# EFFECTS OF STRAIN-AGEING ON NEW ZEALAND REINFORCING STEEL BARS

**A. Momtahan[1], R.P. Dhakal[2] and A. Rieder[3]**

**SUMMARY**

Modern seismic design codes, which are based on capacity design concepts, allow formation of plastic hinges in specified locations of a structure. This requires reliable estimation of strength of different components so that the desired hierarchy of strength of the structural components can be ensured to guarantee the formation of plastic hinges in the ductile elements. As strength of longitudinal reinforcing bars governs the strength of reinforced concrete members, strain-ageing, which has significant effect on the strength of reinforcing bars, should be given due consideration in capacity design. Strain-ageing can increase the yield strength of reinforcing steel bars and hence the strength of previously formed plastic hinges, thereby likely to force an unfavourable mechanism (such as strong beam-weak column leading to column hinging) to take place in subsequent earthquakes. In this paper, the strain-ageing effect of commonly used New Zealand reinforcing steel bars is experimentally investigated. Common New Zealand steel reinforcing bars are tested for different levels of pre-strain and different time intervals up to 50 days, and the results are discussed focussing on the extent of strain-ageing and its possible implications on seismic design provisions. The results indicate that designers need to use a higher flexural strength (in addition to overstrength) for the weaker member in checking the strength hierarchy in capacity design of reinforced concrete frames. Similarly, in designing retrofit measures to restore a damaged reinforced concrete member engineers need to take into account an increase of yield strength of the reinforcing steel bars employed in the member due to the strain-ageing phenomenon and the extent of increase in the yield strength depends on the level of damage.

[1] *Graduate Engineer, Maunsell|AECOME, Hamilton, NZ*

[2] *Associate Professor, Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, NZ (Member).*

[3] *Research fellow, BOKU University, Vienna, Austria*

## INTRODUCTION

Uni-axial stress-strain behaviour of steel has been investigated over the past century. However, limited research has been carried out on the effects of strain-ageing on the mechanical behaviour of reinforcing steel under monotonic and cyclic conditions.

The behaviour of steel is significantly affected by its chemical composition and the method of manufacture. Bauschinger (1887) observed an increase in the maximum load-carrying capacity of a piece of steel after it was tested in the inelastic range, left for some time and then retested. This observation is now known as *strain-ageing*.

Physical basis of this phenomenon can be interpreted by time dependent diffusion of small elements such as carbon and nitrogen to crystal flaws (e.g. dislocations). Plastic deformation (yielding) is triggered by movement of dislocations through the crystal, whereas the presence of small atoms (e.g. carbon and nitrogen) can markedly increase the sliding resistance between the atoms and hence increase the yield strength.

Over the last 50 years a vast range of research has shown that all structural steels are potentially susceptible to strain-ageing. Strain-ageing is known to cause an increase in yield strength and ultimate tensile strength of reinforcing steel bars, while tensile elongation capacity will be less in strain-aged steel bars.

Strain-ageing was first observed in steels that were rolled and annealed. After being stored for weeks or months, during which time the interstitial atoms migrated to the dislocations, the yield point increased significantly and the ductility decreased (as schematically illustrated in Figure 1); the material appeared to have "aged". Structural steels that were cold formed during fabrication by bending or rolling are reported to show increased strength and decreased ductility and toughness (Pussegoda, 1978).

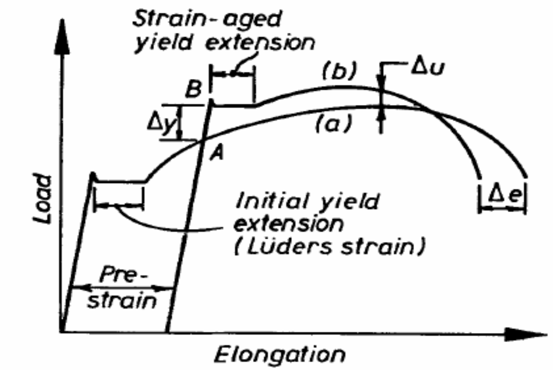



***Figure 1: Schematic illustration of strain-ageing effect on stress-strain behaviour of reinforcing steel bars.***

The mechanisms associated with strain-ageing were first, rationalized by Cottrell and Bilby (1949). They explained how carbon and nitrogen atoms diffuse through the network of the pre-strained steel toward the position of the free dislocations, pinning them as before yielding occurred. The effect of strain-ageing in reinforcing steel has been extensively studied using monotonic tests by Erasmus and Pussegoda (1977) and Erasmus (1981, 1987). They discussed the effects of strain-ageing on the failure mode of bent reinforcing bars.

