ASSESSING THE SEISMIC SAFETY OF EXISTING RC STRUCTURES USING THE EC8-PART 3 PROCEDURES

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ABSTRACT :

The present study is an application of the EC8-3 deterministic procedure for safety assessment. Based on the application of the EC8-3 deterministic procedure, the study aims to assess if the considered methods of analysis lead to similar safety results, identifying the factors that may affect the results. Moreover, the EC8-3 procedure application is complemented with a probabilistic approach to obtain fragility values corresponding to the deterministically assessed safety levels. By comparing deterministic and probabilistic results, the study assesses if similar deterministic results lead to similar probabilistic results. Furthermore, the study will try to establish a correlation between deterministic D/C ratios and the expected fragility values.

KEYWORDS: Eurocode 8 Part 3; existing structures; safety assessment; RC structures.

1. INTRODUCTION, OVERVIEW OF EC8-3 AND OBJECTIVES OF THE STUDY

Earthquake engineering experts, public authorities and general public alike agree on the idea that the assessment of the seismic safety and performance of the built environment is a matter of high priority. With the implementation of Part 3 of the Eurocode 8 (EC8-3) [1], codified provisions for the evaluation of the seismic performance of existing structures are now available. Although this document has been thoroughly checked for consistency, little comparative applications have been performed to date, with a recent exception found in [2].

An application of the EC8-3 deterministic procedure for safety assessment is therefore presented herein, and complemented using a probabilistic approach. Before addressing the performed application study, an overview of the EC8-3 assessment philosophy is presented, though for more detailed information the reader is referred to [1].

For a chosen performance requirement in terms of seismic safety, EC8-3 allows the state of a structure to be quantitatively evaluated by means of linear or nonlinear types of analyses. The verifications of the structural mechanisms depend on their ductile or brittle nature. The performance requirements are defined in terms of 3 Limit States (LSs) which are the Near-Collapse (NC), Significant Damage (SD) and Damage Limitation (DL) LSs. The LS of NC represents a situation close to structural collapse, while SD is equivalent to the Ultimate LS in new designs. The LS of DL corresponds to light structural damage without significant yield of the elements. The return periods of the design action indicated in EC8-3 as appropriate for the 3 LSs and for buildings of ordinary importance are 2475, 475 and 225 years.

A distinctive feature of existing structures when compared to new ones is that their structural properties may be known. In EC8-3, the global Knowledge Level (KL) is defined by the combination of the knowledge available or achieved in geometry, details and materials. EC8-3 defines 3 levels of knowledge denoted by KL1, KL2 and KL3, in increasing order of comprehensiveness, and defines a factor associated with each level, termed Confidence Factor (*CF*). The recommended values of the *CFs* are 1.35, 1.20 and 1.0, for KL1, KL2 and KL3, respectively.

In terms of analysis methods, options range from linear to nonlinear methods, either static or dynamic. For pushover analysis, there are no conditions of applicability related to structural regularity in elevation, while for an in plan non-regular building a spatial model is requested. EC8-3, by referring to EC8-1, requests that at least 2 lateral force patterns must be considered for pushover analysis: one uniform and one modal. Safety verifications are then carried out for the most unfavourable result. When nonlinear dynamic analysis (NDA) is selected, the major issues arise in terms of defining the seismic action. EC8-3 allows the consideration of artificial or recorded accelerograms, in a minimum of 3. Structural demand must be assessed for all accelerograms and member safety

verifications are then carried out for the most unfavourable result. When at least 7 accelerograms are considered, safety verifications can be carried out for mean demand. Besides defining the number of required accelerograms, EC8-3, by referring to rules in EC8-1, also specifies spectral matching conditions they should comply with.
From the safety assessment stage point of view, when a nonlinear method of analysis is used, demand for both ductile and brittle mechanisms is directly obtained from the analysis (to be carried out using mean mechanical property values). On the capacity side, ductile mechanisms are checked in terms of deformations and the values of the capacities for the different LSs are obtained from given expressions computed using mean values of the mechanical properties divided by the *CF*. Brittle mechanisms are checked in terms of strength, and the values of the capacities are obtained from given expressions computed using mean values of the mechanical properties divided by both the usual partial safety factor and the *CF*.
The study presented in the following corresponds to an application of the EC8-3 deterministic procedure for safety assessment. Based on this application, the study aims to assess if the considered methods of analysis lead to similar safety results and identify the factors that may affect these results. To reach these objectives, the deterministic application of the EC8-3 procedure is complemented with a probabilistic approach to obtain the fragility values corresponding to the deterministically assessed safety levels. By comparing the deterministic and the probabilistic results, the study aims to assess if similar deterministic results (D/C ratios) lead to similar probabilistic results (fragility values). Furthermore, the study will try to determine if a correlation can be established between deterministic D/C ratios and the expected fragility values.

