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Monotonic and low-cycle fatigue properties of earthquake-damaged New Zealand steel reinforcing bars. The experience after the Christchurch 2010/2011 earthquakes

Giuseppe Loporcaro[a], Alberto Cuevas[a], Stefano Pampanin[b,a], Milo V. Kral[a]

[a]*University of Canterbury, 20 Kirkwood avenue, Christchurch 8041, New Zealand*
[b]*Sapienza University of Rome, Piazzale Aldo Moro 5, Rome 00185, Italy*

Abstract

The 2010 and 2011 Christchurch seismic events have highlighted the limitations of the current knowledge in assessing the residual capacity of earthquake-damaged reinforced concrete (RC) buildings. An important challenge during the assessment phase was determining the residual ductility and the remaining low-cycle fatigue life of damaged rebars. Low-cycle fatigue is a possible failure mechanism of steel reinforcing bars when subjected to large-amplitude cyclic loads, such as due to earthquakes. While a single seismic event may not cause rebar failure, the low-cycle fatigue life will be reduced due to plastic strain. Also, New Zealand (NZ)-manufactured Grade 300E is prone to strain ageing. This phenomenon causes a change in mechanical properties, such as increase in yield and ultimate tensile strength, return of a discontinuous yield point, reduction in ductility and rise in the ductile/brittle transition temperature, and must be considered in damage assessment.
This paper discusses the effects of strain ageing on the monotonic and cyclic steel mechanical properties. Low-cycle fatigue tests were conducted on Grade 300E steel rebars. Reinforcing bar samples were subjected to constant and fully-reversed strain amplitude cycles. Strain amplitudes ranged from 0.5% to 3%. The strain-fatigue life curve for the un-aged steel was determined. The strain ageing effects on the fatigue life of Grade 300E were then investigated. Specimens were cyclically tested up to the 33% and 66% fatigue life previously determined and “artificially” aged at 100°C. Finally, they were cyclically tested until failure.
The experimental data were analyzed and low-cycle fatigue models were calibrated using the Coffin-Manson empirical relationship. Fatigue lives of the un-aged and aged samples were then compared. Preliminary observations suggested that strain-ageing triggers a premature crack initiation which propagates until failure.

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Keywords: earthquake damage; residual ductility; steel reinforcement; low-cycle fatigue, strain ageing

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1. Introduction

On the 4 September 2010, a magnitude Mw7.1 earthquake hit the city of Christchurch. Five months later, on the 22 February 2011, a Mw6.3 earthquake hit Christchurch again. Because of the proximity to the city and the acceleration produced, the 22 February 2011 earthquake was more destructive. Reports from the after-event building survey showed that 50% of the reinforced concrete (RC) buildings in the Christchurch central business district (CBD) was tagged either red (no entry) or yellow (restricted entry) (Kam & Pampanin, 2011). A report from the Canterbury Earthquakes Royal Commission (CERC) stated that, in total, approximately 1100 CBD buildings were planned to be demolished (CERC, 2012).

The after-event damage inspections showed that pre-1970s buildings, designed before the introduction of "modern codes" based on the concepts of capacity design and hierarchy of strengths, performed inadequately. Structural deficiencies, typical of those buildings, such as insufficient steel reinforcement and confinement, use of plain reinforcing bars (rebars), inadequate anchorage details, and irregular plan and elevation configurations, caused a series of brittle failures. Differently, ductile buildings designed according to the modern codes performed as expected. Plastic hinges formed in the desired locations (beam ends, column and wall bases and coupling beams), while no damage was observed in columns and in beam-column joints (Kam & Pampanin, 2011).

The Christchurch earthquakes highlighted a critical issue: the assessment and reparability of damaged buildings. Methodologies able to estimate the level of subsequent earthquakes that RC buildings could still sustain before collapse were unknown. Neither repair strategies capable of restoring the initial condition of buildings were known. These aspects, added to nuances of New Zealand (NZ) building owners' insurance coverage, encouraged the demolition of many buildings. Moreover, government and industry required information about the state of damage of steel reinforcement of cracked RC elements. In detail, information such as the amount of plastic deformation experienced by the rebars during the earthquake, correlation between crack location and width with the damage of rebars, residual ductility of the rebars, and the remaining number of cycle to fracture were required (CERC, 2012).

