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PERFORMANCE AND RESIDUAL LIFE OF MOMENT RESISTING STEEL FRAMES AFTER SEVERAL EARTHQUAKES

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Summary. The damage assessment in structures that have suffered one or more earthquakes is of paramount importance to predict its post-earthquake behaviour. The actual version of the European seismic design code (EC8) does not give practical indications about the estimation of damage in connections and members, proposing only a general methodology based on direct *in-situ* visual inspections.
The paper proposes a seismic design strategy to appraise the structure residual life, by combining the well-known non-linear time-history (NLTH) analyses with the low-cyclic fatigue design approach (LFC). Applications are proposed for a planar steel moment-resisting (MR) frame in high ductility class. The discussed research outcomes show how the damage, which is currently neglected during the design phase, is a very useful parameter able to allow for an appraisal of the effective frame performance after one or more earthquakes as well as for the identifications of components needing urgent repairs.

1 INTRODUCTION

Steel structures represent an optimal seismic solution because of the ductile properties of the basic material combined with the presence of suitable components generally designed to dissipate earthquake energy[1]. The traditional design approaches, based on linear seismic analysis[2] (i.e. the equivalent force method and the response spectrum analysis method), plus member verification checks[3], cannot give any information about the damage evolution. In particular, these methods are not able to account for the deterioration mechanisms that could occur during the earthquake in the dissipative zones. Their effective behavior is remarkably influenced by pinching and stiffness/strength drops and can be directly captured only by non-linear time-history (NLTH) analysis methods.

In the actual version of the European seismic design code of steel frames[4], identified in the following as EC8, fatigue checks are not required for appraising the residual life and engineer's attention has to be addressed only on the ductility and resistance of the components. In fact, it is only prescribed to perform visual *in-situ* inspection directly on the deformed structure. Unfortunally, from the practical point of view, inspections are often difficult because of the presence of non-structural elements that hamper the identification of damaged structural elements, like members or beam-to-column connections.

Only few researches investigated the topic of the evaluation of the damage appraisal. In particular, Ballio et al.[5] investigated the reliability of Moment Resisting (MR) steel frame components after the earthquake by suitably extending high-cycle fatigue approach. In this paper, a more refined approach, which has been initially proposed for steel storage racks[6], i.e. frames with components belonging to class 4[7], has been suitably extended to traditional carpentry frames. Key results of the procedure applied to a practical case composed by a MR steel frame has been discussed, focusing attention on both the post-earthquake residual load carrying capacity and on the damage level reached, after one or more earthquakes.

2 THE PROPOSED NLTH-LCF PROCEDURE

The proposed procedure is based on the non-linear time-history (NLTH) analysis, suitably improved, to account for low-cycle fatigue (LCF) effects and for the reduction of structural performance. In particular, reference is made to the Bernuzzi *et al.*[8] proposal for LCF analysis of semi-rigid steel joint, where the transition between the safe and unsafe zones (Figure 1) can be expressed as:

$$N(\Delta\Phi)^3 = K \tag{1}$$

where N is the number of cycles to reach failure at the constant rotation $\Delta\Phi$ range[8], distinguishing safe and unsafe regions, in a log-log scale. K is a constant, depending on both joint details and material properties.