2. GENERAL DATA AND METHODS CONSIDERED FOR THE DETERMINISTIC ASSESSMENT

The EC8-3 deterministic procedure was applied for the safety assessment of two reinforced concrete one-bay-four-storeys planar frame structures of similar geometry. The seismic safety of the selected structures was assessed for both deformation (ductile) and strength (brittle) based LSs. In the former, the selected demand parameter was the member chord rotation, while in the latter, demand was assessed in terms of shear force. In terms of deformation demand, the 3 previously referred LSs (DL, SD and NC) were considered while in terms of force demand only the NC LS was selected, as defined in EC8-3. For each LS, the 3 EC8-3 previously referred KL conditions were also considered for safety assessment. Full Knowledge conditions, corresponding to KL3, were assumed to be defined by mean property values of the selected material classes. Knowledge conditions for KL1 and KL2 were assumed to be the KL3 conditions divided by the corresponding *CFs*.
Safety assessment for each LS and KL combination was performed using pushover analysis and NDA. Although, according to EC8-3, KL1 conditions can only be considered with linear analysis, they were also considered with nonlinear analysis to obtain a more comprehensive view of the influence of the *CFs* on the assessment results.

2.1 Structural configuration the selected structures, numerical modelling and definition of seismic demand

The selected reinforced concrete frames for this application were considered to be interior frames of a larger building and have structural characteristics aiming to simulate non-seismic design situations. The two buildings, and corresponding selected frames hereon termed *TF1* and *TF2*, differ only on the orientation of the column cross sections. Columns of frame *TF1* have a gross section of 0.25×0.50 m^2 while those from frame *TF2* have a gross section of 0.50×0.25 m^2. These characteristics and the remaining geometrical and detailing data are presented in Fig. 1. According to the selected material classes, the mean concrete compressive strength f_c was set as *28 MPa*, the mean ultimate concrete strain as *0.006*, the mean longitudinal and transversal steel yield strengths, f_y and f_{yw}, as *440 MPa*, the mean ultimate steel strength as *506 MPa* and the mean ultimate steel strain as *0.09*.
For the sake of simplicity, the effect of the lightweight slab-width on the beam stiffness and strength was not considered in the numerical modelling. According to EC8-3, when carrying out the safety assessment for the LS of DL in terms of deformation, structural demand must be obtained from the analysis of a numerical model where the stiffness of the members is taken equal to the mean value of M_yL_v/θ_y at the two ends of the member, where M_y is the yield moment of the member, L_V is the shear-span (that may be taken as half of the member length) and θ_y is the yield chord rotation. The latter is defined by the yield capacity that is presented in Section 2.3.
The necessary data for the development of the numerical model of the frames for nonlinear analysis, static or dynamic, depends on the analysis program that is used. For the present study, the analysis program presented in [4] for the application of the NSA method. Therefore, the same modelling approach was considered, in which structural elements are modelled as member-type nonlinear macro-models with 3 zones: one internal zone with linear elastic behaviour and plastic hinges, located at the member ends, where inelastic flexural behaviour is con-

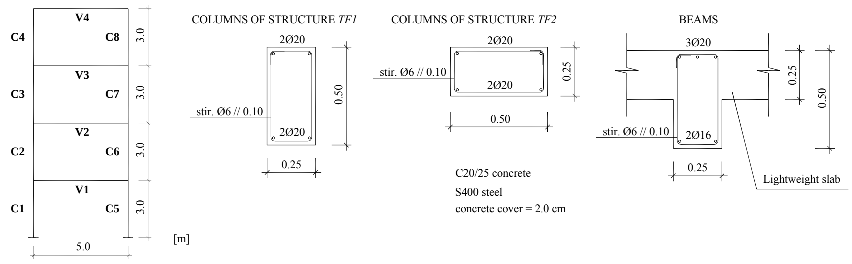

Figure 1. Geometrical description and detailing of frames *TF1* and *TF2*, and numbering of the members.