A series of cyclic and uni-axial tests were conducted at the University of Canterbury on the two types of reinforcing steel which were commercially available in New Zealand in the late 80s and early 90s (Posada 1992). In those tests the effects of cyclic loading, bar deformation, strain rate and strain-ageing on the stress-strain behaviour of reinforcing steel were investigated. Typical monotonic and cyclic test results from that test series are shown in Figure 2. Posada (1992) reported that Grade 300 steel was susceptible to strain-ageing and most of the strain-ageing occurred in the first 37 days. Also, the Bauschinger effect gradually vanished and the initial elastic behaviour reappeared instead, showing a new upper and lower yield point. The results showed that the increase in strength does not cause an increase in stiffness. In the tests a new yield plateau reappeared at stress level of 1.25 times the yield stress of the virgin material. In addition, it was observed that the stress-strain history only influenced the magnitude of the effects of strain-ageing on the yield strength. For example, any increase in strength in an aged reinforcing steel bar was coupled with the increase in strength caused by strain hardening.

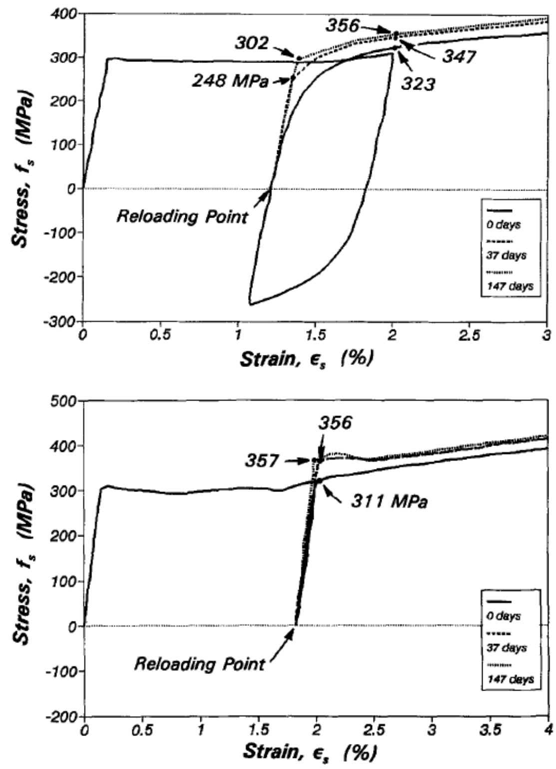



***Figure 2: Cyclic and monotonic stress-strain behaviour (Posada 1992).***

## EFFECTS OF STRAIN-AGEING ON STRUCTURAL MEMBERS

It is essential to ensure that in the case of a severe earthquake, a brittle failure does not occur in structural members. This can be assured only if the structure has sufficient ductility to absorb and dissipate energy by inelastic deformations when several cycles of lateral displacement are applied to the structure. The ductility of a structure is ensured by development of plastic hinges in certain regions where inelastic deformations occur. In reinforced concrete frames, these hinges are to form in the beams adjacent to beam-column joints rather than in the columns. More energy can be dissipated if plastic hinges form in beams because yielding is then spread throughout the frame height and the curvature ductility demands on individual plastic hinges are not so great. Therefore, the emphasis at present is for the design of moment resisting frames with strong columns and weak beams.

Reinforcing steel plays an important role in earthquake resistant design of reinforced concrete structures. The amount, strength and arrangement of longitudinal steel in a member of a structure determines the strength and ductility of the member while, transverse steel provides shear strength, restrains longitudinal compression steel from buckling and also contributes towards the ductility of the section by confining the core concrete. Therefore, for the survival of a reinforced concrete structure during an earthquake, it is essential to completely eliminate brittle fracture in both longitudinal and transverse reinforcing steels in addition to ensuring that plastic hinges form in the beam adjacent to beam-column joints.

Pussegoda (1978) reported that strain-ageing in longitudinal reinforcing bars will occur at plastic hinges formed by earthquake. This will result in an increase in the flexural strength of the plastic hinges due to the increase in yield strength of the steel during the ageing process. Therefore, plastic hinges may be formed during subsequent earthquakes in regions that have not been designed as such, thereby causing a brittle failure. For example, the development of plastic hinges in regions that do not have sufficient transverse reinforcement can cause shear failure.

As bending moment is higher in plastic hinges than in other parts of reinforced concrete members, the longitudinal strain level in reinforcing bars is greater in the plastic hinge region where spalling of concrete around the bars is observed. In this case, the combined effects of strain-ageing and strain hardening will increase the flexural capacity of the member, which results in migration of the plastic hinge to elsewhere in the member or even into the adjacent member.