1.1. Strain ageing

Many low-carbon steels such as those used for rebars, when plastically deformed, are subjected to a time- and temperature-dependent phenomenon known as strain ageing. This phenomenon causes changes in mechanical properties such as an increase in yield strength and in ultimate tensile strength, and a reduction in ductility. Therefore, steel rebars plastically deformed during earthquakes might be subjected to this phenomenon (Loporcaro, Pampanin, & Kral, 2016; Momtahan, Dhakal, & Rieder, 2009). The changes in mechanical properties caused by strain ageing can be appreciated by the example showed in Figure 1. The stress-strain curve in Figure 1 refers to a specimen machined from a NZ-manufactured Grade 300E steel reinforcing bar. The specimen was originally strained beyond yielding, up to stress A, and then unloaded. When the specimen is immediately reloaded, it will show an elastic behavior up to stress A, then strain hardening will continue following the original stress-strain curve. When, after unloading, the specimen is aged for a period of time (e.g. 1 year) at "ambient" temperatures and then reloaded again, a return of the discontinuous yielding point is observed at a higher stress (point B) and the resulting increase in yield strength ($\Delta\sigma_y$) is appreciable. Also, an increase in ultimate tensile strength ($\Delta\sigma_u$) and, more significantly, a reduction in ductility ($\Delta\varepsilon$) are detected.

The increase in yield strength and reduction in ductility caused by strain ageing is a relevant aspect for RC buildings in earthquake-prone countries. RC structures are designed to avoid brittle failure mechanisms by providing the structures and its elements sufficient ductility to dissipate energy through inelastic cycles during earthquakes. In addition, during the design, it is essential to ensure that plastic hinges form at specific locations such as beam-ends, column bases and coupling beams. This is achieved by accounting for the over-strength of plastic hinges with respect to capacity design and hierarchy of strength principles. An increase in yield strength might result in an increase in the flexural strength of damaged and eventually (epoxy) repaired plastic hinges and might change the hierarchy of strength, possibly leading to a strong-beam/weak-column and a soft-storey mechanisms in the case of major earthquakes (Paulay & Priestley, 1992; Tasai, Otani, & Aoyama, 1988).

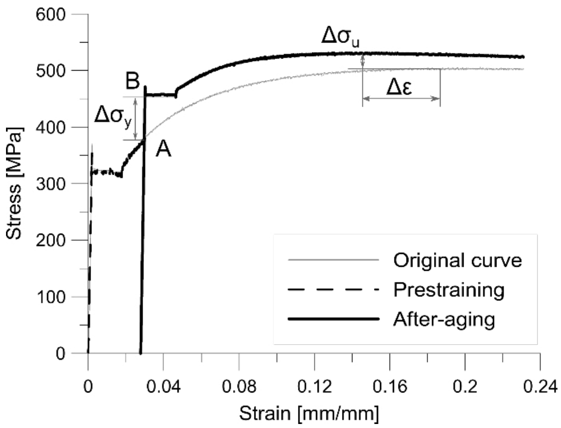

Figure 1 Stress–strain curve of NZ-manufactured Grade 300E subjected to strain ageing

Figure 2 Fractured longitudinal reinforcing bars in a bridge pier close to the Mw 7.8 Kaikoura earthquake epicentre.

1.2. Low-cycle fatigue (LCF)

The cyclic stress produced by repeated loads such as wind, car traffic or earthquakes can cause micro-cracking and cracking, that eventually lead to fracture of the material/element involved. When a structural element is subjected to a small number of repetitions before fracture, the responsible failure mechanism is known as low-cycle fatigue (LCF). During earthquakes, steel reinforcement in RC structures, might be subjected to large-inelastic strain cycles (up to 6%) in tension and compression, eventually leading the rebars to fracture due to low-cycle fatigue (Mander, Panthaki, & Kasalanati, 1994). This mode of failure has been observed in both laboratory testing (El-Bahy, Kunnath, Stone, & Taylor, 1999b; El-Bahy, Kunnath, Stone, & Taylor, 1999a) and post-earthquake damage inspections (see Figure 2) (Palermo et al., 2017). Earthquakes are usually preceded and/or followed by other events of larger or smaller intensity; longitudinal steel failures might not occur during a first event, but in a subsequent one due to the cumulative damage. Seismic events can also occur several months apart and during this period, if the steel has experienced any post-yielding deformation during the first event, strain ageing takes place, modifying the mechanical properties of the material as described in Section 1.1.

