Accounting for residual capacity of reinforced concrete plastic hinges: current practice and proposed framework

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ABSTRACT: According to capacity design principles developed since the 1960s-1970s, structures are designed to withstand major earthquakes by developing inelastic action and energy dissipation in concentrated regions referred to as plastic hinges. This in turn, and almost inevitably when using traditional monolithic connections, leads to structural damage, often over the irreparability threshold. Despite the availability and recent development of seismic assessment and rehabilitation guidelines, they are mainly focused in mitigating the seismic risk of existing buildings designed prior to capacity design principles. Very little information and assistance is provided in assessing the residual capacity of damaged, yet well designed according to modern seismic codes, buildings.

Residual capacity has been traditionally assessed by means of modification factors that account for residual drifts, stiffness and strength deterioration of the damaged elements. However and within this context, the assessment of the residual fatigue life of the plastic hinges has not been completely addressed. When considering the problem, past research tended to focus on the assessment of the low-cycle fatigue of the longitudinal reinforcement, which is only one part of the overall picture and, when looking at member level, has focussed mainly on bridge columns.

As part of a wider research project aiming at investigating the seismic residual capacity of reinforced concrete frames, this paper presents a review of the current know-how and previous research on this topic. A new framework to account for residual capacity in a complete manner, at both the plastic hinge and structure levels in the design and assessment processes, is qualitatively proposed.

1 INTRODUCTION

Reinforced concrete frames have been widely used as part of the lateral force-resisting system in buildings located in high seismic risk areas. Beam, columns as well as beam-column joints, when properly designed and detailed following capacity design principles developed since the 1960s-1970s, are supposed to be able to resist seismic internal actions during several post-elastic displacement cycles without significant strength and stiffness deterioration, by developing inelastic action and energy dissipation in concentrated regions or plastic hinges.

The development of inelastic action in structures with traditional monolithic connections inevitably leads to structural damage, often above what would be considered uneconomical to be repaired as it was observed after the 2010-2011 Canterbury earthquake sequence, where moderate-to-severe ductile beam end hinging was exhibited (see Figure 1).

Despite seismic assessment and rehabilitation guidelines may be available since the inception of the first formal seismic codes in the first half of the 20th century, they are mainly focused in mitigating the risk of death or injury from existing earthquake-prone buildings (typically referring to pre-1970s buildings, thus designed prior to capacity design principles).

Figure 1. Plastic hinges in the beam-ends on the Precast 22-storey perimeter frame of the Price Waterhouse-Coopers PWC in Christchurch, New Zealand, after the 2010-2011 Canterbury earthquakes sequence.

Moreover, residual capacity of buildings has been traditionally assessed by means of modification factors calibrated experimentally, which (and depending on the amount of damage observed after the earthquake) account for residual drifts, stiffness and strength reductions of those damaged elements (Maeda et al, 2004; Polese et al., 2012; for instance). Very little (if any) assistance in assessing the residual fatigue life (i.e., the remaining number of equi-amplitude load reversals before reaching failure) of damaged buildings, has been provided.

The above is especially important when assessing existing, yet well designed buildings, where (and depending on the amount and type of observed damage) the risk of collapse during an aftershock (or a series of) might not be apparent if the assessment is performed based only on stiffness and strength degradation.

This lack of: 1) knowledge on the estimation of the remaining residual capacity of structures to resist aftershocks once a big portion of their initial capacity has been consumed; 2) reliable repairing techniques to bring back the structure at least to its initial conditions; 3) effective tools to estimate their cost-effectiveness for decision-making processes; in addition to the 4) the excessive level of damage observed after a mainshock, have led to disproportionate amounts of demolitions of modern damaged structures.

When considering residual capacity within the context of residual fatigue life, past research has been done on low-cycle fatigue of the longitudinal reinforcement only. Nevertheless, there are other factors such as bond between steel and concrete, amount of longitudinal reinforcement, strength characteristics of steel and concrete, as well as strain-rate effects that strongly influences the plastic hinge cumulative cyclic behaviour and therefore, its residual capacity. In addition, when looking at the member level, research on low-cycle fatigue seems to have been focused mainly on bridge columns (Mander and Cheng, 1999; El-Bahy et al, 1999).

This paper presents a review of the current know-how on fracture of reinforcing steel due to low-cycle fatigue, as well as on seismic residual capacity. A new framework to account for residual capacity in a complete manner at a plastic hinge level is first qualitatively proposed, and further implemented within a full displacement-based approach for the design and assessment processes of multi-degree-of-freedom systems.

Experimental, analytical and numerical work is being conducted to quantitatively define (some of) the key components of the proposed performance-based framework, considering additional factors than low-cycle fatigue on the longitudinal steel (as previously described) when accounting for residual capacity.

