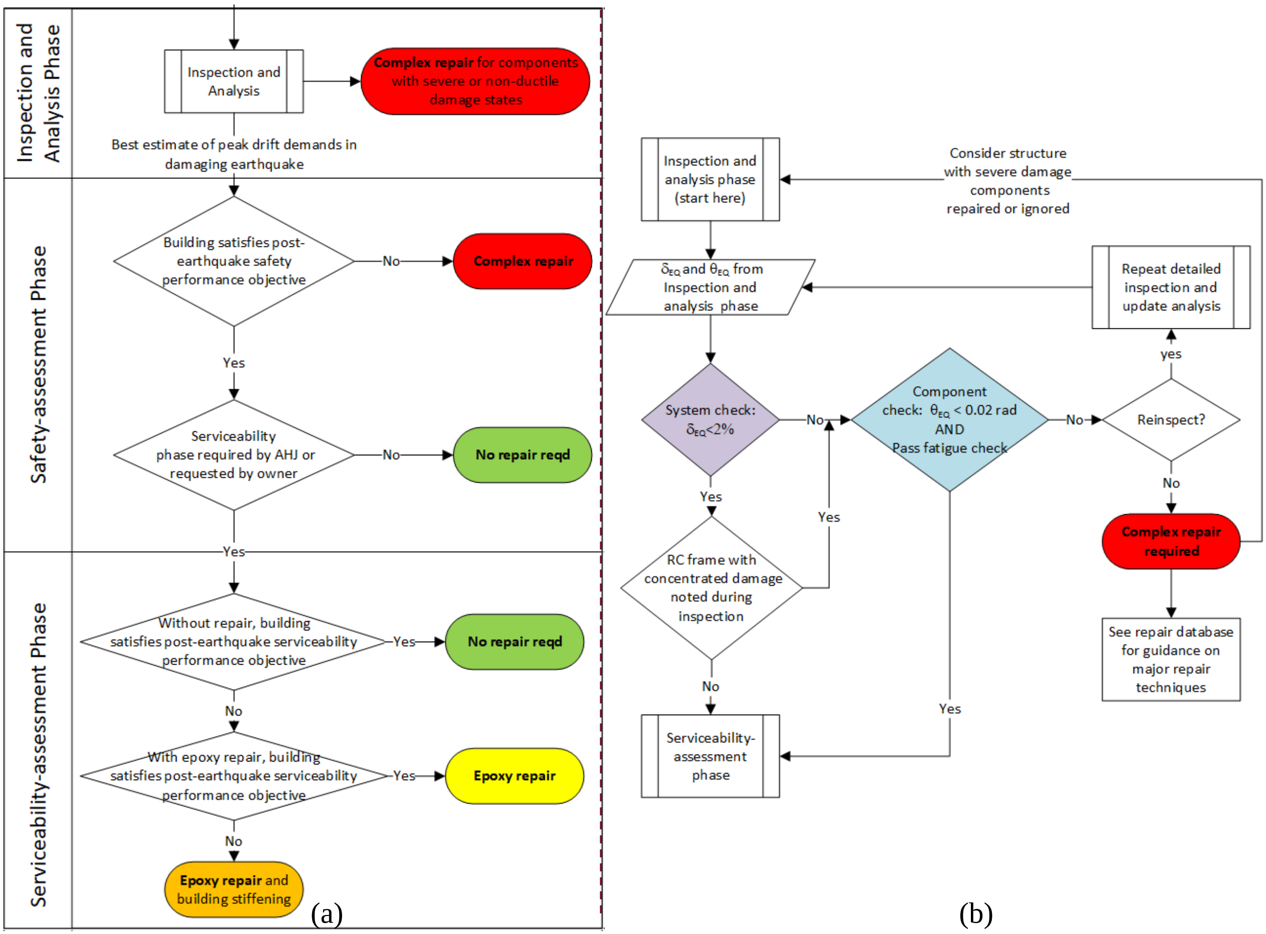

Figure 1: (a) Assessment framework flowchart and (b) safety-assessment phase

As shown in Figure 1b, the safety-assessment phase of the framework comprises checks on the maximum drift sustained by the building ('system check' – purple diamond), the maximum chord rotation imposed on beams ('component check' – blue diamond), and the extent of fatigue demand on longitudinal reinforcement ('fatigue check'). This paper focusses on the studies underpinning these checks, comprising:

- Analytical studies (underpinning the system check) to determine storey drift limits below which simple repairs are sufficient to ensure that future performance is not impaired,
- Consideration of experimental studies to determine component deformation limits below which simple repair is sufficient, and
- Studies to determine when reinforcement low cycle fatigue can be considered inconsequential.

