Experimental validation of “the hardness method” to estimate the residual ductility of plastically deformed steel reinforcement

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ABSTRACT: There is a perceived strong demand from local authorities and industry to develop techniques for assessing damage to steel reinforcement bars embedded in structural concrete elements. However, immediately following the 2010/2011 Christchurch earthquakes, low invasive techniques able to quantify the level and extent of plastic deformation and residual plastic capacity were not available. Although some studies had been conducted in the recent years to answer these questions, a validated method has still not yet been widely accepted.

In the present work, a damage assessment methodology is proposed, based on empirical relationships between hardness versus strain and residual ductility. Experience shows that the portable Leeb hardness test provides an initial, low invasive screening of damaged bars. If damage is indicated in situ, a bar may be removed and more accurate hardness measurements can be obtained using the lab-based Vickers hardness methodology. The Vickers hardness profile of damaged bars is then compared with calibration curves (Vickers hardness versus strain and residual ductility) previously developed for similar steel reinforcement bars extracted from undamaged locations. The proposed methodology also incorporates the effects of strain ageing, which should not be ignored.

The paper presents the recent findings of an extensive campaign of experimental tests conducted to estimate strain and residual ductility of damaged locations in individual bars in earthquake damaged buildings. Note also that this testing is entirely monotonic (not cyclic).

1 INTRODUCTION

The design philosophy at the base of modern seismic codes allows, in case of major earthquakes, the damage of structures. At ultimate limit state structures are designed to prevent collapse during seismic events that have 10% of probability of exceedance in 50 years (500 year return period). The corresponding seismic forces cannot be resisted elastically by structures; therefore designers are allowed to reduce these forces based on the overall ductility and redundancy of the entire structural system. As result, structures are expected to survive to the design-level earthquake at cost of permanent damage consequence of the large inelastic deformation and energy dissipation of the structural materials.

Based on the capacity design and the hierarchy of strengths philosophy (Paulay and Priestley 1992) designers are encouraged to strategically locate and detail sacrificial structural member regions, termed plastic hinges, which will dissipate energy during high intensity earthquakes. In reinforced concrete (RC) structures the role of dissipating the energy originated from the earthquake is assigned to the steel reinforcement.

During the 2011 February earthquake, RC buildings in the Christchurch CBD were subjected to high levels of seismic acceleration. These accelerations were well beyond those predicted for a 500-year return period spectrum. As result, many RC structural elements experienced large inelastic deformation and, as expected, plastic hinges formed in the designed critical locations: beams, coupling beams and at the base of columns and walls (Pampanin 2012).

The capacity design philosophy was conceived to preserve life safety in case of major earthquakes. Buildings permanently damaged were expected to be demolished (Paulay and Priestley 1992). Therefore, after the 2011 Christchurch event, also because of the lack of knowledge regarding assessment and retrofitting technique of plastic hinges many RC buildings were deemed unrepairable and thus demolished.

During the assessment stage local governments, insurances and private engineering practises required information regarding the reparability of buildings. More specifically, in case of RC structures, information was needed about the damage state of the steel reinforcement. Many questions could not yet be answered, such as “Have the steel bars yielded in correspondence to the concrete cracks?” “How much plastic deformation has the steel bars undergone?” “What is the residual ductility of the damaged bars?”

Relatively little work has been done to develop techniques able to answer those questions, e.g. (Matsumoto 2009). The only viable current method (known as “in situ hardness method”) is based on measuring hardness with a Leeb hardness portable device in situ then correlating it to plastic strain based upon laboratory tensile tests. Extensive studies and practical applications have been conducted soon after the Christchurch earthquakes, but the in situ method has not yet been vetted in the open literature, and thus has not been widely accepted. A brief literature review and past applications are provided elsewhere (Loporcaro, Pampanin et al. 2014).

The in situ procedure is structured in several steps: a) structural engineers choose the steel bars on which the test will be conducted; b) the selected bars are exposed for a length of about 500 mm; c) the bar surface is prepared to conduct the hardness test; d) testing locations are identified along the longitudinal section of the bar at the distance of 15 mm; e) six indentations tests are conducted per each hardness location, the average is used to define the longitudinal hardness profile. The hardness profile is then interpreted using laboratory calibration of the hardness vs. stress/strain relationship of undamaged material from the same building. Although pragmatic and relatively low invasive, the Leeb method has relatively large standard deviation in hardness measurements, and has limited dimensional resolution.