2 LOW-CYCLE FATIGUE OF REINFORCING STEEL BARS

As stated by Mander et al (1994), fracture of longitudinal reinforcement due to low-cycle fatigue might be one of the most typical failure modes that can occur in flexural members during an earthquake, especially for structures located in mid-to-high seismic zones when one-to-five fully reversed cycles of large strain equi-amplitudes up to ε_s= 0.06 mm/mm or 6% may be expected.

Despite the importance of this type of failure, in the past there was a lack of research on this topic. The engineering community was more engaged with mechanical engineering applications rather than earthquake engineering ones, where the main interests were specimens tested on low-strain amplitudes (usually less than 0.01 mm/mm or 1%) and high-cycle fatigue (10^3-to-10^7 cycles) regimes. Thus, Mander pioneered the work on experimental low-cycle fatigue campaigns for seismic applications, testing ASTM A722 (f_u= 1083 MPa) and A615 grade 40 (f_y= 276 MPa) unaltered (not machined) bars on constant amplitude cyclic fatigue conditions under a range of seismic (axial) strain amplitudes. The specimens were laterally supported six bars diameters, and incipient failure was defined as initiation of a fatigue crack in the test specimen.

The plastic-strain amplitude with fatigue life relationship proposed by Coffin-Manson (Coffin, 1954; Manson, 1953) was used to fit the data, concluding that a single equation based on plastic strain, ε_p, and number of cycles could be developed to be universally applicable to all reinforcing steels

$$\varepsilon_p = 0.08 \cdot \left(2 \cdot N_f\right)^{-0.5} \tag{1}$$

where N_f = number of cycles to failure.

Later, Kunnath et al (1997) observed that the model of Mander and Cheng (1994) under-predicted the final damage state of all their column specimens experimentally tested, attributing this effect to the fact that the model was capable of evaluating the damage due to low-cycle fatigue of longitudinal reinforcement only, while all tested specimens experienced confinement failure prior to low-cycle fatigue failure. They proposed a new cumulative fatigue model derived from the concrete column as a composite section, indirectly accounting for the cumulated damage due to shear, axial stress and loss of confinement

$$\varepsilon_p = 0.065 \cdot N_f^{-0.436} \tag{2}$$

Brown and Kunnath (2004) performed experimental tests to examine the low-cycle fatigue behaviour of standard ASTM A615 grade 60 (f_y= 420 MPa) reinforcing bars ranging from No. 6 (0.750 inches or 19.1 mm) to No. 9 (1.125 inches or 28.6 mm), aiming at: 1) determining expressions to ensure longitudinal bar fracture failure mode; 2) to correlate failure limit states with seismic damage on reinforced concrete members; and 3) for damage prediction and design. Results indicated that fatigue life is influenced by the diameter of the bar and the geometry of the rolled on deformations.

Hawileh et al (2010) evaluated experimentally the low-cycle fatigue behaviour of ASTM 706 and A615 grade 60 (f_y= 420 MPa) for further application in precast hybrid frame (PRESSS) connections, motivated by the fact that during an earthquake, the gap at the beam-to-column interface opens and can induce high plastic strain levels (up to 6%) at the reinforcing bars which might cause fracture after less than 10 cycles. They encased the specimens into a steel greased collar to provide them lateral support against buckling, similar to the condition of unbonded bars in PRESSS connections. The following low-cycle fatigue relationships were proposed for A706 and A615, respectively

$$\varepsilon_p = 0.103 \cdot \left(2 \cdot N_f\right)^{-0.54} \text{, for A706 bars; and} \tag{3}$$

$$\varepsilon_p = 0.128 \cdot \left(2 \cdot N_f\right)^{-0.57} \text{, for A615 bars} \tag{4}$$

They found that the low-cycle fatigue behaviour of A706 and A615 is very similar, even though when the A706 exhibits higher ductility levels during monotonic tests. Interestingly enough, the average number of cycles to failure in tests with larger strain ranges were higher for A615 specimens.

Figure 2 shows all the plastic-strain, ε_p, to cycles to failure, N_f, relationships previously discussed. As can be seen from the figure, all the curves show a similar trend. It is interesting to note, however, that Kunnath et al (1997) that indirectly accounts for the accumulated damage due to shear, axial stress and loss of confinement, predicts lower plastic-strain levels for low number of cycles to failure (less than around 10 cycles). On the other hand, larger plastic-strain levels are predicted for high number of cycles to failure.

It is noteworthy that the aforementioned investigations on low-cycle fatigue have been carried out with specimens manufactured according to ASTM standards. As part of a wider research project, current research is being done on low-cycle fatigue with specimens manufactured and typically used in New Zealand.