To ensure the formation of the plastic hinge in the beam adjacent to the beam-column joint during cyclic loading in a reinforced concrete frame, an overstrength factor is used for the intended plastic hinge region. The overstrength factor usually varies between 1.2 and 1.3. In fact, it is common in New Zealand to allow for an approximately maximum likely yield stress 25% higher than the specified value when considering the effects of over-strength in design (Posada 1992). Strain-ageing of the flexural steel at the plastic hinge subsequent to a formidable seismic loading may significantly increase the yield strength of the steel. The extent of this increase in the yield strength of reinforcing bars due to strain-ageing observed in previous experimental studies can account for about a 25% increase in the flexural strength of the reinforced concrete section. Hence, the overstrength factor used alone may not be sufficient when consideration is given to the possibility of strain-ageing in flexural steel. However, increasing the overstrength factor to accommodate the ageing effect will further increase the reinforcement in the column and the joint, both of which need to be stronger than the overstrength of the beam. For example if the yield strength of the steel in the beam exceeds the specified value, the section moment capacity will be higher than the predicted one. This increase in moment capacity will increase the shear force acting on the member and may result in brittle shear failure due to diagonal tension cracking of the concrete or an anchorage failure occurring rather than a ductile flexural failure. Therefore, the effects of strain-ageing may also alter the transverse reinforcement required in both the column and the beam to prevent the possibility of brittle shear failure.

In addition, an increase in the flexural capacity of the beam will increase the bending moments acting on the column and may cause plastic hinges to form in the column rather than the beams. This may result in a brittle column sideway mechanism rather than a ductile beam sideway mechanism. Therefore, stronger longitudinal steel may lead to brittle forms of collapse rather than ductile energy dissipating behaviour.

Pussegoda (1978) reported that strain-aged regions such as bends in reinforcing bars, returns, hooks and stirrups are susceptible to brittle failure which may cause catastrophic collapse of the structure. Previous investigations have shown that failures can occur at bends by cleavage fracture resulting from strain-aged embrittlement associated with the stress concentration effect. This fact must be considered by structural engineers so that both in design and detailing of reinforcement consideration is given to the effects of strain-ageing embrittlement of bent reinforcing bars in order to eliminate such failures.

## OBJECTIVE AND SCOPE

In this study, emphasis is given to investigate the stress-strain behaviour of Grade300 reinforcing steel currently available in New Zealand and to determine the amount of time dependent increase in its strength due to strain-ageing. Therefore, a series of monotonic tensile tests were carried out in the structural laboratory of the University of Canterbury.

In performance based earthquake engineering (PBEE), the performance level of a structure after an earthquake is generally classified into 4 different categories based on the extent and level of damage.

- *Operational* (structure has very little damage and is fully functional; no need of repair)
- *Immediate Occupancy* (it is safe to occupy but minor repairs might be needed)
- *Life Safety* (moderate damage but the structure remains stable with significant reserve capacity; substantial repair/retrofit needed before it could be occupied)
- *Collapse Prevention* (structure remains standing but only barely; beyond repair)

This classification comes in handy when defining the rehabilitation and retrofitting objectives. As indicated above, in the operational level of performance no damage is expected. Therefore, the reinforcing bars are expected to be elastic, and hence strain-ageing does not occur in subsequent earthquakes.

If a reinforced concrete structure undergoes ductile response inducing some damage in an earthquake (as in the *immediate occupancy* level of seismic performance), plastic hinges are likely to form and the reinforcing bars in these hinges yield but the plastic strain is small. In such a case, strain-ageing might happen and the strength of reinforcing bars may be higher when a subsequent earthquake strikes.

If a major earthquake induces severe damage (as in the *collapse prevention* level), the plastic strain in the reinforcing bars is substantially large but the chance of the structure being repaired/retrofitted is very slim. In such cases, it is very likely that the severely damaged structure will be demolished as any effort to regain the functionality of the structure is uneconomical. Nevertheless, if the damage is moderate as in the *life-safety* level of seismic performance, several plastic hinges may form but the plastic strain in reinforcing bars is limited. As repair may be economically viable in such cases, strain-ageing will increase the strength of the reinforcing bars (and also of the plastic hinges) when another earthquake next strikes.

Hence, the effects of strain-ageing on structural post-earthquake performance is a major concern only when the plastic strain level is moderate; say $10–15\varepsilon_y$,, where $\varepsilon_y$ is the yield strain of the reinforcing steel bar. Beyond this level of pre-strain, repair is not economically viable, and strain-ageing of reinforcing bars at large strain (although significant) becomes practically irrelevant. Therefore, the pre-strain levels in this study were decided to be $2\varepsilon_y$, $5\varepsilon_y$, $10\varepsilon_y$ Additional tests with pre-strain level of $15\varepsilon_y$ were carried out in order to assure the effects of large pre-strain level within the range of practical interest on the strain-ageing phenomenon.

The specimens were pre-strained to the different strain levels and stored for 3, 7, 15, 30 and 50 days before being retested. Although earthquakes occur in the space of years where strain-ageing may be significantly more pronounced, the tests were conducted within a space of days. However, the results obtained from these tests are likely to provide a clearer understanding of the effect of strain-ageing on the yield strength of reinforcing bars. Also, a trend between the extent of yield strength increase and time may be obtained, which could be extrapolated to estimate the longer term effect of strain-ageing.