The purpose of the Vickers hardness method presented in this paper is to refine the in situ hardness method, and also has the novelty to introduce a new aspect that has not been discussed in previous works: strain ageing. This phenomenon affects many steel types such as NZ Grade 300. The result of strain ageing is a change of mechanical properties (yield stress, tensile strength, elongation and hardness). More information regarding this phenomenon can be found in the open literature (Hall 1951) (Baird 1971) (Erasmus and Pussegoda 1977) (Paulay and Priestley 1992).

2 METHODOLOGY AND APPLICATION TO EARTHQUAKE DAMAGED BUILDINGS

Four Christchurch CBD buildings of interest were identified as having been damaged in the 2010/2011 earthquakes. It was desired to determine the extent of damage to reinforcing bars in cracked concrete in these buildings. The methodology is illustrated in this paper primarily by example of one building referred to as 'Building A'. The overall methodology of the Vickers hardness method is shown in a flowchart in Figure 13.

2.1 Phase I - In situ Leeb hardness testing

In Phase I, structural engineers identified 10 locations in each building as having evidence of damage, i.e. cracks in reinforced concrete structural members (see Fig. 1). First, the location of a specific reinforcing bar crossing a crack was determined using electro-magnetic methods. The cover concrete was then removed to expose the steel bar, using mechanical equipment, taking care to prevent further damage to the steel. The steel was typically exposed over a length of approximately 300-500 mm (see Fig. 2). The exposed steel was surface-finished to approximately 120 grit, taking care not to heat the sample, to a level deep enough to remove the bar deformations and create a flat surface approximately 10 mm across the width of the bar. The bar was then marked into a series of 15 mm increments. Within each increment, an average Leeb hardness was obtained from 6 individual measurements taking care that no measurement was taken closer than 2mm from any other measurement. A first screening of the damaged bars was performed on site using a Proceq Equotip 3 Leeb hardness tester (Figure 3). A typical Leeb hardness profile from Building A is shown in Figure 4.

Figure 1. Cracks in a reinforced concrete structural wall.

Figure 2. Steel reinforcing bar exposed for and prepared for Leeb hardness testing.

Figure 3. Leeb hardness testing.

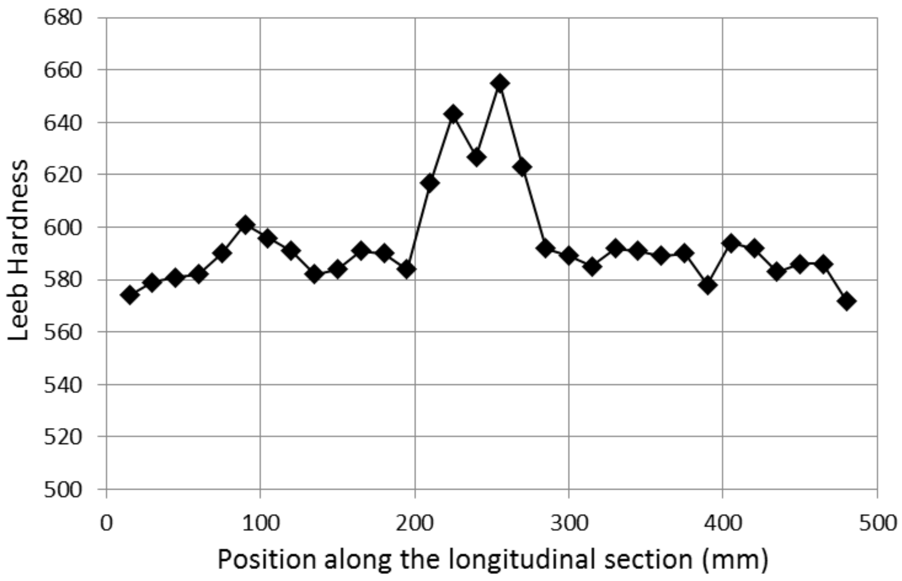

Figure 4. Typical Leeb hardness profile of a damaged steel reinforcing bar from Building A.

2.2 Phase II - Calibration Testing

The fundamental basis of the "Vickers hardness" method is to develop empirical calibration curves for hardness versus strain and hardness versus residual ductility. These are obtained carrying out interrupted monotonic tensile tests and measuring the hardness at each interruption before and after ageing. For each building, undamaged lengths of reinforcing bar were recovered, taking care to obtain the same grade and diameter as the bars in damaged locations.

Twenty-one undamaged specimens were machined to a "dog-bone" shape with a gauge length of 50 mm from the un-damaged steel bars (Figure 5). Geometry and samples dimensions were defined based on the ASTM Standard E8/E8M – 11$^{\varepsilon 1}$ (ASTM 2011a) .The surface of the samples was then sequentially ground using silicon carbide grinding papers up to 600 grit to remove residuals of the machining preparation, to facilitate the hardness reading and reduce the errors due to the optical measurement of the indentations.