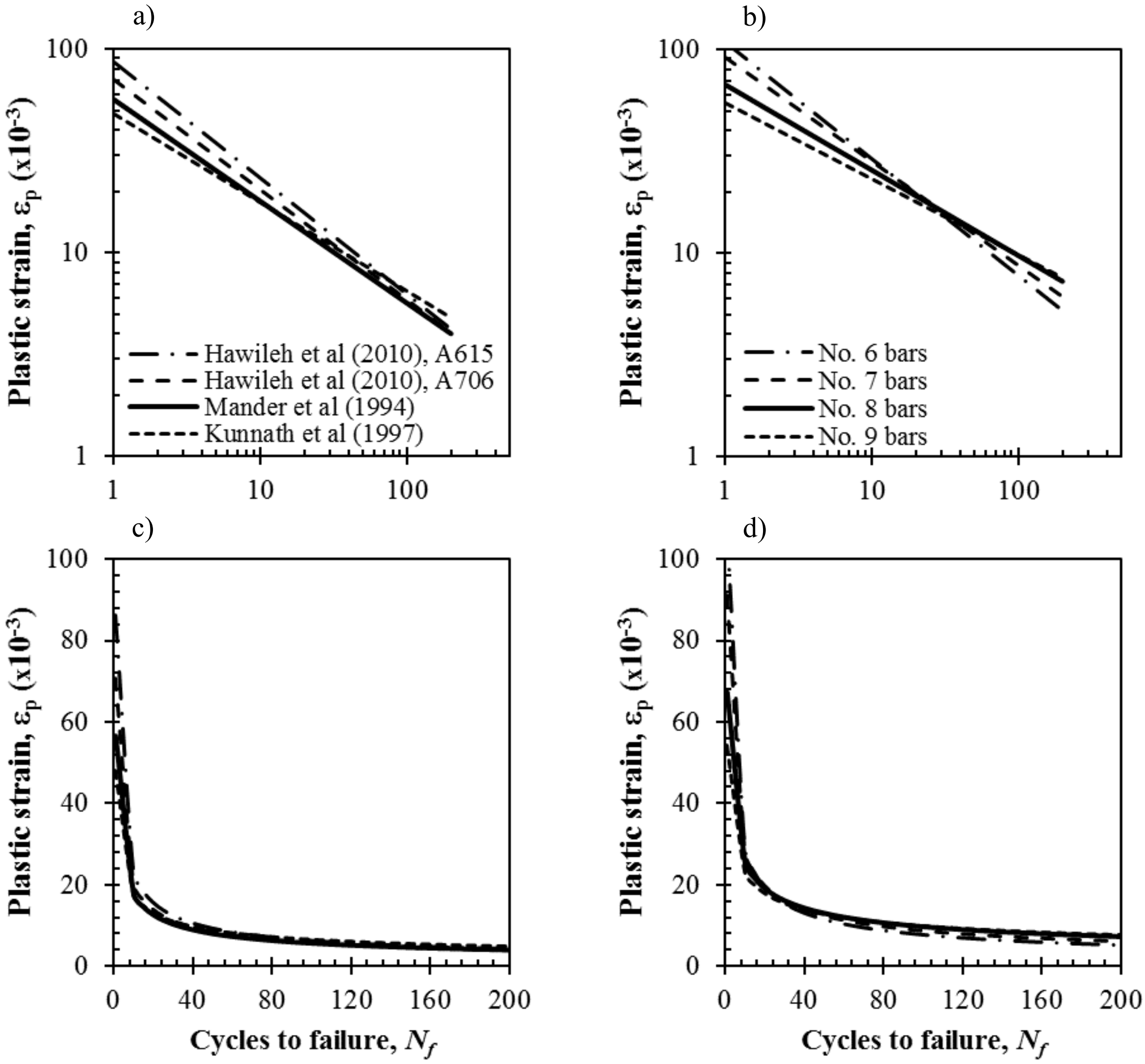

Figure 2. Plastic-strain with fatigue life relationships found in the literature, on a logarithmic (above) and arithmetic (below) scale. Figures b) and d) correspond to Brown and Kunnath (2004) relationships.

3 RESIDUAL CAPACITY: CURRENT PRACTICE

As previously discussed, residual capacity of buildings has been traditionally assessed by means of residual drift, stiffness and strength degradation of those damaged elements; very little information for assessing the residual fatigue life of the plastic has been provided. When considering this, past research has been mainly focused on bridge columns.

Mander and Cheng (1999) investigated the low cycle fatigue failure as part of a study on replaceable (i.e., specially-detailed reinforcing fuse-bars) plastic hinges in bridge columns. They stated that failure in a concrete bridge column might be due to either: 1) fatigue of the longitudinal reinforcing steel; 2) failure of the concrete due to lack of confinement or fracture of the transverse reinforcement; and or 3) compression buckling of the longitudinal reinforcement. Since potential plastic hinge zones are properly designed and detailed with adequate transverse reinforcement, they assumed the fatigue of the steel was the main failure mode, supporting the need of replaceable plastic hinges as an alternative repairing technique.