## METHOD OF THE TEST

To precisely observe the effects of strain-ageing on the stress-strain behaviour of reinforcing steel bars, both monotonic and cyclic tests are required to be carried out for different strain levels. Nevertheless, only monotonic behaviour has been investigated in this study because it has been shown by Posada (1992) that there is no significant difference in the extent of yield strength enhancement due to strain-ageing between the samples tested monotonically and cyclically.

**Table 1. Schedule of test specimens for different combinations of pre-strain level and interval.**

| Level of pre-strain | Interval (day) | Number of Sample |
|---|---|---|
| Benchmark | 0 | 2 |
| $2\varepsilon_Y$ | 3 | 3 |
| | 7 | 3 |
| | 15 | 3 |
| | 30 | 3 |
| | 50 | 3 |
| $5\varepsilon_Y$ | 3 | 3 |
| | 7 | 3 |
| | 15 | 3 |
| | 30 | 3 |
| | 50 | 3 |
| $10\varepsilon_Y$ | 3 | 3 |
| | 7 | 3 |
| | 15 | 3 |
| | 30 | 3 |
| | 50 | 3 |
| $15\varepsilon_Y$ | 30 | 3 |
| | 50 | 3 |
| **Total** | | **53** |

*Figure 3: Test set up.*

The test specimens are of Grade 300 steel manufactured in New Zealand by the PACIFIC STEEL Company. As mentioned earlier, the specimens were tested under monotonic tension to different strain levels and stored for different times (up to 50 days) before being retested in axial tension. As listed in Table 1, 53 specimens were tested. While the tensile test is independent of size and length of the steel bars, the test coupons were made from 16 mm diameter reinforcing steel bars due to the capacity limit of the tensile test machine available. The length of the specimens was chosen to be 500mm including extra length for the machine's grips. To keep any differences in mechanical characteristic of the tested reinforcing steel bars to a minimum, emphasis was placed on choosing the bars from the same batch delivered to the laboratory.

For the tensile tests, a standard pressure controlled testing machine with a nominal capacity of 500 kN and a calibrated extensometer with a gauge length of 50 mm were used to capture the applied load and the elongation of the bars. The test specimens were placed with minimum eccentricity within the grips and held by a small preload. The extensometer was then attached approximately in the middle of the specimen in order to capture the actual strain of reinforced steel bars. Data acquisition was performed through LabVIEW at a sampling rate of 3 Hz. Figure 3 shows a specimen with an extensometer before the test.

## TEST RESULTS

The monotonic tensile test of two specimens was first carried out to a large strain up to 0.2, and an average stress-strain behaviour was generated in order to provide a benchmark for the successive tests. The yield strength was then determined from the obtained data. Figure 4 shows the early part of the

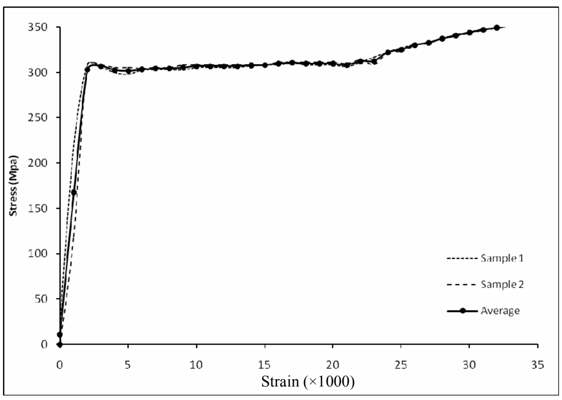



*Figure 4: Stress-strain curves of the two benchmark specimens and the average stress-strain curve.*

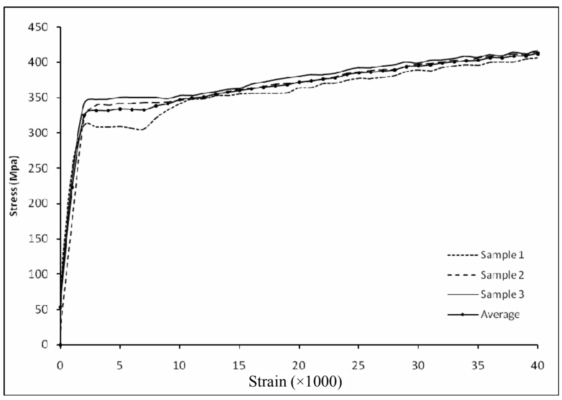
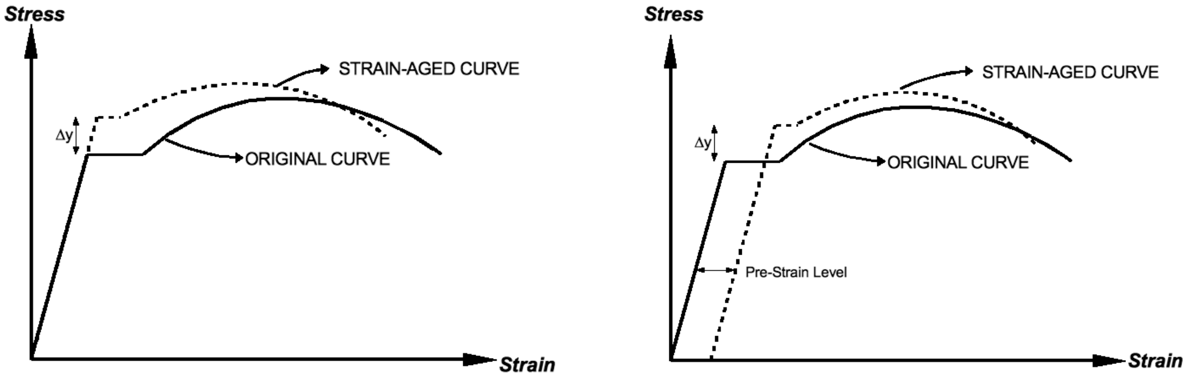