Figure 5. "Dog-bone" steel specimen samples.

For each building, three samples were used to obtain the unstrained, unaged baseline mechanical properties: stress – strain curve, upper yield stress (YS), ultimate tensile stress (UTS) and strain at UTS. Five pre-strain limits of 0.01, 0.02, 0.03, 0.04, 0.05 mm/mm were selected with three samples for each pre-strain level). A further calibration point at 0.10 mm/mm strain was performed to confirm that the hardness vs strain calibration could be extrapolated beyond 0.05 mm/mm if required.

Tensile tests were all performed with a MTS 810 with 100kN load capacity using an MTS 25 mm gauge length extensometer capable of 12.5 mm travel in tension. Baseline tensile properties are presented in Table 1 and see Figure 6. In addition, ten baseline hardness were performed on these three samples, as well as three indentations on each sample destined for pre-straining in order to obtain a hardness baseline. The average baseline hardness was 153 HV (Vickers Hardness) with standard deviation (S.D.) of 3.29 HV.

Knowing the benchmark mechanical properties, hardness versus strain calibration curves were developed. The first calibration curve, relative to the “un-aged” steel, was obtained pre-straining the samples up to the aforementioned pre-strain and measuring the hardness immediately after. Note that, during the pre-straining phase, the software controlling the MTS machine was set to stop at the above-mentioned strain limits. The elastic recovery during the unloading phase was considered negligible. In addition, it was believed more convenient to use engineering strain instead of true strain.

Table 1. Average Tensile properties of steel reinforcing bar.

Sample	Building	Yield strength (MPa)	Ultimate tensile strength (MPa)	Uniform Elongation (mm/mm)
1	**A**	401	501	0.205
2	**A**	404	504	0.203
3	**A**	392	506	0.204

For each pre-strained sample, 10 hardness measurements at 4 mm spacing were made. To investigate the strain ageing effect, an accelerated ageing process was adopted. Strained samples were immersed in boiling water (100ºC) for four hours, which is intended to simulate the effect of ageing steel at 15ºC for one year (Hundy 1954). The accelerated ageing procedure was also specified in the superseded NZS 3402:1989 standard (Steel bars for the reinforcement of concrete) (NZS 1989).

After artificial ageing, the pre-strained samples were hardness tested again. Finally, the samples were tensile tested up to failure to determine the basic mechanical properties: YS, UTS and strain at UTS. The outcome of this initial phase of the experiment was: a) hardness versus plastic strain calibration curve (Fig. 7) and b) residual plastic capacity versus hardness curve (Fig. 8). Note that the 'residual plastic capacity' is the strain at UTS for the strain aged samples as a percentage of the original strain at UTS. These calibration curves were used to quantify the permanent plastic deformation and the predicted remaining ductility of the damaged bars.