2 STUDIES UNDERPINNING THE SYSTEM CHECK

The studies underpinning the system check were intended to identify an appropriate damage measure and threshold above which damage impairs future performance and, consequently, necessitates complex repair. Previous work has assessed the effect of repeated earthquakes (e.g. Amadio et al. 2003; Chintanapakdee and Chopra 2004; Mahin 1980; Michalis et al. 2006; Raghunandan et al. 2015) and found that a building's stiffness, strength, deformation capacity, and susceptibility to P-Delta effects influence the change in performance due to structural damage, but that ductile buildings can undergo significant deformations before experiencing a major increase in collapse risk. A computationally efficient framework for assessing the effect of damage on future performance of reinforced concrete frame structures was developed and applied to four frames representative of ductile buildings in the United States to develop the 'system check' referred to in Figure 1b. The results are also considered relevant for ductile frames in New Zealand, but do not account for any influence of precast floors.

2.1 Buildings and analytical models

Four frame buildings were analysed to illustrate the framework and quantify the effects of building characteristics on the repair trigger. The U.S. code-conforming buildings were based on buildings designed by Haselton (2006) that were updated to meet requirements of ASCE/SEI 7-16 (2016) by the ATC-123 project (Applied Technology Council 2018). Each building has the same floor plan but varying height, with perimeter "special" moment frames that are roughly equivalent to New Zealand ductile or limited ductile frames. Interior frames were designed to resist gravity actions and deformation demands from seismic loads.

Analysis of the buildings was undertaken using OpenSees (McKenna et al. 2010), as illustrated in Figure 2a. Nonlinear response was captured using lumped plasticity models connected by elastic beam-column elements. Ibarra-Medina-Krawinkler (IMK) hinges (Ibarra et al. 2005) were used with properties calibrated according to member details using equations developed by Haselton et al. (2016) that suggest only modest cyclic deterioration. The panel zone is modelled with nearly rigid elastic elements, while diaphragms were assumed to be rigid. Gravity actions were applied as mass and distributed loads, and P-delta effects were considered. Static pushover results are illustrated in Figure 2b and other results are summarised in Table 1.

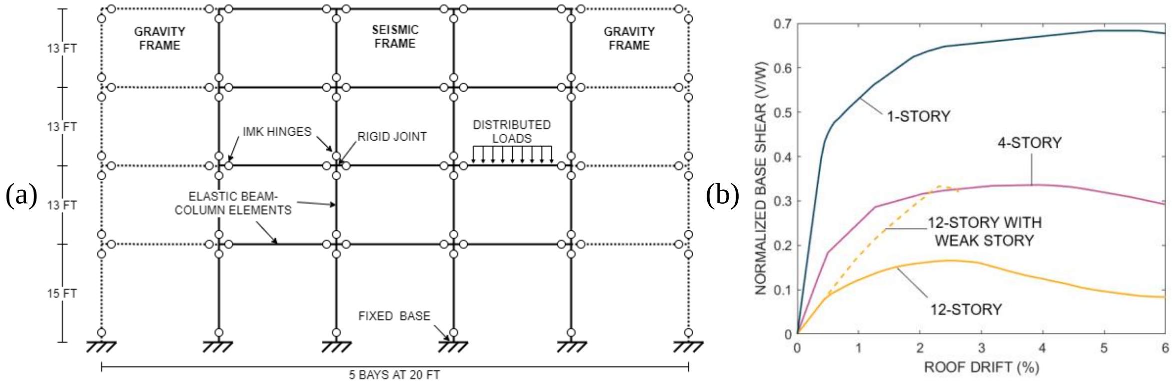

Figure 2: (a) Typical perimeter frame configuration showing elements used in numerical models. (b) Response of the four reinforced concrete frame buildings under static pushover loading.

Table 1: RC frame buildings assessed for reparability in this study, with repair triggers

Stories	Design	T(s)[1]	V_{max}/W[2]	Ultimate storey drift (%)[3]	Storey drift (%) in undamaged building[4]	Repair trigger (% storey drift)
4	Code-conforming	1.0	0.33	4.9	2.7	3.2
12	Code-conforming	2.4	0.16	5.8	3.4	2.1
12	Code-conforming with weak story	2.3	0.30	4.0	2.8	2.5
1	Code-conforming	0.4	0.68	5.0	1.5	2.2

2.2 Methods used in developing system check for repair trigger

The models of each building were analysed using the framework summarized in Figure 3. This framework involves quantifying the change in seismic performance that results from damage, which is quantified as the amplification of drift demands between the damaged and the undamaged building when subjected to the same ground motion. Drift demands are assessed at the ground motion level of the risk-targeted maximum considered earthquake (MCE_R). For each building, 15 "performance assessment" ground motions were selected for this level for a site in California, U.S.A. using the conditional mean spectrum selection tool developed by Baker (2011). Each building was subjected to each of these motions to interrogate the response of each undamaged building, giving 60 analyses in total with resulting drift demands listed in Table 1.