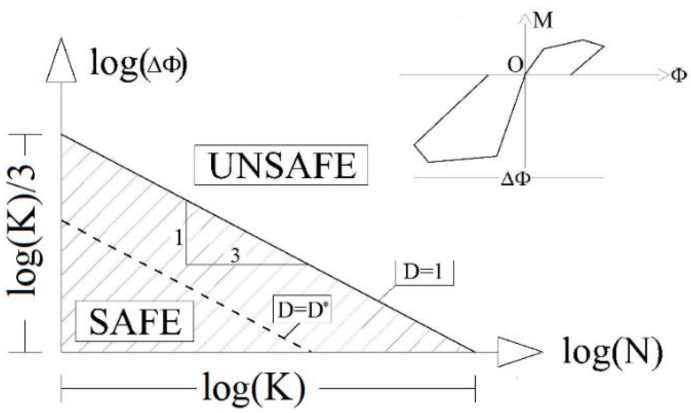

Figure 1: Fatigue resistance line in log(S)-log(N) domain

This criterion allows only for the evaluation of the fatigue failure of the component of interest when it is subjected to constant amplitude loading history. In the case of variable amplitude loads, instead of $\Delta\Phi$ an equivalent rotation range value, $\Delta\Phi_{eq}$, has to be adopted, which is related to the constant loading history characterized by the same number of cycles (n) leading to the same damage. The term $\Delta\Phi_{eq}$ is defined as:

$$\Delta\Phi_{eq} = \left[\frac{1}{N}\sum \Delta\Phi_i^3\right]^{\frac{1}{3}} \tag{2}$$

where $\Delta\Phi_i$ is the total rotation range of each cycle of the variable amplitude loading history.

As to the cycle counting methods, i.e. the approaches to evaluate $\Delta\Phi_{eq}$, reference can be made to the *rainflow* procedure, which is recommended by the European fatigue design code[9]. Furthermore, it should be of great interest for design purposes to measure the damage associated with a set of subsequent seismic events. The well-known Miner's rule[10] could be conveniently applied also to frame components, making reference to the damage index D, which ranges from 0 (no damage) to 1 (failure for LCF), expressed as:

$$D = \frac{N \cdot \Delta\Phi_{eq}^3}{K} \tag{3}$$

After the steel frame is modelled, for each seismic accelerogram, the proposed procedure is comprised of the following steps:

I. definition of the initial maximum monotonic load carrying capacity (W_{ini}), obtained by a Non-Linear Static Analysis (NLSA) under incremental vertical load;

II. NLTH analyses in which the vertical loads are defined according to EC8 seismic loading condition (W_{seis}), taking into account both geometrical and mechanical non-linearities;

III. basing on the output data, evaluation of the damage index (D) for each joints and components via the LCF theory. Evaluation of the maximum transient ($\theta_{t,max}$) and of the residual (θ_r) interstorey drift;

IV. evaluation of the residual load carrying capacity (W_{res}) via a NLSA on the damaged structure, i.e. by considering the effective performance of the structural components accounting for the reductions due to one or more earthquakes;

V. definition of the most appropriate retrofitting strategy on the base of the interstorey drift and the W_{res}/W_{ini} ratio.

3 THE CASE STUDY

The proposed approach, based on NLTH analysis combined with the LFC theory, is applied to the frame depicted in Figure 2, whose beam-to-column joints and members have been tested in the past, under cyclic loads[11]. To reduce the number of variables a perfectly rigid connection is supposed at the base. Seismic mass is approximately 32 tons at each floor.

For the NLTH analysis two earthquakes have been considered: El Centro 1979, and Landers 1992 having a magnitude of 6.5 and 7.3, respectively. The finite element model (FEM) is developed in OpenSees[12] including both geometrical and mechanical non-linearities.