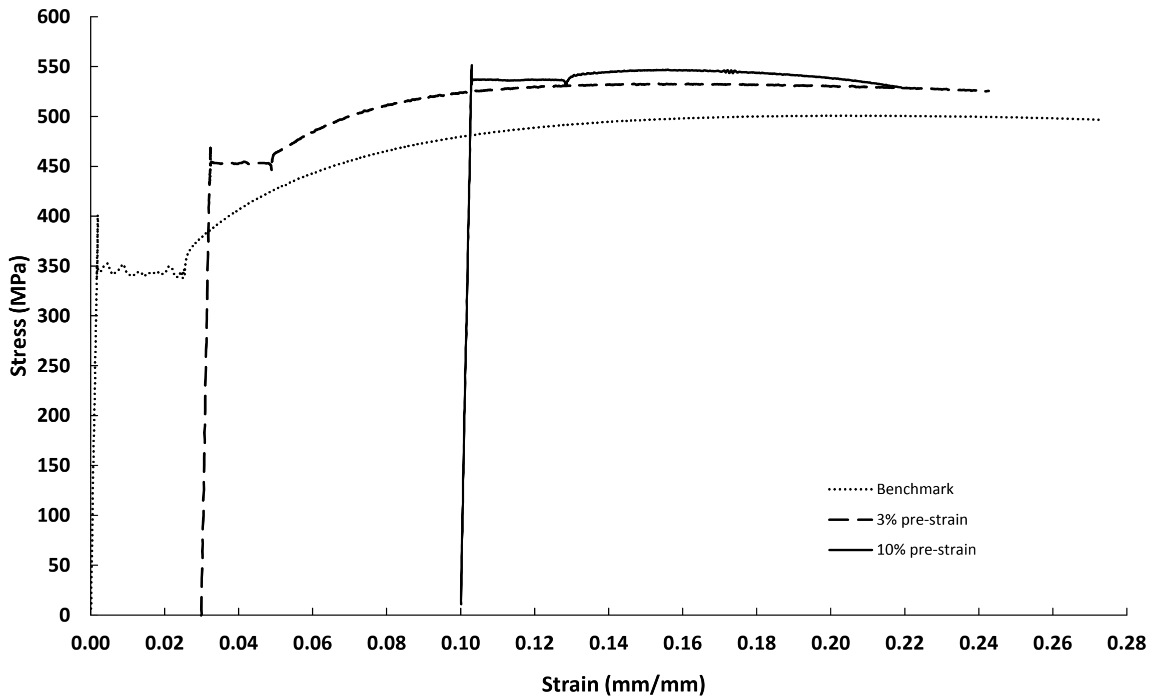

Figure 6. Sample tensile stress-strain curves for baseline materials, 0.03 mm/mm pre-strain and age and 0.10 mm/mm pre-strain & age.

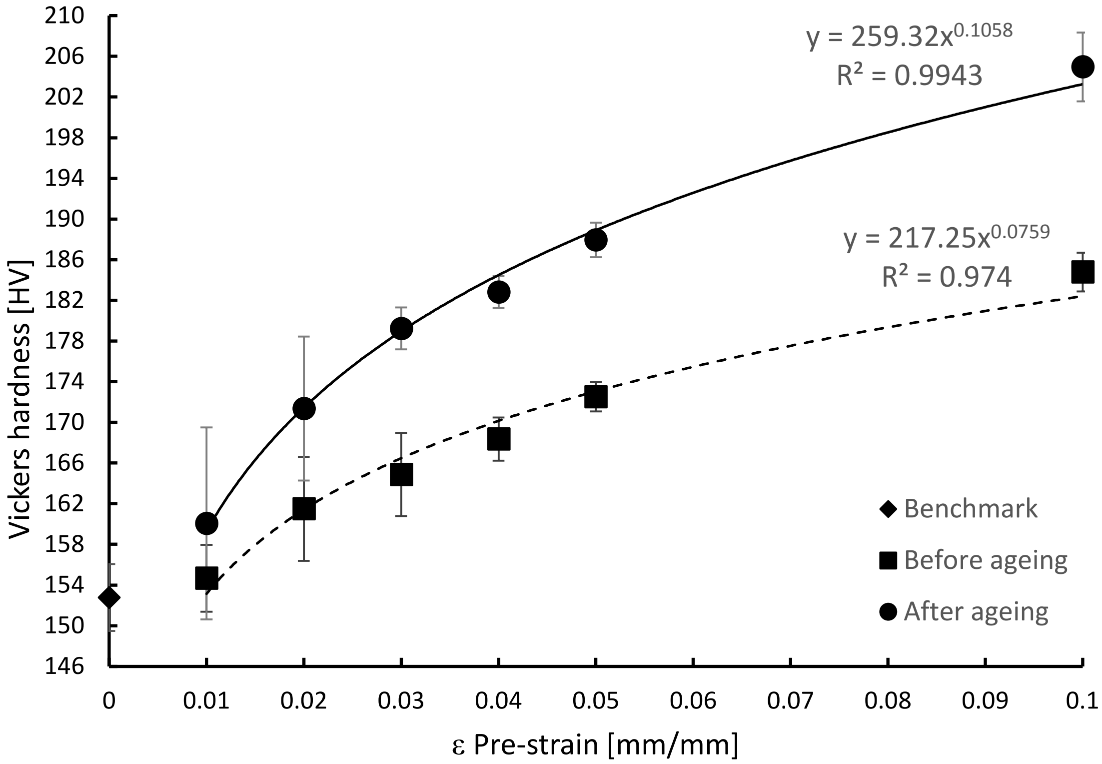

Figure 7. Hardness versus pre-strain calibration curve for the un-damaged steel re-bar obtained from building A.