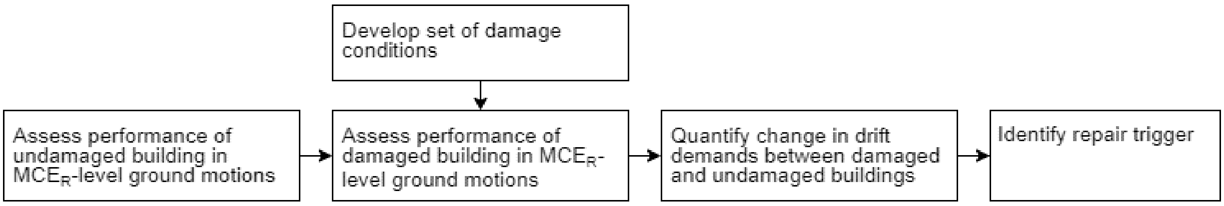

Figure 3: Framework for assessing the level of damage at which a building's seismic performance is impaired, i.e., the repair trigger.

Next the performance of the damaged building in the MCE_R-level ground motions through back-to-back analyses was assessed, as illustrated in Figure 4. In these back-to-back analyses, the building is first "damaged" with an initial motion, and then subjected to the same performance assessment (MCE_R-level) ground motions applied to the undamaged building. Initial damaging motions of various intensities were selected to induce a range of damage conditions, quantified by peak story drifts in the damaging motion, such that the effect of different levels of damage on MCE_R drift demands could be investigated.

For each building, damage condition, and performance assessment motion, the peak story drift for the damaged building was compared to the peak story drift for the undamaged building. This ratio, which captures drift amplifications, was used to quantify any change in performance due to building damage.

The drift demand ratio, quantifying drift amplification, is plotted against the demand in the damaging motion (in terms of peak story drift), as shown in Figure 5 for the 1-story, code-conforming building. A bilinear regression is fit to these data to identify the level of damage, or repair trigger, at which performance is affected. This regression is used to identify the range over which the ratio of drift demands is (on average) 1.0 (indicating performance is generally unaffected by prior damage) as well as the amplification of drift demands as damage increases. For the building and results shown in Figure 5, the repair trigger is 2.2% drift, indicating repair is not needed if drift during the damaging ground motion did not exceed 2.2%.

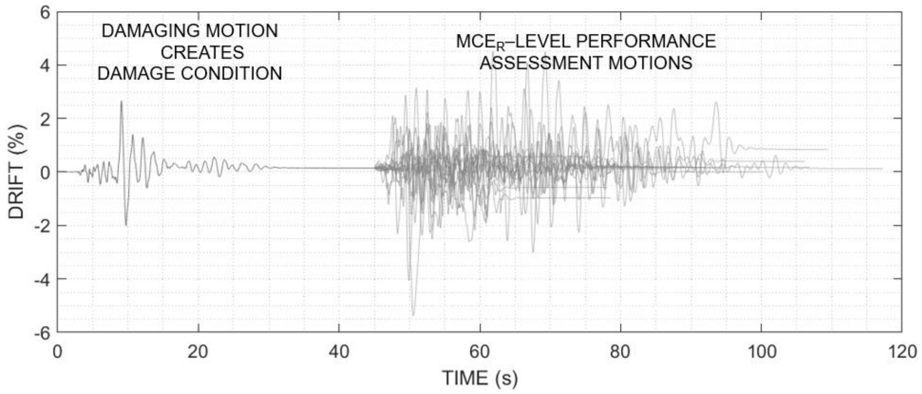

Figure 4: Story drifts for one damage condition for one of the buildings. Each performance assessment motion is preceded by a damaging ground motion that creates a selected damage condition (i.e. damaging motion peak drift demand).