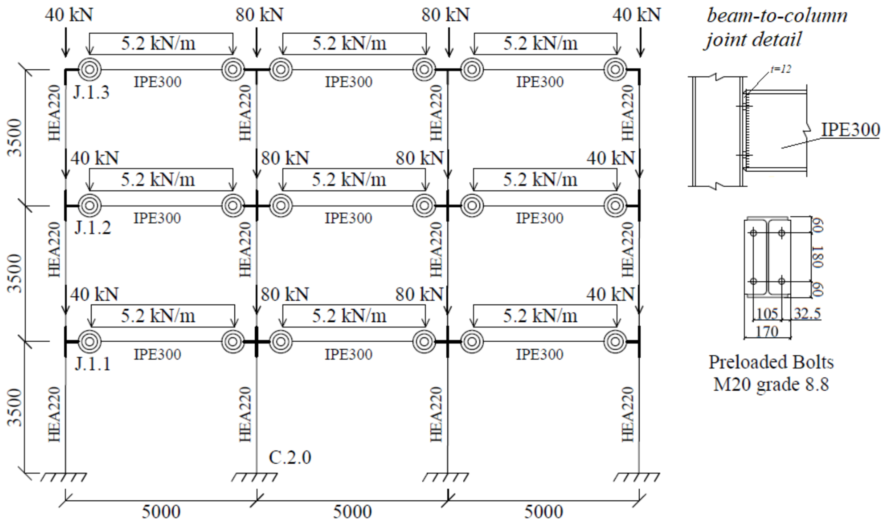

Figure 2: The considered MR steel frame

The initial joint stiffness has been assumed equal to 17625 kNm/rad for TSC and 27050 kNm/rad, obtained as a mean value of the initial stiffnesses experimentally observed[11]. It is worth noting that the bilinear Krawinkler model[13] has been assumed for the members while the cyclic joint behavior has been predicted by using the Pinching4 model[14]. It is well-known that there is a relevant dependence of the response obtained in non-linear analysis from the hysteretic law adopted in the dissipative zones and for this reason, both models have been suitably calibrated by using experimental results[11]. In Figure 3 the comparisons between experimental and numerical hysteretic laws are reported. In all the test maximum error between numerical and experimental dissipated energy is located at the first cycles of the test. The lowest error in energy dissipation is provided for HEA220 profile equal to 7%, while the highest errors are provided in joint with maximum up to 30%.

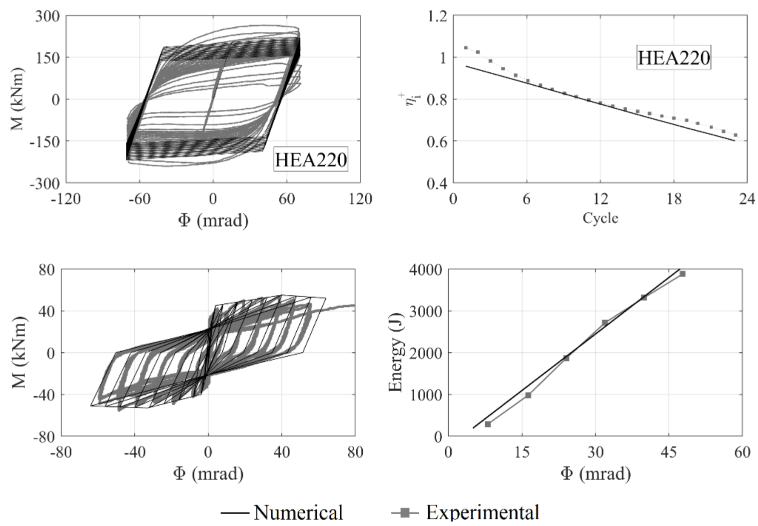

Figure 3: Calibration of the joint behavior by using experimental results[11]

The proposed procedure allows for the appraisal of the damage cumulation process in all the joints and the members of the frame, during the earthquake. As an example, Figure 4 can be considered where the damage cumulation is plotted for the base column section C.2.0 and the beam-to-column joints (J.1.1, J.1.2 and J.1.3 in Figure 2), after a single El Centro or Landers earthquake.