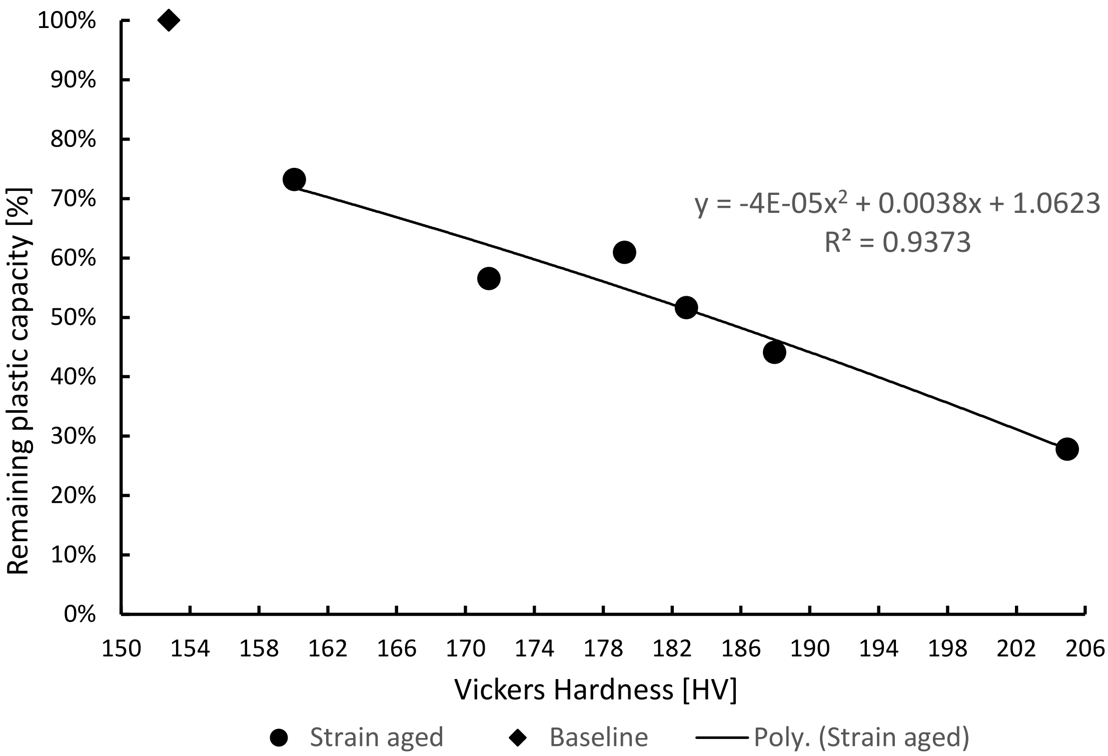

Figure 8. Remaining ductility versus Vickers hardness calibration curve for the un-damaged steel re-bar obtained from building A.

2.3 Phase III – Vickers Hardness Testing and Damage Assessment

In Phase 3, the 4 bars that showed the greatest increase in Leeb hardness along the longitudinal section were removed from the building and brought to the University of Canterbury laboratories for further testing. It was first required to cut the entire bars as received into 120-150mm lengths to allow for surface preparation. Two opposite sides of each bar section surface were ground flat and parallel (oriented 90 degrees to the Leeb hardness surface) using a water cooled grinder, sequentially ground to 600 grit using silicon carbide papers, and then finally polished to a 9-micron finish. Hardness measurements were conducted along the longitudinal section of the steel bar at 2 mm spacings. A typical Vickers hardness profile (ASTM 2011b) of Building A is shown in Figure 9, which exhibited two peaks corresponding at hardness values of 192 and 204.