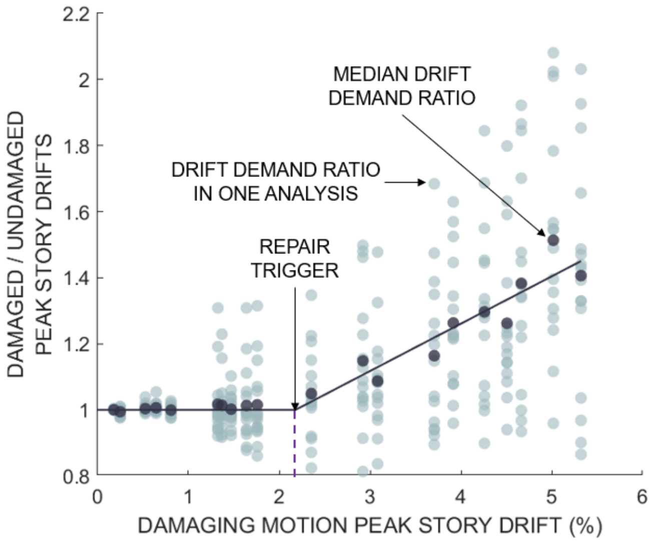

Figure 5: Relation between drift amplifications (damaged/undamaged building peak story drifts) and drift in the damaging motion. A bilinear regression is fit to these data and identifies a "corner point", or repair trigger, that corresponds to the level of damage at which performance in a future event is impaired.

2.3 Results and proposed system check for ductile RC frame structures

For each building, a repair trigger, or level of damage at which seismic performance is impaired, was identified as listed in Table 1. Structures with less deformation capacity have lower repair triggers. The results are in agreement with previous analytical work (e.g. Li et al. 2012; Raghunandan et al. 2015) that indicates that ductile buildings can safely withstand drifts below 2% without significant impairment of future performance, and experimental work (e.g. Abrams and Sozen 1979; Cecen 1979; Kajiwara et al. 2017; Laughery 2016; Schultz 1986; Wood and Sozen 1985) that also indicates that moderate prior shaking does is unlikely to affect future performance. The system check is expected to be system dependent, i.e. it is expected that different drift limits (or even damage parameters) would be applicable to (e.g.) wall buildings. Ongoing ATC-145 work is exploring appropriate system checks for less ductile frames and wall systems.

3 STUDIES UNDERPINNING THE COMPONENT CHECK

The component-level check is triggered if the system-level safety check is not satisfied (see Figure 1). The primary objective of the component-level check is to assess if prior earthquake demands have reduced the deformation capacity of ductile RC frame elements. Figure 6 outlines the component check process. A safe component deformation limit, θ_{Lim} is proposed, below which earthquake demands are deemed to not impact the deformation capacity of ductile RC frame elements. Estimates of the earthquake demands, θ_{EQ} from the Inspection and Analysis Phase of the assessment framework are then compared against the safe deformation limit, as shown in Figure 6. A brief summary of the studies and analysis used to propose the safe component deformation limit used in the assessment framework is given below.

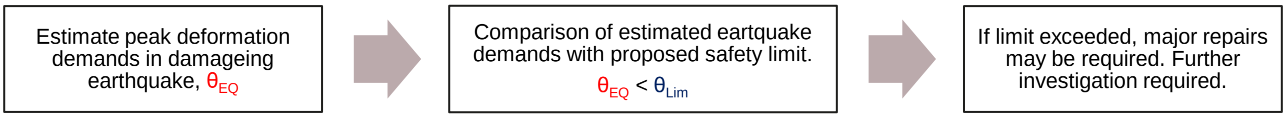

Figure 6 Significance of the proposed safe deformation limit in the context of general post-earthquake assessment frameworks.

3.1 Prior Experimental Investigations

Surveying existing literature on the seismic performance of ductile RC components, a subset of studies with a focus on varying loading histories were identified for an initial proposal of a safe deformation limit. A summary of three column studies and one beam study from the assessed literature are provided below.

3.1.1 El-Bahy et al. (1999)

El-Bahy et al. (1999) conducted an experimental program on 12 nominally identical circular reinforced concrete cantilever column specimens. The columns had shear span to depth ratios of 4.5, reinforcement ratios of 2.08%, and unusually small transverse reinforcement spacing, s, equal to two times the longitudinal bar diameter, d_b which precluded performance degradation due to bar buckling. All specimens were tested under a constant axial load ratio of $0.1A_g f'_c$. A variety of loading histories were applied to the columns. As shown in Figure 7, Specimen A1, subjected to monotonic loading, was able to achieve a lateral drift of 11% prior to 20% reduction in lateral resistance. Other protocols applied to the specimens included a typical cyclic loading protocol and constant amplitude cycles to failure at 2%, 4%, 5.5% and 7% lateral drift. Notably for this study, specimen A3 sustained 150 cycles of 2% drift without failure, followed by a monotonic push to 11% drift prior to the test being stopped. In contrast, specimens subjected to higher constant amplitude loading failed after 30 or fewer cycles. These tests therefore indicated that when bar buckling is precluded, cycles at or below 2% drift had little to no impact on the deformation capacity and showed a marked decrease in cyclic capacity when amplitude was increased from 2% to 4% drift.