Low-cycle fatigue (LCF) problems are analyzed by adopting the strain-based approach. Life estimation is performed using strain-fatigue life curves (Figure 3). Strain and fatigue life are plotted on log-log coordinates with the number of cycles to failure (N_f) or half cycle ($2N_f$) on the x-axis, and the strain amplitude (ε_a) on the y-axis. These curves are obtained from completely reverse ($R = -1$) strain controlled tests in which strain limits are constant (Dowling, 2013). Several models to predict the low-cycle fatigue life of structural components can be found in the literature. The most common is known as the Coffin–Manson relationship. The Coffin–Manson relationship allows the calculation of the number of cycles to failure N_f, for a given ε_a.

$$\varepsilon_a = \frac{\sigma'_f}{E}\left(2N_f\right)^b + \varepsilon_f\left(2N_f\right)^c \tag{1}$$

The materials constants σ_f, b, ε_f, c are material dependent constants obtained experimentally by performing a linear regression analysis.

1.3. Purpose of this paper

Although, the LCF behaviour of rebars has been extensively studied, no previous research exists on the effects of strain ageing on the fatigue life of steel rebars. In this paper, fatigue lives for unaged and aged reinforcing bars are compared. The strain amplitude versus fatigue-life curve of unaged Grade 300E steel is first determined. Then, specimens of the same grade, diameter and steel heat are subjected to a number of (pre-) cycles, aged and cyclically retested to failure. The objective is to determine to what degree the fatigue life of the aged samples changes. The implications of these changes are then discussed.

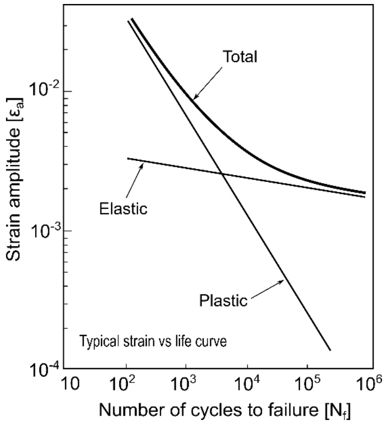

Figure 3 Typical strain versus life curve.

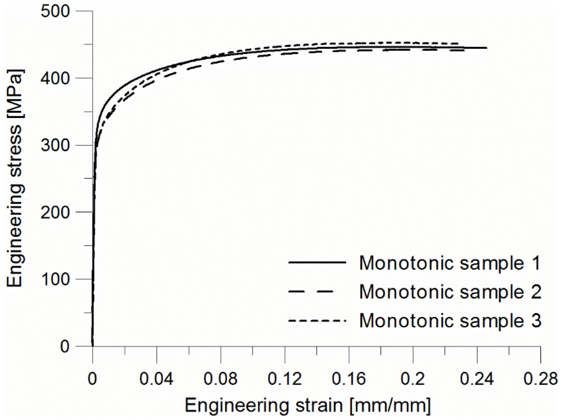

Figure 4 Stress–strain curves of the benchmark samples.

2. LCF experimental testing

2.1. Material and mechanical properties

In this research, locally manufactured Grade 300E (Earthquake ductility) 12-mm diameter reinforcing bars were tested. In order to obtain the fundamental mechanical properties, the rebars were monotonically tested. The stress–strain curve (see Figure 4) showed that the 12-mm diameter rebars do not exhibit the discontinuous yielding point typical of low-carbon steels, thus the yield stress was determined as the 0.2% proof stress. The average yield strength was 314 MPa, the average ultimate tensile strength was 447 MPa, while the average ultimate tensile strain was 0.193 mm/mm. The material met the requirements of the local steel reinforcement standard AS/NZS 4671:2001 (Standards Australia and New Zealand, 2001).