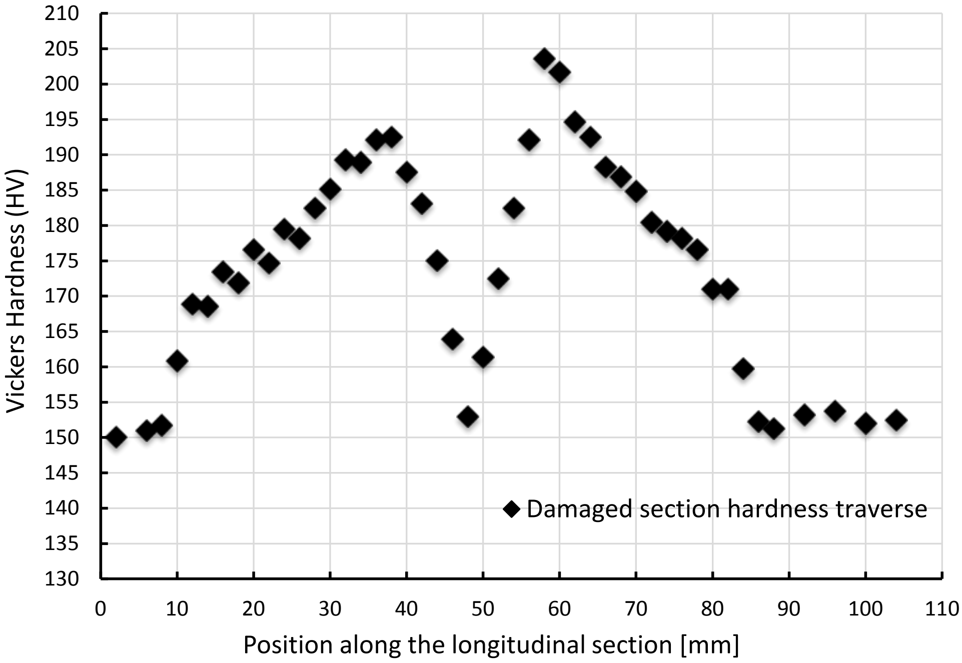

Figure 9. Hardness longitudinal profile of the damaged re-bar obtained from building A.

Based on the calibration curves produced (Fig. 7 and Fig. 8) and the maximum hardness of 204, the bar in the most damaged location had undergone approximately 0.10 mm/mm plastic strain and the residual plastic capacity predicted at that location would be reduced to approximately 28% of the original capacity (i.e. reduced from 0.20 mm/mm to 0.06 mm/mm). On the other hand, the lower yield stress of the steel in the damaged location has increased from ~325MPa to ~430 MPa.

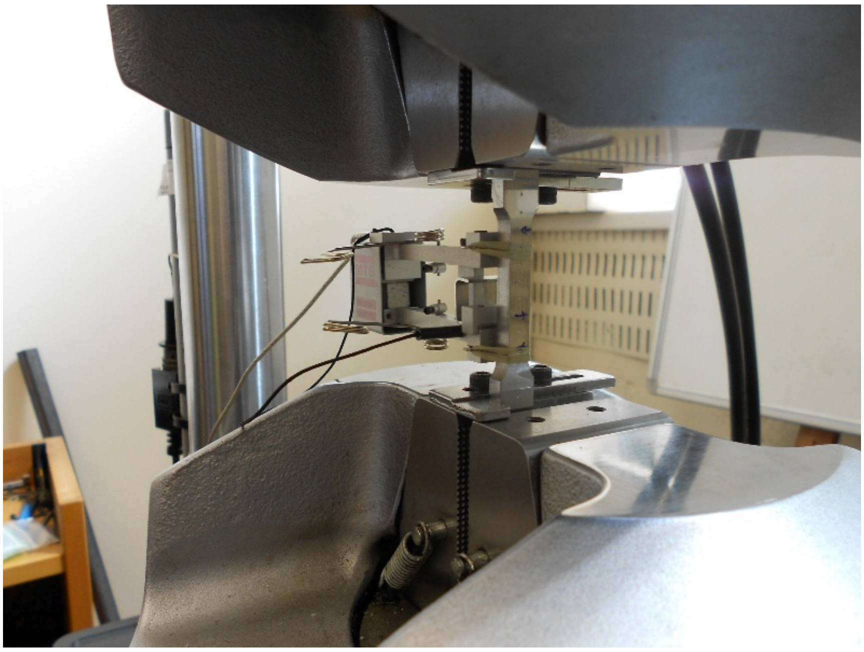

Figure 10. Tensile testing of the damaged steel re-bar, the extensometer gauge length is meauring the strain in the damaged location.

Finally, to verify that predicted residual plastic capacity was correct, the damaged bar was machined for tensile testing. The extensometer was located within the damaged area so to measure the mechanical properties on that location (see Fig. 10) and the specimen was tensile tested (see results in Fig. 12). The strain at UTS of the damaged re-bars found during the test over the entire extensometer gauge length was of 0.067 mm/mm, similar to the prediction via the hardness calibration. As expected during the tensile test the damaged specimen started to neck in correspondence to the "valley" location (position 45-55 mm, Fig. 9) where the hardness and yield strength were lower. Furthermore, observation of the specimen post-test indicated that the location of highest hardness the bar seemed not to have deformed at all (see Fig. 11). In fact, the bar in the most damaged location (where the hardness was the highest) has a yield stress of 500 MPa, and most likely remained elastic during the tensile test.

Figure 11. The damaged samples after the final tensile test, the neck region corresponds to the area where the hardness was lower.