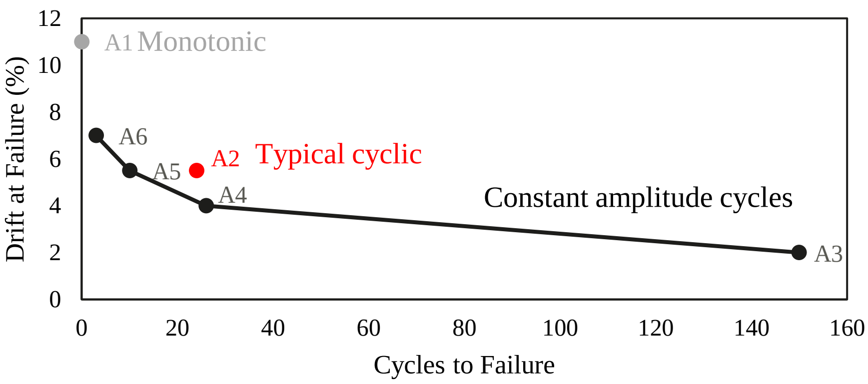

Figure 7 Variation in cycles to failure for column specimens tested by El-Bahy et al. (1999)

3.1.2 Pujol et al. (2006)

Pujol et al. (2006) conducted an experimental investigation on the seismic performance of eight column assemblies. The specimens consisted of two cantilever column specimens connected to a central stub and tested under a constant axial load ratio of 0.1 or $0.2A_g f'_c$. Specimens were constructed with transverse reinforcement spacings corresponding to s/d_b ratios of 2, 3 or 4. Varying displacement histories were applied to the columns consisting of initial cycles at either 1%, 2% or 3% drift, followed by cycles to failure at either 3% or 4% drift. Three specimens were tested with an s/d_b ratio of 3, all tested at 0.1 axial load ratio. The focus with these specimens was the impact of variation in applied cyclic content. A specimen subjected to constant amplitude loading at 3% lateral drift was able to withstand 15 fully reversed cycles prior to failure. Comparison specimens were subjected to seven initial cycles at 1% or 2% drift, prior to cycling to failure at 3% drift. These specimens showed that cycles at 1% drift did not affect the number of cycles to failure, while the cycles at 2% drift caused a minor reduction in the number of cycles sustained. Similar results were also achieved while testing specimens with s/d_b ratio of 4. Overall, the results suggest that cyclic loading up to 2% drift had a limited impact on the deformation capacity of column specimens with up to 0.1 axial load ratio.

3.1.3 Xing et al. (2017)

Xing et al. (2017) focussed on the application of low cycle fatigue loading histories to column specimens. The results for 5 of the columns are considered here. All columns were tested in identical conditions with axial load ratio of 0.2, s/d_b ratio of 4.55 and a load span to depth ratio of 4.38. The specimens discussed here were subjected to constant amplitude cycles at 5.1%, 4.1%, 3.1%, 2.4% drift as well as one specimen subjected to a combination of 1.4% and 2.5% drift cycles. As for the tests discussed by El-Bahy et al. (1999), a progressive reduction in the number of cycles to failure was observed as the magnitude of the constant amplitude cycles was increased (Figure 8). Comparison of specimens C6 and C8 is notable; specimen C6 was able to withstand 136 cycles at 2.4% drift prior to failure, while specimen C8 was subjected to 1000 cycles at 1.4% followed by 136 cycles at 2.5% prior to failure. These results show a lack of impact from cycles up to 1.4% on the deformation capacity of the tested specimens and the large number of cycles which can be sustained at 2.4-2.5% drift.

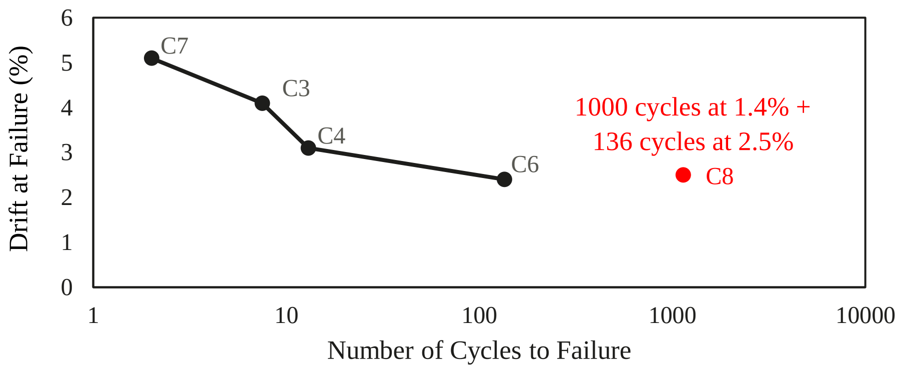

Figure 8 Variation in cycles to failure for column specimens tested by Xing et al. (2017)