Based on the dependable plastic strain and fatigue life relationship proposed by Mander et al (1994) assuming a linear strain profile across the concrete column, they obtained the following plastic curvature and fatigue life relationship

$$\phi_p \cdot D = \frac{0.113}{1 - 2 \cdot d'/D} \cdot N_f^{-0.5} \tag{5}$$

where $\phi_p D$ = dimensionless plastic curvature amplitude; d' = depth from the outermost concrete fibre to the centre of reinforcement; D = overall column diameter.

The plastic curvature experimentally determined is estimated as

$$\phi_p \cdot D = \frac{\theta_p}{L_p/D} \tag{6}$$

where $\theta_p = \theta_u - \theta_y$ (maximum experimentally observed drift minus experimentally observed yield drift); L_p = equivalent plastic hinge length (value typically assumed by the engineer based on literature).

In order to apply the theoretical model to a variable amplitude displacement history, they combined a dependable total strain and fatigue life relationship (Mander et al, 1994) with the well-known Miner's rule (Miner, 1945)

$$N_{f,eff} = \sum_i \left(\frac{\theta_{ui}}{\theta_{eff}} \right)^3 \tag{7}$$

where $N_{f,eff}$ = effective number of cycles at a constant drift amplitude, θ_{eff}.

Column specimens were tested under constant and variable drift amplitude. They observed that with the exception of one specimen, low-cycle fatigue of the longitudinal reinforcement was the dominant failure mode. The predicted number of cycles were compared with the experimentally observed effective number of cycles at first fatigue crack, finding the aforementioned theoretical prediction a little conservative, especially for high-cycle fatigue (i.e., $N_f \sim 100$ or more), attributed to the conservatism in the assessment of the effective plastic hinge length.

Later, El-Bahy et al (1999) investigated the cumulative damage in reinforced concrete circular bridge columns, aiming at determining their low cycle fatigue characteristics while addressing issues related to damageability and reserve capacity. The investigation was based on the premise that structural damage is related to the fatigue behaviour of concrete and steel.

They observed three failure modes: 1) global buckling of the longitudinal bars; 2) confinement failure following the rupture of the confining spirals; and 3) low-cycle fatigue of the longitudinal reinforcement. Only flexural failure members were considered in this study, and based on the results of constant amplitude tests, they developed the following drift-based fatigue-life expression for damaged-based seismic design of circular flexural dominated bridge columns

$$\delta = 10.6 \cdot N_f^{-0.285} \tag{8}$$

where δ= lateral drift, in percent.

4 NEW FRAMEWORK TO ACCOUNT FOR RESIDUAL CAPACITY IN SEISMIC DESIGN AND ASSESSMENT PROCESSES

While past research on residual capacity (within the context of residual fatigue life) has been mainly focused on bridge columns, current design and assessment procedures for new structures are based on an expected performance level given a certain level of seismic excitation. This concept although very useful, provides an incomplete description of performance level from a "damage control" point of view, as there is no correlation between the expected level of structural damage (to which we are designing for) and the number of load reversals the element can resist (i.e., the expected level of structural damage given a certain amount of equi-amplitude cycles, for a given ductility demand).

Therefore, in a refined performance-based design and assessment framework it would be very important to relate the performance level not only with the maximum ductile structural response expected during the design earthquake, but also to the expected level of damage from a residual capacity point of view. This is especially true when the building remains standing under the mainshock and there is a need for estimating the remaining residual capacity of the structure to resist aftershocks once a big portion of their initial capacity has been consumed, in order to support the complex decision-making process on how and whether or not to make safe, repair, retrofit and/or demolish.

In the following, the main components of the proposed framework are first explained. The first two are related to current practice adapted for this purpose; while the other two refers to past research that is revisited, modified and complemented with new research based on the current knowledge, new evidences and latest available tools. All these components are then incorporated in a full displacement-based approach so that it can be considered for both design and assessment processes.

Experimental work on real beam-column joints extracted from the PWC building after the 2010-2011 Christchurch earthquakes sequence (see Figure 1), as well as analytical and numerical simulations are in progress as part of the aforementioned research program to quantitatively define (some of) the key components of the proposed performance-based framework accounting for residual capacity.

4.1 Main components of the proposed framework

Figure 3 outlines the main components of the residual capacity framework. As it can be seen, the first step (Figure 3a) is related to the *seismic demand*. It is well-known that elastic displacement spectra can be developed using the same set of accelerograms used to develop acceleration spectra, or through pseudo-spectral relationships based on the assumption that peak responses follow the equations of steady-state sinusoidal response.