The hardness measurements, supported by the results of the tensile test, demonstrated that the material no longer has uniform mechanical properties. Yield stress, ultimate tensile stress and ductility varied substantially along the longitudinal section. It can be deduced that the regions of steel with higher hardness are "protected" from further damage because their yield stress is higher than adjacent regions. However, the material has lost ductility over the length containing the damage. If further plastic capacity is not made available by further debonding from the concrete, then further elongation will be limited.

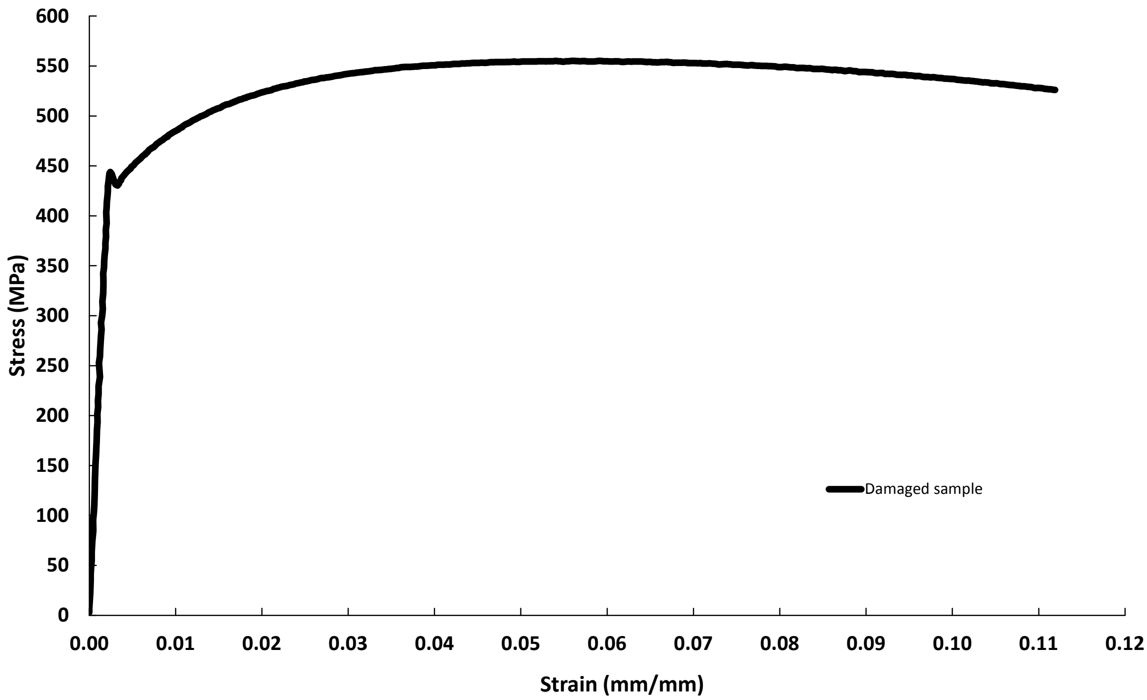

Figure 12. Stress-strain curve of the damaged steel re-bar.