3.1.4 Marder (2018)

As part of a study on the residual capacity and repair of reinforced concrete beams, Marder (2018) investigated 17 different configurations of loading histories, axial restraint and loading rate on the behaviour of ductile, flexure-controlled beam elements. The specimens were all constructed with s/d_b ratios of 6 and tested with a shear span to depth ratio of 3.58. Figure 9 presents a comparison of specimens with three different loading protocols: monotonic (Mono), standard cyclic (CYC), and standard cyclic but omitting all cycles below 2.17% (CYC-NOEQ). A clear difference is seen between the cyclic and monotonic specimens; however, the figure demonstrates there was essentially no difference in the performance of the cyclic specimens, regardless of the omission of cycles at or below 2.17% drift, highlighting the limited impact of these lower deformation cycles.

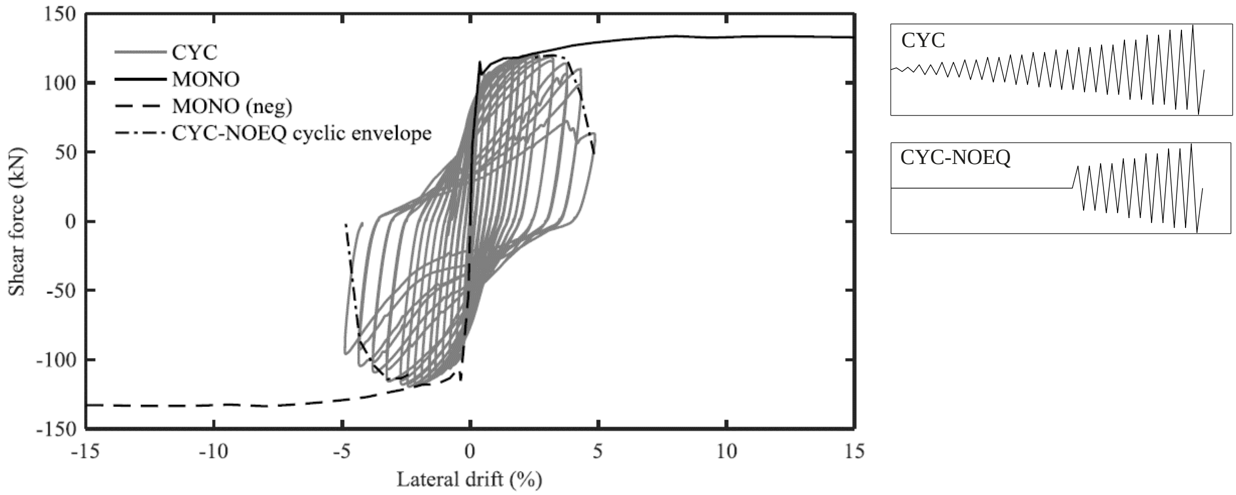

Figure 9 Comparison of response from beams tested by different loading protocols illustrated at right (only cyclic envelope of CYC-NOEQ shown for clarity). Figure adapted from Marder (2018).

3.2 Safe Deformation Limit

A safe deformation limit corresponding to a chord rotation of θ_{Lim} = 0.02 rad has been proposed for safety assessment of ductile RC frame components. This limit is based on the summaries presented in the previous section along with further work to assess the impact of cyclic loading between 1-2% on the deformation capacity of ductile frame elements. More detailed analysis of test results from the studies summarised above showed no significant correlation between the number of cycles at or below 2% component drift and deformation capacity. Following initial development, the hypothesis of a 2% limit was tested using the 153 column specimens from the ACI rectangular column database (Ghannoum and Balaji 2012), which confirmed that no significant correlation of deformation capacity with variations in cyclic loading was observed until a component drift of 2.5%, after which a weak correlation was observed. Ongoing work under ATC-145 is identifying θ_{Lim} for nonductile frame components and walls.

4 STUDIES UNDERPINNING THE FATIGUE CHECK

While not commonly the case, low cycle fatigue (LCF) may limit the performance of a reinforced concrete element if other failure modes are suppressed by adequate detailing (Dutta and Mander 2001). Consequently, consideration is required regarding when LCF may necessitate complex repairs (e.g. replacement of bars).