Depending on the local or site specific seismicity of the area being considered, displacement spectra may be represented as two straight lines and a corner period T_C (first period defining the plateau or maximum displacement). As stated by Priestley et al (2007), the corner period appears to increase almost linearly with magnitude, while the maximum displacement depends on magnitude, epicentral distance, and the stress drop during rupture, which can be estimated as part of a PSHA for the site of interest, for the mainshock and "possible" aftershock(s). Therefore, the elastic response spectrum for which the structure will be designed (and at a later stage, the corresponding to one or more aftershocks) should be defined. For post-earthquake evaluations and required earthquake interventions, elastic response spectra can also be computed from actual records.

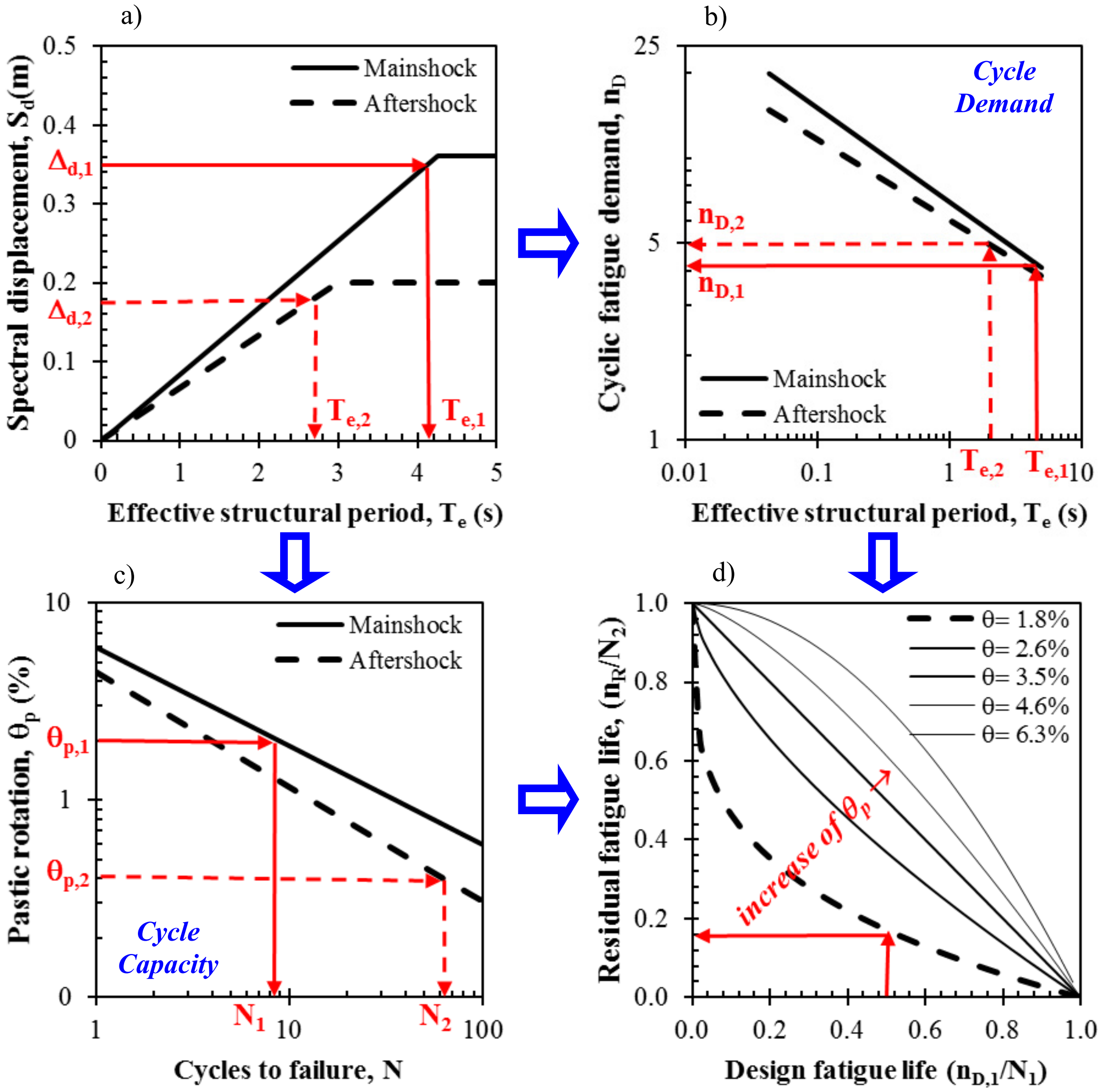

Figure 3. Main components of the new residual capacity framework.