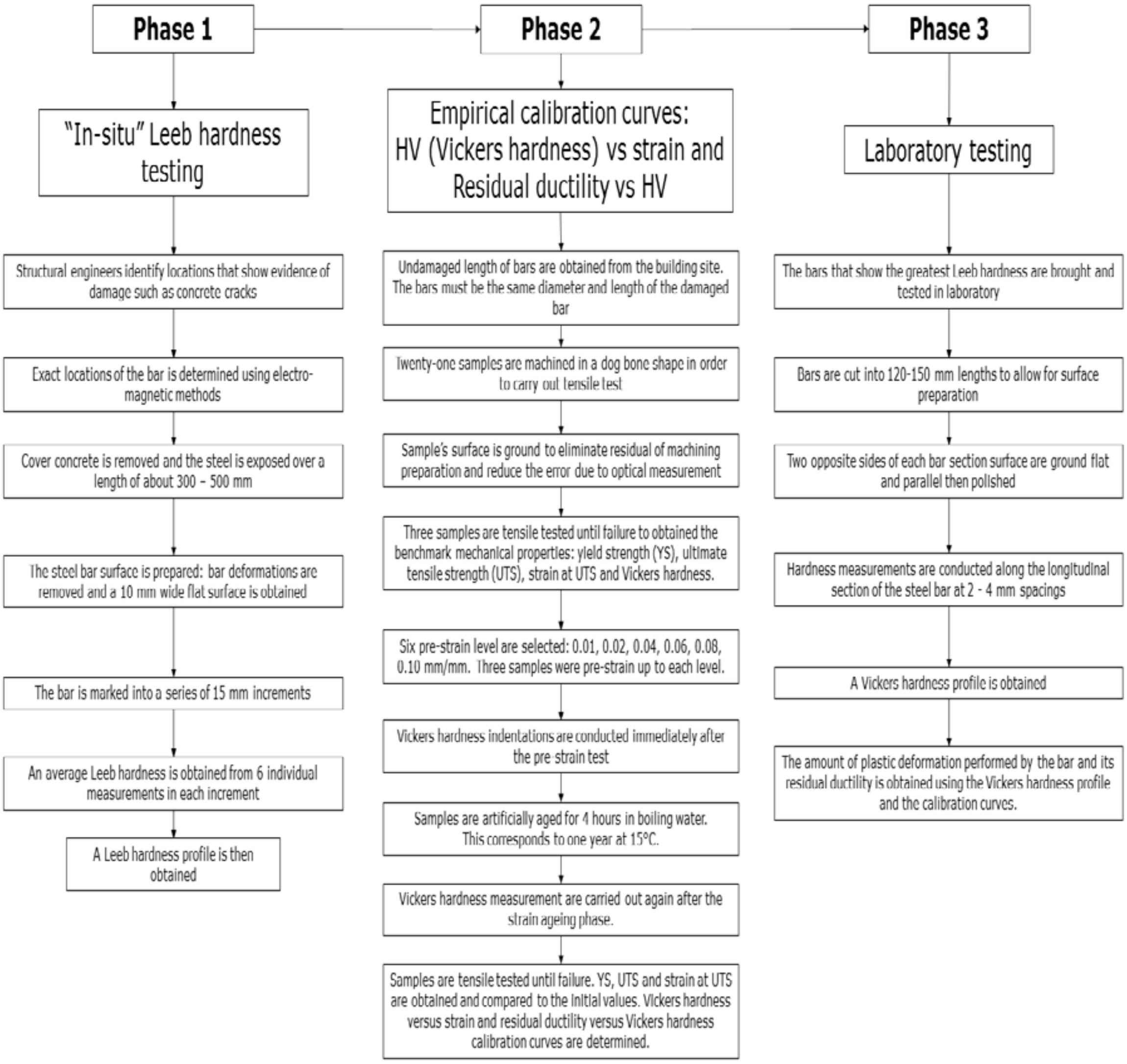

Figure 13. Flowchart of the overall methodology.

3 SUMMARY AND CONCLUSIONS

The experimental testing campaign presented in this paper was carried out to verify the feasibility and potential of the "Vickers hardness method" intended to be a tool integrated with the "in-situ" Leeb hardness test. The Vickers hardness method aims to provide more accurate results, better insight about the mechanical properties of the damaged steel bars, and takes strain ageing into account. The results obtained in the bars obtained from the Christchurch CBD buildings showed a good agreement between predicted and effective residual ductility. The Vickers hardness method is therefore a reliable tool to provide accurate information regarding the damage of steel re-bars.

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5 REFERENCES

ASTM. 2011a. E8/E8M - 11. Standard Test Method for Tension Testing of Metallic Materials

ASTM. 2011b. E384 - 11$^{\varepsilon 1}$ Standard Test Method for Knoop and Vickers Hardness of Materials

Baird, J. 1971. The effects of strain-ageing due to interstitial solutes on the mechanical properties of metals. *Metallurgical Reviews*, 16(1): 1-18.

Erasmus, L. & Pussegoda, L. 1977. Strain age embrittlement of reinforcing steels. *New Zealand Engineering*, 32(8).

Hall, E. 1951. The deformation and ageing of mild steel: III discussion of results. *Proceedings of the Physical Society. Section B*, 64(9): 747.

Hundy, B.B. 1954. Accelerated Strain Ageing of Mild Steel. *Journal of the Iron and Steel Institute* 178.

Loporcaro, G., Pampanin, S. & Kral, M.V. 2014. Investigating the relationship between hardness and plastic strain in reinforcing steel bars. NZSEE conference, Auckland

Matsumoto, Y. 2009. Study on the residual deformation capacity of plastically strained steel. *Behaviour of Steel Structures in Seismic Areas*CRC Press.

NZS. 1989. 3402:1989. Steel bars for the reinforcement of concrete

Pampanin, S. 2012. Reality-check and Renewed challenges in Earthquake Engineering: Implementing low-damage structural Systems–from theory to practice. 15th World Conference on Earthquake Engineering.

Paulay, T. & Priestley, M.J.N. 1992. *Seismic design of reinforced concrete and masonry buildings*. New York: John Wiley & Sons, Inc.