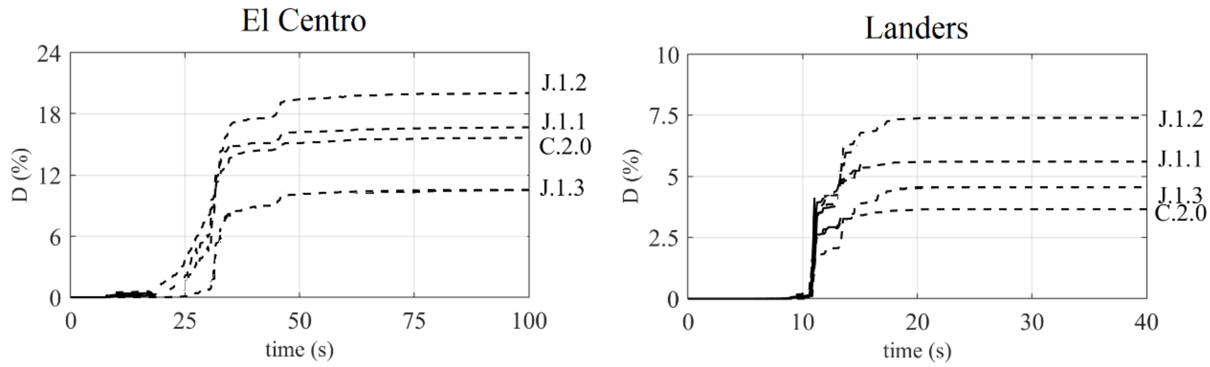

Figure 4: Damage cumulation (%) during the considered earthquakes

For both cases the highest damage index value is always located at second floor (J.1.2), while the lowest is at the top floor (J.1.3) or at the column base section (C.2.0). The uppermost deformations are provided in joints and the nethermost at the column bases because of the cyclic excursions in plastic range are more limited. Furthermore, it is worth noting that despite El Centro response is characterized by quite low rotation amplitude, the obtained damage level is not negligible (up to 20%). In fact, with El Centro earthquake the total number of cycles experienced from the dissipative zones is three times greater than the one associated with Landers ground motion.

To have a general overview of the earthquake effects, Figure 5 can be considered, presenting the damage indices of both joints and column ends, expressed in percentage and located on the proper frame position. Moreover, a refined representation of joints fatigue is also proposed in the log-log Whöler's plane. In this case, tha fatigue damage of each joint is represented by a point whose coordinates are the number of cycles and the equivalent rotation ($\Delta\Phi_{eq}$), that have been evaluated in the re-analysis of the NLTH MR frame data. It can be noted that:

- damage at columns are quite small, like the ones associated with joints at the top level;
- damage index in the joints is maximum at the second floor and minimum at the top and this is due to the shape of the first and second vibration modes;
- external joints are more highly damaged than the internal ones.