*Figure 5: Individual and average stress-strain curves for specimens pre-strained to $10\varepsilon_y$ tested after 50 days.*




**a. Ignoring residual strain** **b. Considering residual strain**

***Figure 6: Comparison of stress strain curves of original and strain-aged bars.***

stress-strain curves obtained from the two specimens and the average stress-strain curve to be used as the benchmark.

The average curve and the yield strength for the specimens with the pre-strain levels of $2\varepsilon_y$, $5\varepsilon_y$, $10\varepsilon_y$ and $15\varepsilon_y$ after 3, 7, 15, 30 and 50 days were also determined from the test results. Figure 5, for instance, illustrates the early part of the stress-strain behaviour of the three specimens with pre-strain level of $10\varepsilon_y$ tested after 50 days and the average stress-strain behaviour.

To be able to compare the strain-aged yield strength with the original yield strength, the residual plastic strain of the pre-strained bars is not accounted for when the stress-strain behaviour of the reloaded steel bars is presented. In other words, as shown in Figure 6a the strain is readjusted to zero when the second phase of the test is conducted after storing the specimen for a number of days. Note that this is different from previous practice (Posada 1992) where the residual strain is also shown while presenting the stress-strain behaviour of stain-aged reinforcing steel bars (Figure 6b). This simplification is justified because it will make no significant difference to the interpreted overall deformability of the reinforcing bars as the range of pre-strain investigated in this study is very small in comparison to the ultimate strain of New Zealand's Grade 300 reinforcing steel bars.

From the average stress-strain curves, the yield stress was determined and then compared to the average yield stress of the original (un-aged) bars in order to obtain a ratio of the strain-aged yield strength over the original yield strength. To take into account the record-to-record randomness of yield