The approach to assessment of fatigue damage taken in the ATC-145 project builds on the consideration of reinforcement fatigue that exists in literature from the last three decades (e.g. Brown and Kunnath 2004; Mander et al. 1994; Marder 2018; Zhong and Deierlein 2019). In common with this literature the approach taken is based on equations that relate the imposed plastic (Coffin 1953; Manson 1953) or total (Koh and Stephens 1991) strain amplitude to the number of half cycles that cause failure, with the fatigue damage from demands of varying amplitude (e.g. an earthquake) calculated using ‘Miner’s rule’ (Miner 1945).

Assessment of LCF is only required for bars that have been exposed by spalling but have not visibly buckled. Visibly buckled bars are assumed to require complex repair, while bars not exposed by spalling are assumed to be unaffected by LCF. This latter assumption is based on the fact that bars cannot buckle without spalling occurring, and the findings of both the ATC-145 study and others (Marder 2018) that the number of strain cycles that can be sustained by unbuckled bars is greatly in excess of the demands imposed by earthquakes.

4.1 Fatigue performance measures

Two performance measures are considered to represent reasonable alternatives for determining whether the residual fatigue life of reinforcement in an earthquake-damaged structure is sufficient. These are:

1. The fatigue life of the reinforcement not having been reduced by more than 10%, or
2. The residual fatigue life being sufficient to withstand a future 'maximum considered' earthquake (MCE).

While not justified by specific science, the 10% threshold represents a level of degradation that has no more than a minor impact on the risk of fatigue failure during a future earthquake. Notably the same value has been proposed on occasions as 'de minimis' by insured's engineers during discussions of insurance claims. The second criterion indicates sufficient residual fatigue life to resist future demands irrespective of any degradation that has occurred. Fulfilment of either of these criteria provides sufficient surety about the condition of reinforcement; it is not necessary for a structure to fulfil both. Analyses for the ATC-145 project have focussed on the first criterion. The second criterion cannot easily be applied in a generic study because it requires knowledge of the specific characteristics of a building and the hazards it is exposed to. Analysis focussed on the second criterion may be undertaken where a damaged structure does not achieve the first criterion based on the generic approach outlined below.

4.2 Representation of cyclic earthquake demand

Calculation of fatigue damage requires not only an estimate of the peak deformation imposed on a structural element, but also an estimate of the complete deformation history imposed during a damaging earthquake. Rather than incurring the complexity of considering actual earthquake records, the simplified fatigue life assessment adopted in the ATC-145 process is based on the assumption that the cyclic deformation imposed by an earthquake may be satisfactorily approximated by consideration of an artificial displacement history of the type described in FEMA 461 (FEMA 2007) for quasi-static cyclic testing.

The FEMA 461 displacement history specifies that increasing increments of displacement be applied to a component, with two reversed cycles applied at each drift increment. While deemed appropriate for testing of structural components, two reversed cycles per drift increment is not necessarily representative of the fatigue demands imposed by a particular earthquake. Instead, based on consideration has been given to the number of effective cycles anticipated during a 'typical' earthquake following the approach outlined by Malhotra (2002), fatigue calculations for the ATC-145 project have been undertaken based on three reversed cycles of demand at each drift increment. This is felt to generally provide a degree of conservatism, but may not be sufficient for long duration (e.g. significant duration, $D_{5\text{-}95}$, in excess of 45 seconds) earthquakes.

4.3 Assessment of fatigue damage

Assessment of fatigue damage has focussed on the damage expected in components subjected to the 0.02 radian chord rotation identified as the component deformation limit (Section 3 above). Taking this rotation as the maximum deformation for the deformation history discussed previously, the strains for each deformation increment have then been calculated for cantilever elements with varying aspect ratios. The resulting fatigue demand corresponding to this strain history has then been calculated using fatigue life relationships (Brown and Kunnath 2004; Marder 2018) that are typical of range of relationships available in the literature.

The calculations undertaken assume a triangular distribution of curvature along the element until yield. The effective yield curvature has been calculated with account made for non-flexural sources of deformation such as shear and bar slip (Opabola and Elwood 2020). Plastic deformation was assumed to occur within an effective plastic hinge length between 30% and 60% of the member depth centred on the base of the cantilever. The fatigue damage calculation is affected by the reinforcement compression strain. This is not readily calculable due to its dependence on the tensile strain arising from previous displacement. For this study it has been assumed that the compression strain is equal to -0.5 times the tensile strain, but not more than -0.01.

Output from an example analysis is shown in Figure 10a. This shows:

- Blue line - typical displacement history used, comprising 3 cycles per displacement increment with a maximum chord rotation (drift) of 2.0%, and
- Red line - Miner's fatigue damage sum, determined using Brown & Kunnath (2004) parameters for No. 8 bars and with the total damage sum reaching 9% at the end of the deformation history.