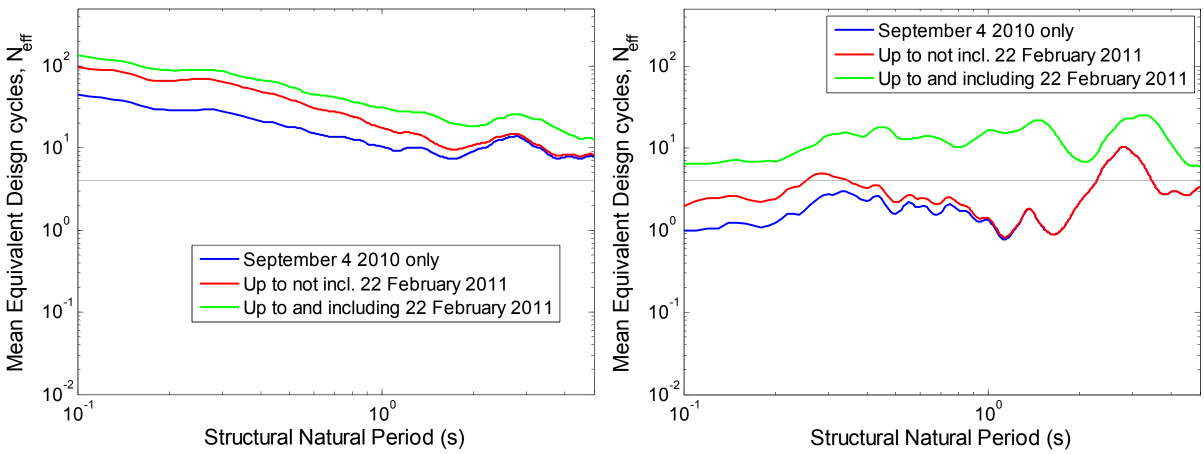

Figure 4. Total number of equivalent design code cycles for fatigue exponent C= 1 (left) and C= 2 (right), based on NZS1170.5 (Mander and Rodgers, 2013). Courtesy of G. W. Rodgers.

The second step involved in the process (Figure 3b) is to estimate *the equivalent number of cycles* the structure will experience during the mainshock (and at a later stage, during the aftershocks). The difference between the two relationships (i.e., mainshock and aftershocks) might depend on the seismogenic source characteristics, magnitude (i.e., duration) and epicentral distances, in addition to the dynamic characteristics of the structure (e.g., more or less damping, stiffness and strength reduction, yielding point and post-yielding stiffness, among others).

According to past studies, low-cycle fatigue demand of reinforced concrete structures might be assessed through fatigue demand spectra generated from spectral results of inelastic SDOF systems, based on hysteretic models calibrated to be able to capture the inelastic behaviour of MDOF systems. Based on the previous approach, Mander and Cheng (1999) proposed the following empirical equation to conservatively estimate the cyclic fatigue demand

$$n_d = 7 \cdot T_e^{-1/3} \tag{9}$$

where n_d = cyclic fatigue demand; and T_e = effective structural period.

More recently, Mander and Rodgers (2013) normalized time history responses to an equivalent number of cycles at a code-based seismic displacement amplitude (based on NZS 4203:1984 and NZS1170.5:2004), using fatigue exponents C of 1, 2 and 3, for concrete-critical fatigue, reinforcing-steel critical fatigue, and structural-steel fatigue (see Figure 4). The reader is referred to Mander and Rodgers (2013) for further information.

The third step (Figure 3c) is to determine *the plastic drifts and fatigue life relationships*, referred to as the amount of cycles the plastic hinge can sustain during the mainshock (and eventually, during the aftershocks) before failure occurs. As stated in Priestley et al (2007), the yield curvature is relatively insensitive to the axial load ratio and reinforcement ratio, depending mainly on structural geometry and material sizes. Thus, for circular columns such as bridge piers, for instance, the yield curvature ϕ_y and then, the yield drift θ_y, can be estimated as

$$\phi_y = 2.25 \cdot \frac{\varepsilon_y}{D} \tag{10}$$

$$\theta_y = \phi_y \cdot \frac{H}{2} \tag{11}$$

where $\varepsilon_y = f_y/E_s$; and H = column height.

Similar expressions for yield curvature and drift can be found in the literature for reinforced concrete wall and frame structures. In general, plastic drifts can be obtained subtracting the yield drift from the maximum (i.e., design) drift θ_d

$$\theta_p = \theta_d - \theta_y \tag{12}$$

Once the plastic strain has been estimated, it can be related to plastic drifts or rotations assuming that the latter occurs at the centre of a plastic hinge of length L_p, and neglecting shear deformations (Tsuno and Park, 2004)

$$\varepsilon_p = \left(\frac{\bar{d}}{2}\right) \cdot \left(\frac{\theta_p}{L_p}\right) \tag{13}$$

where $\bar{d}$ = distance between centres of longitudinal bars.

Equation (13) can be combined with relationships such as equations (1) to (4) to provide a reasonable estimate of maximum plastic drifts for a given number of cycles before failure.