2.2. Methodology

The strain-life curve for undamaged steel was first derived. Steel specimens were subjected to completely reversed cyclic loading (R = -1) between constant-strain limits. Fatigue-life curves were obtained by applying a number of strain amplitude cyclic histories, maintaining the mean strain equal to zero. The fatigue life is determined for each strain limit and plotted in a strain-life diagram on log–log coordinates similar to that in Figure 3. The test set-up was designed with consideration of the laboratory constraints such as the 100 kN load capability of the MTS 810 machine, the geometry of the Vee-wedge devices for gripping the steel samples, the extensometer gage-length dimensions and its travel lengths. The selected bar diameter size was 12 mm.

The objective of the test was to determine the low-cycle fatigue behavior of reinforcing bars used in RC members. Therefore, contrary to the common approach in material testing, the rebars were not machined and buckling was not prevented (ASTM, 2012). An example of the 180-mm long unmachined rebar samples is represented in Figure 5. Consistently with previous works by Mander et al. (1994) and Brown and Kunnath (2004), the unsupported length selected was 72 mm, which is six times the bar diameter. Strain-controlled cyclic tests conducted on unmachined rebars prone to buckling could potentially cause machine instability when the extensometer controlling the tensile machine is directly attached to the testing specimen. Thus, an “indirect method” was used to conduct the strain-controlled fatigue tests. A steel device was rigidly mounted to the top and bottom head of the tensile machine. The device was made by two L-shaped steel elements welded to two smooth steel rods. The extensometer responsible for controlling the machine was attached to the top and bottom rods (see Figure 6). The indirect method required an initial calibration (Dowling, 1977). For this purpose, two extensometers were used: one was attached directly to the steel specimen and another ("external extensometer") was placed on the external steel device. Details of the calibration method can be found in G. Loporcaro (2017). Due to limitations of the extensometer travel lengths in compression, the maximum strain amplitude allowable was 0.03 mm/mm. The wave type selected was sinusoidal. Sample codes, strain limits, and frequencies selected are presented in Table 1. The sample rate was approximately 200 measurements per cycle.

Depending on the amount of strain amplitude applied to the specimen, two methods were used to determine the number of cycles to failure. In the case of those specimens subjected to a strain amplitude above 0.02 mm/mm, the life to failure was determined by counting the cycles when the first crack was visually observed. When the strain amplitude was below 0.02 mm/mm, the normalized stress at reversal (f_i/f_0, where f_i is the stress at reversal the generic i_{th} cycle, and f_0 is the stress at the first reversal) versus number of cycles curves were used as defined by Mander et al., 1994. Failure was determined when the ratio f_i/f_0, (tension) after the cycle-dependent hardening effect reached 1. The fatigue life of the tested specimens is presented in Table 1.

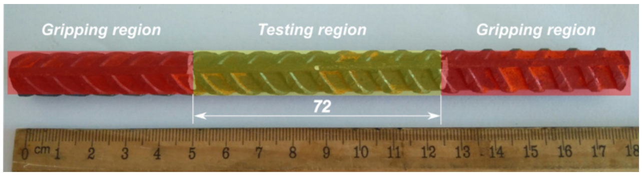

Figure 5 Reinforcing bar specimen used for the experimental testing. Gripping and testing region are specified (dimensions are in mm).

Figure 6 Test set-up employed in order to perform the low-cycle fatigue tests.

Table 1 Low-cycle fatigue tests initial parameters and results

Sample code	Strain amplitude	Frequency (Hz)	Number of cycles to failure
01	0.0078	0.12	125
02	0.0078	0.12	130
03	0.0083	0.12	98
04	0.0107	0.11	61
05	0.0140	0.09	34
06	0.0140	0.09	32
07	0.0178	0.06	14
08	0.0179	0.06	16
09	0.0179	0.06	13
10	0.0271	0.04	6
11	0.0272	0.04	6
12	0.0275	0.04	7

The effects of strain ageing on the fatigue life of the steel rebars were determined by adopting the following protocol:

- Steel specimens were first precycled up to preidentified number of cycles. In a first experiment, the bar specimens were cyclically loaded up to 33% of the fatigue life previously obtained. In a second experiment, the number of precycles was equal to the 66% of the original fatigue life;
- The precycled specimens were "artificially" aged for four hours at 100° C in boiling water. This corresponds to 1 year aging at 15° C as showed by Hundy (1954) and Loporcaro et al. (2016);
- Finally, the specimens were cyclically tested (at the same strain amplitudes) until failure assuring that the same unsupporting length, defined by the white marks (see Figure 5), was maintained.