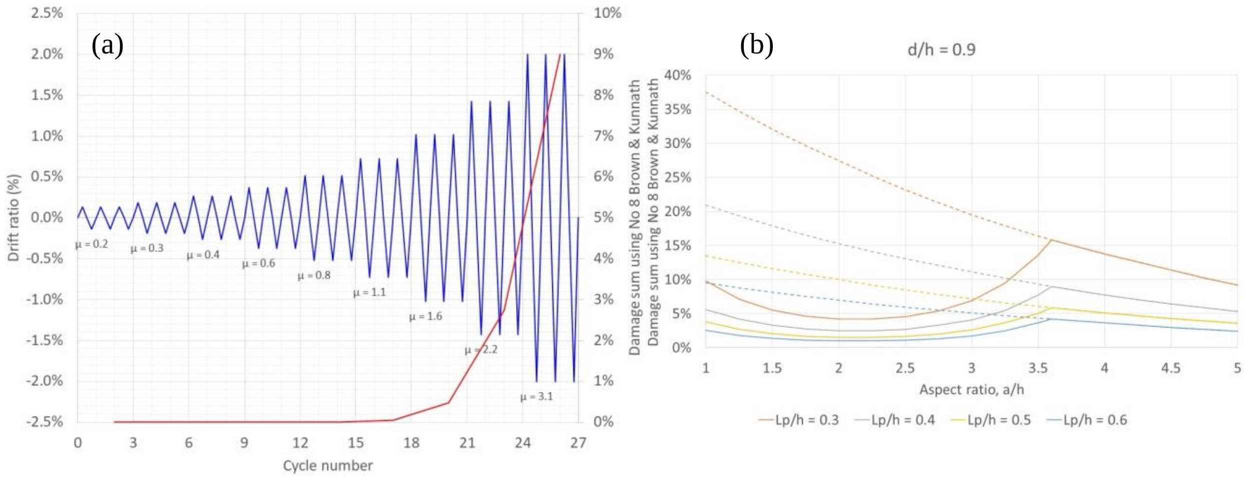

Figure 10: (a) Fatigue damage progression for cantilever element with aspect ratio of a/d = 4, d/h = 0.9, and l_p/h = 0.4, and (b) Impact of aspect ratio and plastic hinge length on fatigue damage sum

The results of a parametric analysis is shown at the right of Figure 10b where the variation of fatigue sum at end of the adopted loading protocol is plotted against aspect ratio (a/h) for different effective plastic hinge lengths. Solid lines show the damage sums calculated including account of non-flexural (shear and bar-slip) deformation characteristics, while dashed lines show how the fatigue damage sums with reducing aspect ratio if these sources of deformation are ignored.

The solid lines, representing the realistic assumption that non-flexural deformations contribute significantly, show that the fatigue damage sum does not exceed approximately 10%, provided the effective plastic hinge length is 0.4h (40% of the member depth) or greater. This conclusion was not sensitive to the parameters used, including the fatigue life relationship, meaning that fatigue is unlikely to be consequential provided:

- The maximum chord rotation is less than 0.02 rad,
- The significant duration ($D_{5\text{-}95}$) of the damaging earthquake was less than 45 seconds, and
- The effective plastic hinge length is greater than 0.4 times the member depth when calculated on the basis commonly adopted in New Zealand (NZSEE et al. 2018).

The focus of the study described in section 4.3 has been on grade 60 reinforcement that is not susceptible to strain ageing as is typically used in the U.S. Additional work is required to cover situations where longitudinal reinforcement is susceptible to strain ageing such as when mild steel (e.g. Grade 300) reinforcement is used.

5 SUMMARY AND CONCLUSIONS

Studies undertaken to date for the ATC-145 project have shown that ductile concrete frames are generally robust, and that future life safety performance is unlikely to be degraded provided:

- Storey drift does not exceed 2.0%, or

- Component chord rotation does not exceed 0.02 radians and low cycle fatigue demands are insignificant.

If a ductile concrete frame complies with the criteria above, then no more than simple repairs (e.g. epoxy injection) are considered necessary to achieve satisfactory future earthquake performance. These conclusions should currently only be taken as applying to ductile concrete frames. Ongoing ATC-145 studies are seeking to extend this work to cover walls and non-code conforming frames.

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ATC-145 is an ongoing project, and conclusions reached to date are subject to change. The authors are solely responsible for the accuracy of statements or interpretations contained in this publication. No warranty is offered with regard to the results, findings and recommendations contained herein, either by the Federal Emergency Management Agency, the Applied Technology Council, its directors, members or employees. These organizations and individuals do not assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any of the information, product or processes included in this publication.

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