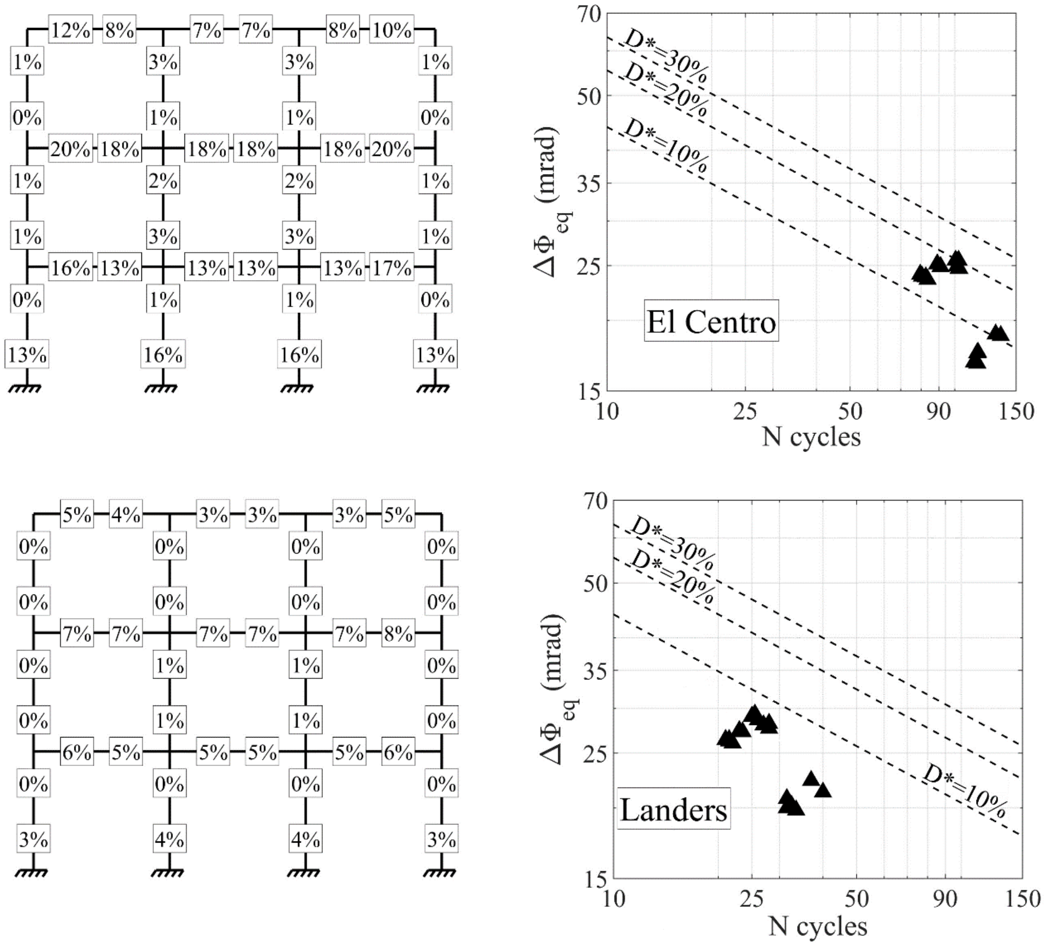

Figure 5: Damage values (%) and raperesentation in the Whöler domain

Another very important aspect for a safe use of the frame after the earthquake is the accurate appraisal of the residual load carrying capacity. The post-earthquake load carrying capacity of a structure is significantly influenced by the value of the interstorey drift $|\theta|$, that can be intended as the maximum value of the residual drift (θ_r) and/or as the maximum transient drift ($\theta_{t,max}$) reached during the earthquake. The interstorey drifts are directly dependent on the plastic deformation of the dissipative zones. When relevant transient drifts are measured, the dynamic stability of the frame is compromised.

In Figure 6 the absolute value of residual and transient instersory drift $|\theta|$ is reported over the height.

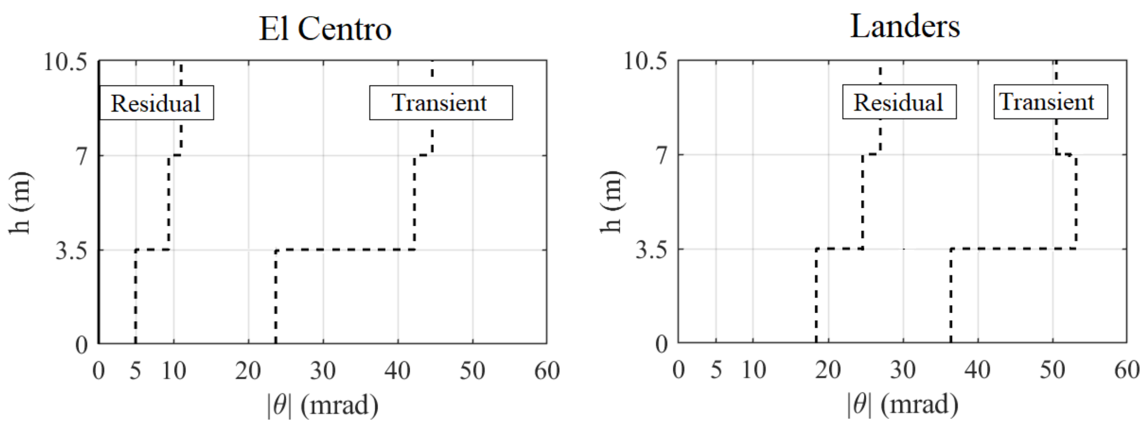

Figure 6: Residual and maximum transient drift distribution

The proposed procedure to assess post-earthquake frame behaviour can be applied also in case of additional earthquakes as briefly shown in the following, herein refering, for the sake of simplicity, to sequence of events, having the samer magnitude to the original one. From the numerical point of view, earthquake sequence has been obtained as a repetition of the same main shock signal, by inserting an interval with zero acceleration between each of the three repetitions that have been herein considered. Due to relevant reduction in load carrying capacity, structural failure due to Dynamic Instability (DI) is expected. Moreover, a direct correlation between the drift accumulation process under sequential earthquake and the residual properties of the structure against vertical load condition is expected.