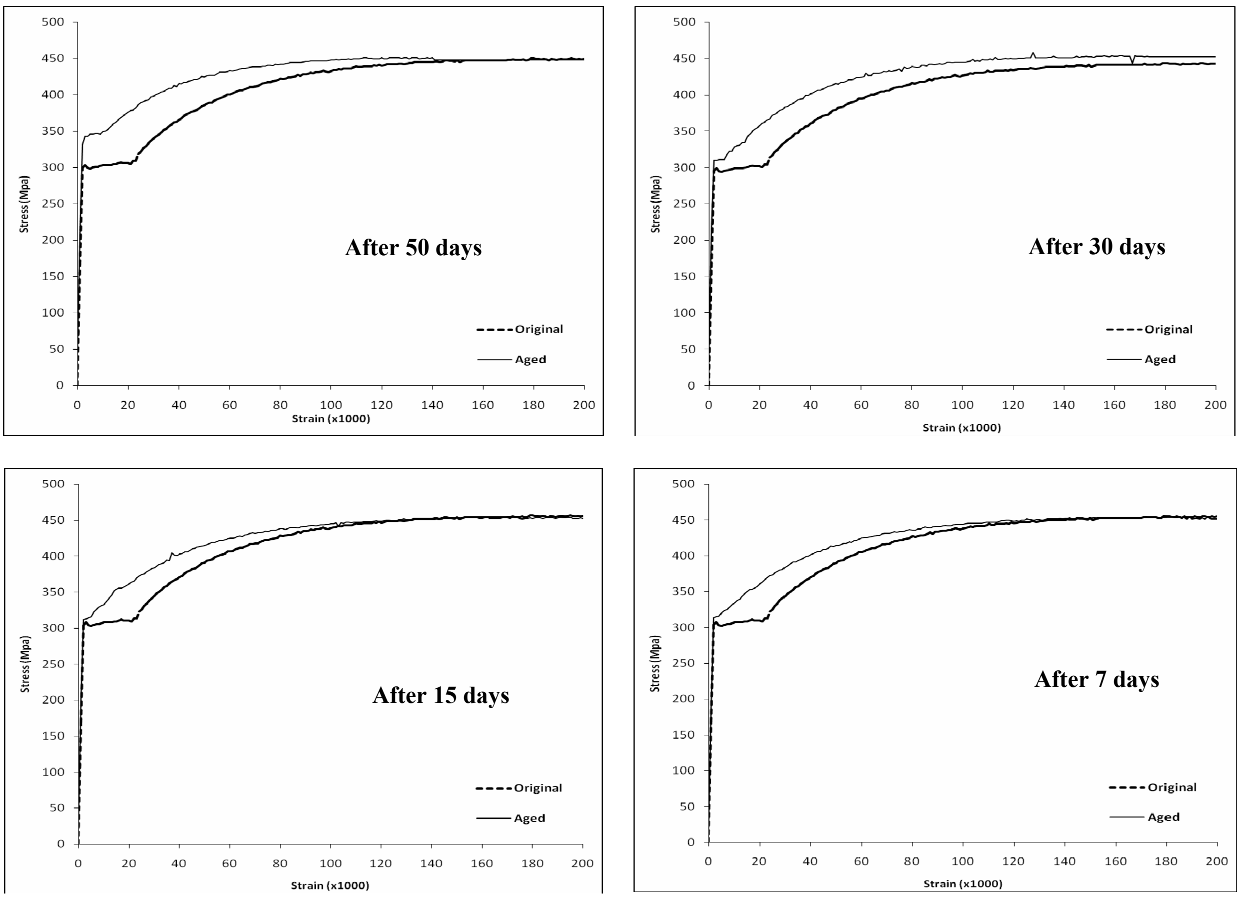
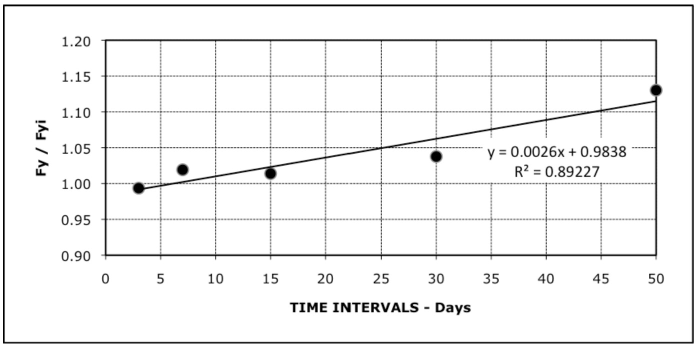



***Figure 7: Average stress-strain curves of strain-aged samples tested with a pre-strain of $10\varepsilon_y$.***

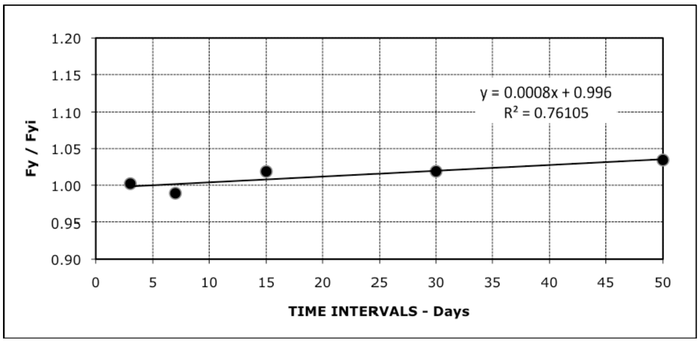
strength in normal reinforcing steel (which is approximately 10%) and to evaluate the ultimate strength and the amount of embrittlement in the strain-aged steel bars, it is essential to have the complete original stress-strain behaviour for each specimen. The complete original stress-strain curve can be obtained by multiplying the benchmark curve by a correction factor, which is equal to the ratio between the yield strength of the pre-strained bars and the average benchmark yield strength.

In general, the ultimate strength for both strain-aged and original materials remained approximately the same for all specimens in each pre-strain category. No noticeable brittleness was observed in the strain-aged specimens. The yield strength of the strain-aged specimens increased with the time interval; and for the first few days (up to 15 days), the increase in yield strength due to strain-ageing was small and significant strength enhancement was observed for bars retested after 30 and 50 days. As expected, the effect of strain-ageing was more pronounced in specimens with larger pre-strain.