For practical reasons, six specimens were tested in both first and second experiments.

2.3. Results

Fatigue-life results from the experimental tests for the aged and unaged samples are compared in Table 2. The comparison shows a reduction in fatigue life for each samples that ranges from 15% to 50%. Then, the results obtained were fitted using the Coffin-Mason model as explained in Section 1.2. First, the experimental data obtained by testing the unaged samples are fitted in Equation (2); these results are consistent with those obtained by Mander et al. (1994).

$$\varepsilon_a = 0.0025\left(2N_f\right)^{-0.076} + 0.080\left(2N_f\right)^{-0.464} \tag{2}$$

Then, experimental data for the precycled and aged samples were fitted. Equation (3) and (4) refers to the aged samples precycled to 33% and 66%, respectively.

$$\varepsilon_a = 0.0025\left(2N_f\right)^{-0.076} + 0.067\left(2N_f\right)^{-0.456} \tag{3}$$

$$\varepsilon_a = 0.0025\left(2N_f\right)^{-0.076} + 0.088\left(2N_f\right)^{-0.507} \tag{4}$$

Table 2 Low-cycle fatigue tests initial parameters and results for precyled and aged samples

Strain amplitude	Frequency (Hz)	Number of cycles to failure for aged samples precycle to 33%	Change in fatigue life compared to unaged samples (%)	Number of cycles to failure for aged samples precycle to 66%	Change in fatigue life compared to unaged samples (%)
0.0078	0.12	108	-14	102	−18
0.0083	0.12	69	-30	74	−24
0.0107	0.11	45	-26	48	−21
0.0140	0.09	23	-28	27	−16
0.0179	0.06	8	-50	13	−19
0.0275	0.04	5	-29	6	−14

Strain versus number of cycles curves for aged and unaged samples are plotted in Figure 7 and Figure 8. Equation (3) is plotted in Figure 7, superimposed on the unaged strain-fatigue life curve [Equation (2)]. Both curves are approximately parallel but shifted because the fatigue life of the aged samples is shorter, as also shown in Table 2. Equation (4) is then plotted in Figure 8 and also superimposed on the unaged strain-fatigue life curve. In this last case, the two curves are not parallel: at shorter fatigue lives the curves almost coincide, while at longer fatigue lives, the effect of strain ageing becomes more significant. This is explained because at very short lives, e.g., less than 10, the number of precycles (66% of the original fatigue life) is close to the fatigue life. Only 3 – 4 more cycles are sufficient to reach the original fatigue life. In other words, for strain amplitudes above 2%, the effects of strain ageing could be neglected if the number of cycles experienced is approximately two-thirds of the fatigue life.

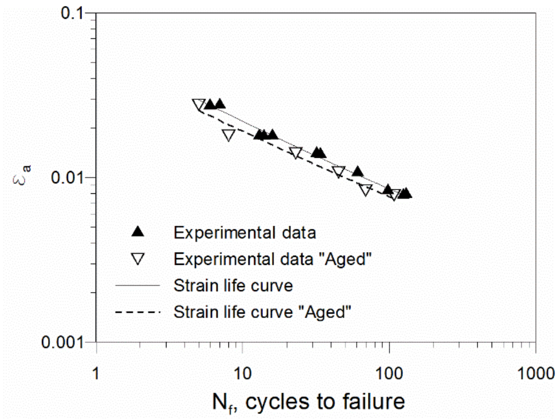

Figure 7 Comparison between unaged and aged samples (33% precycled). Coffin–Manson model using total strain.