In the previous equation is noteworthy that the plastic strain is highly dependable on the assumption of the plastic hinge length, which has been observed from previous earthquakes to be much lower than expected by formulas in the literature (a lower plastic hinge length inevitably leads to higher plastic strains and consequently, longitudinal bar fracture).

The fourth and final step (Figure 3d) in the proposed framework relates to a procedure to estimate the *remaining residual capacity* of the plastic hinge to resist aftershocks, once a big portion of its initial capacity has been consumed during a mainshock. This issue has not been properly addressed in past research, and for that purpose, in order to provide reasonable relationships for seismic design, a concept similar to that of Damage Curves proposed by Manson and Halford (1981) for treating cumulative fatigue damage under complex or variable loading history, is used.

Given a cyclic fatigue demand during the mainshock $n_{D,1}$ (see Figure 3b) as well as a capacity fatigue life N_1 for a plastic rotation demand $\theta_{p,1}$ (see Figure 3c), a series of design ($n_{D,1}/ N_1$) and residual capacity (n_R/N_2) curves can be created, one for each plastic drift demand $\theta_{p,2}$ expected during the aftershock(s). Once N_2 is estimated (see Figure 3c), the remaining dependable capacity n_R to resist an aftershock can be estimated and compared with the cyclic fatigue demand $n_{D,2}$ the plastic hinge will be subjected to during the aftershock(s). If the calculated residual capacity is not satisfactory, the plastic hinge characteristics should be modified until the new structural period leads to a n_R that exceeds $n_{D,2}$. Alternatively, the design displacement or drift level should be reduced accordingly.

After the 2010-2011 Canterbury earthquake sequence, in a number of cases few major cracks opening were observed instead of a well distributed cracking pattern expected in those plastic hinge locations were plastic deformation was expected to occur, causing a large amount of deformation concentrated at a single location and leading to a low-cycle fatigue of the reinforcing steel (SESOC, 2011). In order to provide reliable estimates of the residual capacity of a plastic hinge region, it is necessary, therefore, to address the main issues of how such plastic strains and fatigue life relationships, as well as plastic hinge lengths are affected by factors like strain rate, bond deterioration, longitudinal reinforcement ratios, as well as material characteristics (e.g., strain hardening of steel, concrete tensile strengths). The above observations and are being subject of study by means of the experimental, analytical and numerical work comprised within this research program.

For assessment processes, methodologies to estimate residual plastic strains in the steel such as those investigated by Loporcaro et al (2014) are very useful and might be implemented within the proposed framework for better post-earthquake evaluations and decisions on required earthquake interventions.

4.2 Incorporation into a full displacement based design framework

Christopoulos and Pampanin (2004) proposed a displacement-based design procedure accounting for residual deformations or drifts in combination with the more traditional maximum deformations or drifts as performance indicators. In their proposed framework, a critical residual deformation or interstorey drift is estimated and then compared with a performance target; the structural design is then changed in order to improve the predicted response and satisfy the requirements, if needed. Herein, this framework is further developed to explicitly incorporate residual capacity in the overall procedure.

As shown in Figure 5, two design check levels (one for the residual capacity and one for the residual deformations) are required, once they are met, the procedure follows as in the original DDBD formulation proposed by Priestley et al (2007). Within this full procedure, residual capacity can be used in SDOF systems for seismic design and assessment processes. The proposed method (qualitative at this stage) can be extended and further implemented for MDOF systems by relating the plastic rotation demands at each plastic hinge with an equivalent plastic drift on the substitute SDOF structure.