sidered. Inelastic flexural behaviour of the members was modelled by the piecewise linear hysteretic Costa-Costa model [5] which represents a generalization of the original Takeda model [5]. The model incorporates a trilinear monotonic envelope defined by cracking and yield points, and is able to include pinching effects, stiffness degradation, strength deterioration and non-symmetric response.

Damping was only considered for the analyses involving low intensity seismic actions of the DL LS. In such cases damping was assumed to be of Rayleigh type with a critical damping equal to 3% for the first and second mode periods. Periods were obtained assuming the mass to be distributed on the beams. The vertical loading considered consists in uniform loads of 30.16kN/m on the first, second and third storeys and of 26.56kN/m in the fourth storey. These represent the self-weight of the beams and of the slabs, the finishes and the quasi-permanent value of the live load. In addition, a set of concentrated loads were considered to represent the self-weight of the columns. Since nonlinear behaviour is expected to develop at the structural member ends, the beam reinforcement defined in Fig. 1 is that of the end zones only. Each structural member, defined according to Fig. 1 has, therefore, two demand control sections located at each end and termed *bot* and *top*, in columns, and *left* and *right*, in beams.

Seismic demand was set for Zone 1 of the Italian territory and considering a soil of type B. According to [4] the PGAs for the different LSs considered are 0.14g, 0.35g and 0.525g for the LSs of DL, SD and NC, respectively.

When pushover analysis was used, the effective seismic demand was characterized by a set of target displacements defined for each PGA value and for each force pattern. In this study, safety assessment of the frames was performed using the uniform force pattern and the modal force pattern proposed in EC8-1. In the cases where NDA was used, 3 different sets of accelerograms were defined to set the effective seismic demand.

2.2 Definition of the accelerograms for nonlinear dynamic analysis

Of the 3 considered sets of records, the comprises 7 artificial spectrum-compatible accelerograms with 15 seconds for each LS; the second set is made of 7 real ground motion records scaled for the PGA of each LS and the third set is made of the same 7 real ground motion records now scaled for the 5% damping spectral acceleration value at the fundamental period ($S_a(T_f)$) for each LS. The artificial accelerograms were computed in order to meet the spectral matching requirements defined by EC8-3. On the other hand, recorded ground motions were chosen from an existing larger set of records [4] in order to have moment magnitudes between 5.3 and 5.7 and epicentral distances between 15 km and 30 km. Fig. 2a) shows the response spectra of the 7 artificial accelerograms for the LS of SD and their average response spectrum against the EC8-1 elastic response spectrum with ±10% bounding limits. Fig. 2b) presents similar information now for the real ground motions when scaled using the PGA and for the LS of DL. Similar information is also presented for real ground motions scaled using the $S_a(T_f)$ and for the LS of NC in Fig. 2c) and d) for frames *TF1* (T_f = 0.46sec) and *TF2* (T_f = 0.65sec). Observation of these figures shows that

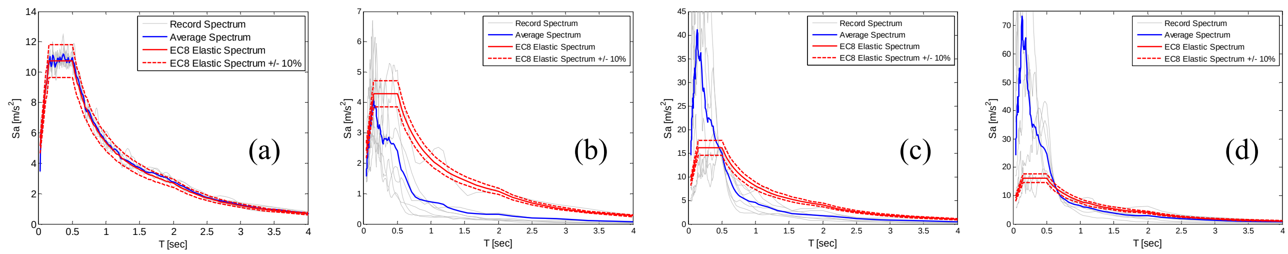

Figure 2. Response spectra of the artificial records for the LS of SD (a); of the PGA scaled records for the LS of DL (b); of the $S_a(T_f)$ scaled records for the LS of NC for frame *TF1* (c) and frame *TF2* (d), their average spectrum and the EC8 elastic response spectrum +/- 10%.