Two different responses characterize the structural behaviour under sequential shocks (Figure 7), that can be classified as:

i) frame vibration around a deformed configuration obtained after the first earthquake. This can be defined as a “structural adaptation” of the frame to the earthquake and can be appreciated during the El centro earthquake;

ii) progressive increment of residual and trasient drift after each shock, with the extreme case on wich transient displacements are so high that the structure reach DI during the sesimic event (as in case of Landers repetitions).

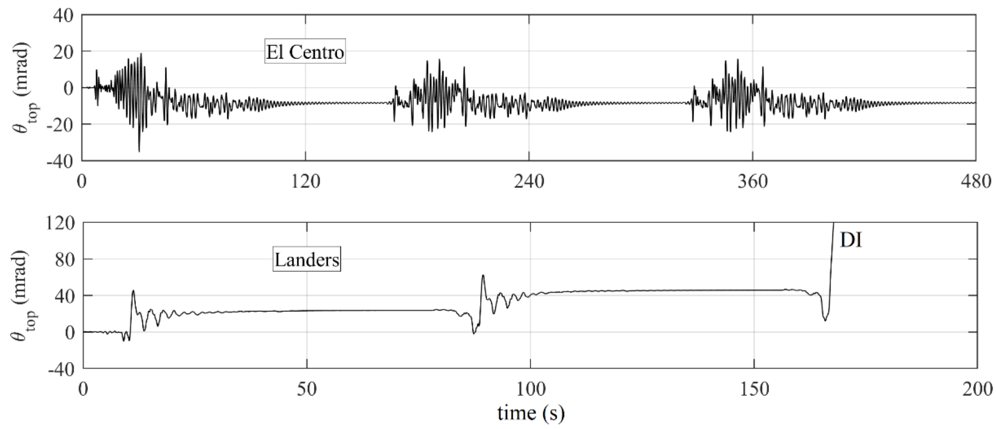

Figure 7: Top drift response under sequential shocks

In Figure 8, residual load carrying capacity of each sequence is presented in term of the ratio between residual (W_{res}) and intial load carrying capacity (W_{ini}).

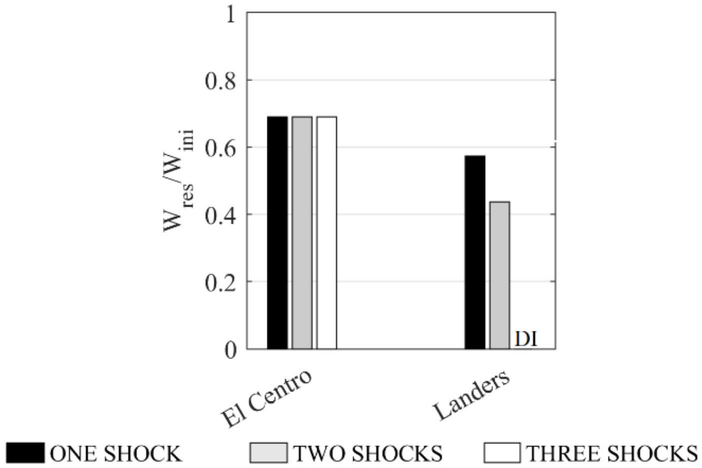

Figure 8: Residual load carrying capacity after sequential shocks

It can be noted that El Centro earthquake leads to the same reduction (that is of about 30%) considering one or more seismic sequences. On the contrary, if Landers sequences are considered, the reduction increases from 42%, in case of one earthquake, to 56% in case of two earthquakes; when three Landers earthquakes occur, the frame collapses, i.e. DI is reached. As an example of the joints damage cumulation process, Figure 9 can be considered, which is related to the relevant damage value in the Whöler domain. It can be highlighted that, even if in some case structural stabilization occurs (as happen in the case of El Centro earthquake), fatigue life of each dissipative zones is progressively reduced after each earthquake, with the increase of local damage indices. It can be remarked that if El Centro is considered, damage cumulation is not properly linear due to an additional vertical translation of the most damaged points during the second and the third earthquake.