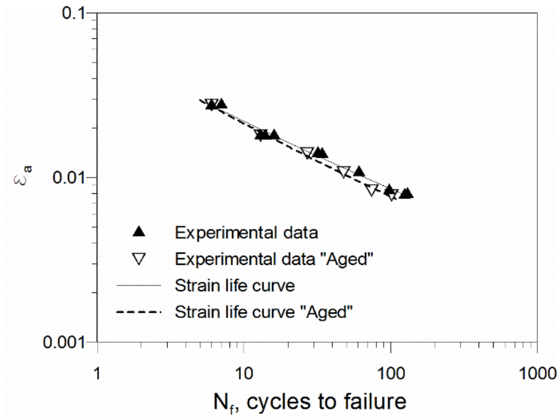

Figure 8 Comparison between unaged and aged samples (66% precycled). Coffin–Manson model using total strain.

A further observation can be made by comparing the expected remaining life (calculated as the difference between the original fatigue life and the precycles applied) with the actual remaining life. A drastic reduction in fatigue life can be observed in the results presented in Table 3. The remaining fatigue-life loss varied from 20% to 70% in the case of those sample precycled up to 33% of the original fatigue life. In the case of samples precycled up to 66%, the reduction in the remaining fatigue life was more dramatic. It ranged from 33% to 73%, with an average loss of about 53%. Therefore, given the same amount of ageing time, the larger the amount of precycling, the more significant is the remaining fatigue-life loss.

Table 3 Remaining LCF life-loss for “aged” samples precyled to 33% and 66% of the initial fatigue life

Strain amplitude	Loss due to strain ageing for samples precycled to 33% [%]	Loss due to strain ageing for samples precycled to 66% [%]
0.0078	20.5	56.1
0.0083	43.9	72.7
0.0107	39.0	61.9
0.0140	42.9	45.5
0.0179	72.7	50.0
0.0275	40.0	33.3

3. Discussion

The results obtained from the experimental work clearly show that, when the assessment of the residual ductility and remaining fatigue life is undertaken, strain ageing must be considered. For example, if strain ageing is ignored during the damage assessment of plastically deformed rebars, the residual ductility might be overestimated (G. Loporcaro, Pampanin, & Kral, 2014), while the new yield strength might be underestimated. Calibration curves developed for the Vickers hardness method described in Loporcaro, Pampanin and Kral (2018) incorporate the changes in mechanical properties due to strain ageing effects. Also, the remaining fatigue life could be underestimated as demonstrated in this paper. Currently, no techniques able to discern the number of cycles experienced by the steel reinforcement during seismic event are known. The Vickers hardness method cannot provide information about the numbers of cycles experienced by the steel reinforcement during a seismic event. This information might be derived from a non-linear time history analysis considering the ground motion to which the structures was subjected and the geometrical and mechanical properties of the RC elements/structure. Moreover, the findings from this research might be used to perform analysis low-cycle fatigue effects on structures such as those conducted by Mander and Rodgers

(2015).However, strain ageing does not affect all steels. NZ Grade 500E is manufactured with additions of 0.08% to 0.10% by mass of vanadium. Beside enhancing the tensile strength of steel, vanadium forms an insoluble nitride that eliminates the effect of strain ageing at temperature below 150°C. Therefore, this is another aspect to be considered during the assessment of earthquake-damaged reinforcing bars.

4. Conclusions

In this paper the changes in mechanical properties of earthquake-damaged steel reinforcement have been discussed. A short introduction to the current method for assessing the plastic deformation of rebars was provided. Finally, the reduction in the LCF life of NZ-manufactured Grade 300E steel 12 mm diameter reinforcing bars was determined. Strain-life fatigue curves were obtained from constant-strain-amplitude cyclic-loading tests. The experimental results were fitted using the Coffin–Manson fatigue models. The experimental results obtained from the unaged specimens were used as a benchmark to calculate the fatigue-life loss due to strain ageing. Experiments demonstrated that strain ageing reduced the expected fatigue life of 12-mm Grade 300E steel rebars. For example, the expected remaining life of rebars precycled at 1% constant-strain amplitude up to 66% of the original fatigue life was reduced by 62%, from 21 to 8 cycles. The modified fatigue-life relationship derived might be used to determine the LCF fatigue effects on structures subjected to earthquake sequences (Mander & Rodgers, 2015).

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