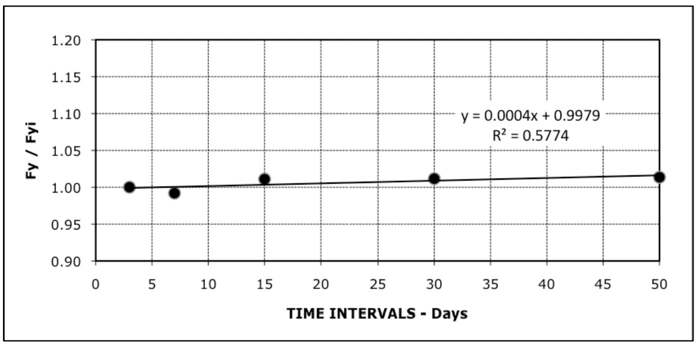
The plots in Figure 7 show the average stress-strain curves for specimens pre-strained to $10\varepsilon_y$ tested after different time intervals along with the corresponding original average stress-strain curves for the respective pre-strain level. Although not shown in the figure, the stress-strain curve obtained from the tensile test of the specimen tested after 3 days is very similar to that from the one tested after 7 days. It can be observed from the curves that the strain-ageing effect on the yield strength of the tested specimens was not significant during the first 15 days. However, the specimens showed an increase in yield strength by about 13% after 50 days. Moreover, the yield plateau seen in the original specimen does not exist in the stress-strain curves of specimens tested within 15 days and it reappeared to some extent in specimens tested after 30 and 50 days. Note that inherent yield plateau in the original Grade 300 was found to exist until the strain reached slightly more than $10\varepsilon_y$ and the specimens were about to enter strain hardening range when they were pre-loaded to $10\varepsilon_y$. When they were reloaded after a short time (up to 15 days), the specimens continued along the strain hardening path immediately after yielding, whereas the specimens probably




**Pre-strain level of $10\varepsilon_y$**

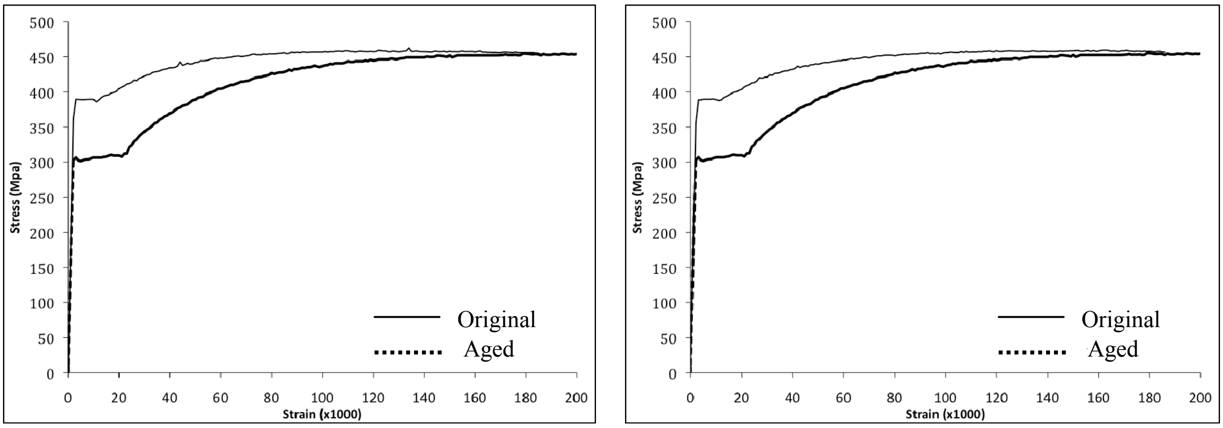



**Pre-strain level of $5\varepsilon_y$**




**Pre-strain level of $2\varepsilon_y$**

***Figure 8: Variation of yield strength with time for different levels of pre-strain.***




**After 30 days** **After 50 days**

***Figure 9: Average stress-strain curves of strain-aged samples tested with a pre-strain of 15$\varepsilon_y$.***

acquired a new equilibrium between 15 and 30 days and behaved like a new bar by showing a yield plateau.

Figure 8 illustrates the level of strain-ageing in the tested Grade 300 reinforcing steel bars after 3, 7, 15, 30 and 50 days for three different pre-strain levels (2$\varepsilon_y$, 5$\varepsilon_y$ and 10$\varepsilon_y$). The test results show that no significant strain-ageing was observed for the pre-strain level of 2$\varepsilon_y$. This can be attributed to the low pre-strain level. The results also show no remarkable strain-ageing effect in the tested specimens with the pre-strain level of 5$\varepsilon_y$ and 10$\varepsilon_y$ during the first 15 days, after which the effect was more pronounced; especially in the specimens with the pre-strain level of 10$\varepsilon_y$. Larger pre-strain means a higher density of dislocations and therefore the interstitial atoms have to migrate a smaller distance to the new free dislocations resulting in more pronounced strain-ageing.

Although the level of strain-ageing is more likely to have a non-linear trend with respect to time, the best-fitted lines in Figure 8 represent the early part of this trend. The slopes of the best fitted-lines are, however, indicative of the significance of pre-strain level in the strain-ageing process. It may be agreed, based on these results that it is more likely to get a more pronounced strain-ageing effect if the reinforcing bars are subjected to a larger initial strain.