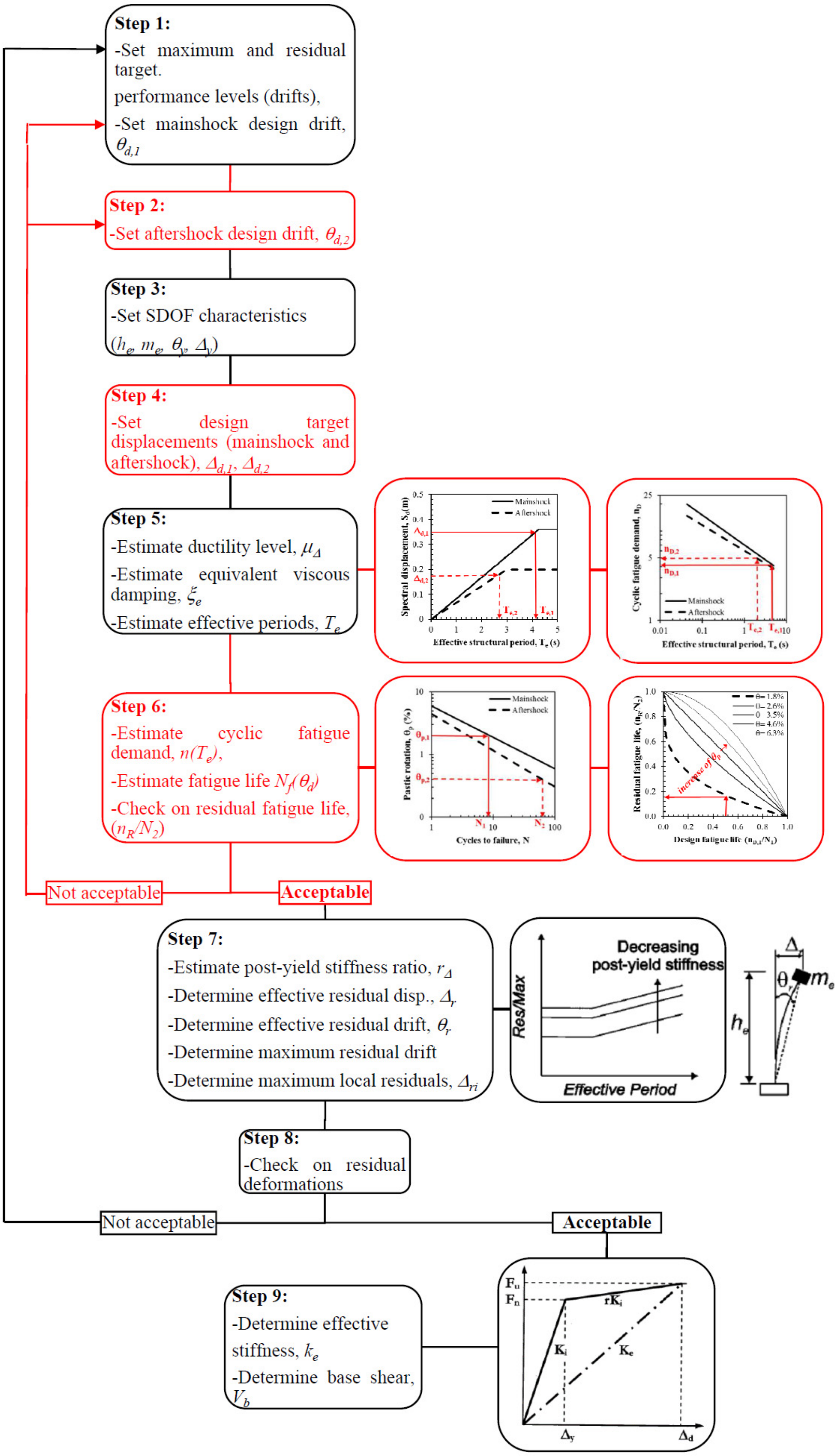

Figure 5. Flowchart of DDBD procedure with consideration of residual deformations and residual capacity (adapted from Pettinga et al, 2007).

5 SUMMARY AND CONCLUDING REMARKS

The excessive level of damage observed during previous earthquakes on structures with traditional monolithic connections, yet well-designed according to modern codes, has highlighted the crucial need to move towards a damage control philosophy and low-damage technologies whilst improving design, assessment and repairing techniques for such traditionally designed structures. This excessive level of damage, in addition to the lack of knowledge on the estimation of the remaining residual capacity of structures to resist aftershocks once a big portion of their initial capacity has been consumed, among others, have led to disproportionate amounts of demolitions of modern damaged structures.

Residual capacity has been traditionally assessed by means of modification factors that account for residual drifts and stiffness and strength reductions of those damaged elements. However, very little assistance in assessing the residual fatigue life (i.e., the remaining number of equi-amplitude load reversals before reaching failure) of damaged buildings, has been provided. When considering the problem, past research seems to have been focused mainly on bridge columns, investigating the fracture of reinforcing steel due to low-cycle fatigue without considering other factors such as bond between steel and concrete, strain-rate effects, amount of longitudinal reinforcement, and strength characteristics of steel and concrete that strongly influences the plastic hinge cumulative cyclic behaviour and therefore, its residual capacity.

This paper tentatively propose in a qualitative manner a new framework to account for residual capacity at a plastic hinge level in a complete manner, that can be extended and further implemented for MDOF systems by relating the plastic rotation demands at each plastic hinge with an equivalent plastic drift on the substitute SDOF structure. Experimental, analytical and numerical work is being conducted to quantitatively define the key components of the proposed performance-based framework.

At a later stage of the ongoing process, and due to the significance of the cyclic loading effects on the cumulative damage induced in reinforced concrete structures, the proposed framework will be calibrated with numerical and experimental results such that the design drift and fatigue life are related to performance levels (e.g., cumulative damage), useful for seismic design and assessment processes where preliminary calculations and non-destructive testing (for seismic assessment) can provide reliable estimates of the remaining residual capacity of the building.

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