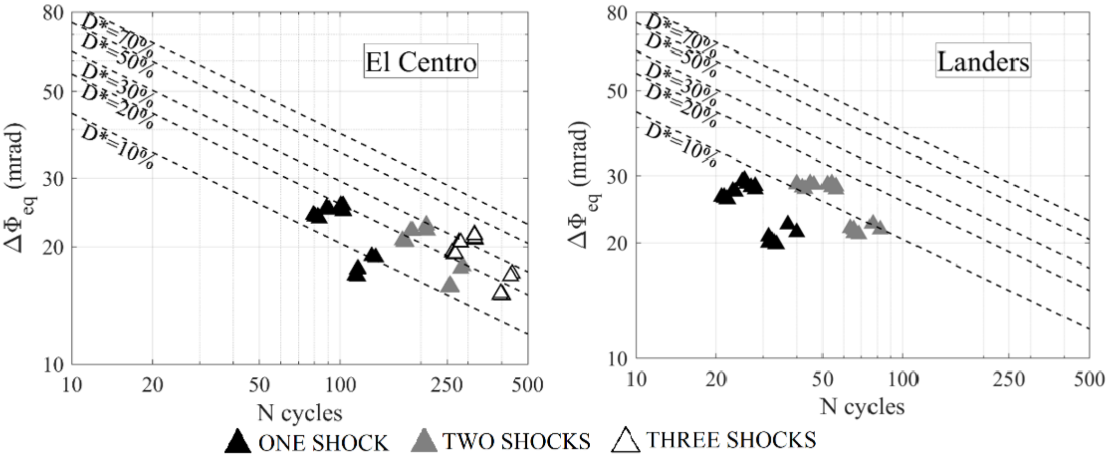

Figure 9: Joint damage in sequential earthquake

Finally, it can be concluded that enough post-earthquake performance can be appreciated, only in case El Centro earthquake. Even if local damage linearly increases after each earthquake, a stable trend is detected in the residual properties of the structure, leading sustainable further repair actions. On the contrary, the other case experiences relevant reduction of safety under further shocks, leading repair not sustainable. This aspect is highlighted also by the fact that, under the third Landers shock, DI is experienced.

4 CONCLUDING REMARKS

A numerical procedure combining NLTH analysis with LCF theory has been presented and applied for the evaluation of the residual performance of a MR steel frames damaged by one or more earthquakes. Results related to the considered frames shown that, in general:

- the post-earthquake load carrying capacity is greatly influenced by the residual drifts, that are strongly dependent from the hysteretic law and the deterioration mechanism of the dissipative zones, i.e. joints and members;
- the maximum damage values are always located at the external joints on the second floor levels, while the minimum ones are at the top floor or on the columns;
- after more earthquakes the damage cumulation process is characterized by a quite linear trend, depending mainly on how the frame modify its modal behavior.

Moreover, for both the selected earthquakes, after two sequential shocks, no failure occurs despite the observed high values of damage in joints, reflecting direcly in a non-negligible reduction of the load carrying capacity; structural collapse has been observed after three repetition of the Landers earthquake. This behaviour is attributable to dynamic instability problems and not to the exhaustion of the fatigue life of the steel components.

Furthermore, it can be stated that the proposed numerical procedure provides results useful in the definition of the intervention strategy on a damaged structure. Due to sensitivity of the procedure to the hysteretic law representing cyclic behavior of the MR frame key components, an accurate calibration of the numerical model is always required. The results obtained from the proposed procedure, should be validated with those detectable during a preliminary inspection phase (i.e. *in-situ* evaluation of the damage index and residual interstorey drifts).

Finally it is worth noting that, due to limited number of data provided in literature on LCF of steel components, extensive experimental campaigns are required for extending the application of the proposed procedure to a wider case of connections and steel members.

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