As mentioned earlier, for all pre-strain levels, there is no significant strain-ageing observed for the first 15 days. For further investigation, additional tests were performed with a pre-strain level of 15$\varepsilon_y$ which is already in the strain hardening region of the original specimen; hence the stress before unloading was slightly (within 5%) higher than the yield strength. These specimens were tested after 30 and 50 days only to obtain the upper bound values of yield strength enhancement due to strain-ageing. The results of this test series are presented in Figure 9, which shows that the strain-ageing effect observed in the specimens tested after 30 and 50 days was similar, and the amount of increase in yield strength in this test series was double of that in the 10$\varepsilon_y$ series specimens after 50 days. It should be noted that at this high pre-strain level the density of dislocations is high enough to enable interference and mutual blocking of dislocations without the contribution of migrating atoms.

## DISCUSSION

It can be concluded that the chemical composition has a fundamental impact on the strain-ageing behaviour. Carbon is identified as an interstitial atom that migrates to crystal flaws (e.g. dislocations), which results in pinning of dislocations. Since yield strength is the critical shear stress which causes movement of dislocations, diffusion of carbon atoms to dislocations increases the yield strength. The fact that the investigated material has a lower carbon content (0.17%) compared to the material used by Posada (1992) explains why the diffusion process needs more time than 37 days in order to achieve an adequate carbon concentration at dislocations to enable a noticeable level of interaction. Another decelerating factor of the strain-ageing process could be the temperature during storage of the test coupons. But due to the small variation of temperature in a real structure, the dependency of the diffusion process on temperature might be neglected. The specimens were stored at 10$^o$C to simulate an average temperature that a structure is exposed to during the year.

Pussegoda's test results (1978) suggested that reinforcing steel bars that have vanadium content between 0.018% – 0.06% in their chemical composition are significantly affected by strain-ageing even at moderate levels of pre-straining and its effect should be carefully evaluated in members reinforced with such bars. The chemical properties of steel bars manufactured by Pacific Steel Company tested in this study had a vanadium content of approximately 0.003%. Yet, the impact of strain-ageing on the behaviour of these bars was significant; especially at pre-strain levels in excess of 10$\varepsilon_y$. Hence, it appears from these test results that the effect of strain-ageing may be significant in reinforcing bars regardless of the amount of vanadium content.

Due to the limit of the extensometer used for the tests the strain could not be increased beyond 0.2. Therefore, the test specimens could not be loaded until rupture, but the tested strain range easily covered the range of strain reinforcing bars are likely to sustain during seismic excitations. Hence, based on the test results it can be assumed that within practical strain ranges (e.g. at the notch roots in the inner surface radii of bends) there is no considerable embrittlement of the material.

As explained earlier, the level of residual plastic strain in reinforcing bars for which repairing the damage is economically viable is limited. Hence, although it is obvious from these tests that the increase in yield strength of main steel bars (and consequently the increase in flexural strength of reinforced concrete members) is significantly higher for larger pre-strain levels, designers do not need to look beyond the pre-strain range used in these tests. Nevertheless, major earthquakes do occur once in several years and the 50 days time interval used in these tests does not give a full picture of the actual strain-ageing effect the reinforcing bars would have gone through between two successive earthquakes. The results have indicated that as the separation time between the two plastic loadings extends, the effect of strain-ageing becomes more pronounced and the yield strength of reinforcing bars becomes larger. Nevertheless, test results corresponding to more realistic time intervals (in years) are not easily obtainable. Hence, these interval test results need to be used to generate qualitative guidelines for designers. It is hence suggested that as an upper bound value approximately a 15% increase in flexural strength of damaged reinforced concrete members could be allocated to strain-ageing.

## CONCLUSION

This work shows that the strain-ageing phenomenon does exist for the investigated reinforcing steel bars which are commonly used in New Zealand. The effect is most pronounced when steel bars are subjected to a high residual strain and then again exposed to a plastic loading after a long time. In contrast, reinforcing bars with low residual strain subjected to inelastic action after a short time (up to a few days) are not influenced much by strain-ageing. Since the modern capacity design approach allows reasonable damage to occur in moderate earthquakes and the design working life of a seismically retrofitted reinforced concrete members will be much longer than the 50 days used in this study, strain-ageing could easily account for a significant increase in the yield strength of reinforcing bars used.

As observed in the tests, the increase in yield strength after 50 days was 13% in the bars pre-strained to $10\varepsilon_y$ and 25% in the bars pre-strained to $15\varepsilon_y$. Although larger residual strain in reinforcing bars is likely to result in much larger amplification of yield strength, such large strains will correspond to severe damage after which repair of structures is probably economically and structurally unreasonable. Hence, for adoption in seismic retrofitting measures it can be concluded, based on the test results, that the yield strength of the existing main reinforcing bars in damaged members be increased by 15% while designing retrofit measures for a reinforced concrete structure after an earthquake. Nevertheless, more experimental results (especially from long-term tests) are required to quantify the strain-ageing effect more precisely.

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