artificial accelerograms meet the EC8-3 specifications in terms of spectrum matching, while the real ground motions chosen based on moment magnitudes and epicentral distances do not. Nonetheless, the response spectra of the $S_a(T_f)$ scaled records seem to be closer to the code spectrum. Based on these observations, the real records scaled by both procedures are expected to underestimate the response when compared to the artificial ones.

2.3 Capacity models for the selected limit states

EC8-3 defines the member-level capacities for ductile and brittle mechanisms to be used in the safety assessment verifications for the several LSs. Ductile capacities are defined in terms of the admissible DL, SD and NC member chord-rotations while brittle capacities are characterized by the admissible NC shear force.
The NC chord-rotation capacity θ_{NC} was defined using the expression [1]:

$$\theta_{NC} = \frac{1}{\gamma_{el}} \cdot 0.016 \cdot 0.3^{\upsilon} \cdot \left(\frac{\max(0.01,\omega')}{\max(0.01,\omega)} f_c \right)^{0.2} \cdot \left(\frac{L_s}{h} \right)^{0.35} \cdot 25^{\alpha \cdot \rho_{sx} \cdot \frac{f_{yw}}{f_c}} \cdot 1.25^{100\rho_d} \tag{1}$$

where γ_{el} is 1.5 for primary members and 1.0 for secondary ones, v is the normalized axial force, ω and ω' are the mechanical reinforcement ratios of the tension and compression, respectively, longitudinal reinforcement, $\rho_{sx} = A_{sx}/b_w s_h$ is the ratio of transverse steel area A_{sx} parallel to the direction x of loading (s_h is the stirrup spacing and b_w is the cross section width), ρ_d is the steel ratio of diagonal reinforcement (if any), in each diagonal direction, L_s is the shear span taken constant and equal to half of the member length, h is the cross section depth, and α is the confinement effectiveness factor [3]. As stated in EC8-3, the SD chord-rotation capacity θ_{SD} was defined as 75% of θ_{NC}. In the case of the DL chord-rotation capacity θ_{DL}, and assuming that no shear cracking is expected to precede flexural yielding, the chosen expression was [1]:

$$\theta_{DL} = \phi_y \cdot \frac{L_s}{3} + 0.0013 \cdot \left(1 + 1.5 \frac{h}{L_s} \right) + 0.13 \cdot \phi_y \cdot \frac{d_b \cdot f_y}{\sqrt{f_c}} \tag{2}$$

where ϕ_y is the yield curvature of the member end section and a_b is the mean diameter of the tension reinforcement. Due to the asymmetry of the beams longitudinal reinforcement, chord rotation capacities were computed for both bending signs. According to EC8-3, shear force capacity V_{NC} for the LS of NC was set by [1]:

$$\begin{gathered} V_{NC} = \frac{1}{\gamma_{el}} \left[\frac{h-x}{2 \cdot L_s} \cdot \min(N; 0.55 \cdot A_c \cdot f_c) + \left(1 + 0.05 \cdot \min\left(5; \mu_{\Delta}^{pl}\right)\right) \cdot \right. \\ \left. \cdot \left[0.16 \cdot \max(0.5; 100 \cdot \rho_{tot}) \cdot \left(1 - 0.16 \cdot \min\left(5; \frac{L_s}{h}\right) \right) \cdot \sqrt{f_c} \cdot A_c + \rho_w \cdot b_w \cdot z \cdot f_{yw} \right] \right] \end{gathered} \tag{3}$$

where x is the compression zone depth, N is the compressive axial force (equal to zero for tension), A_c is the cross section area taken equal to $b_w.d$ (d is the structural depth), ρ_{tot} is the total longitudinal reinforcement ratio, μ_{Δ}^{pl} is the ratio between the plastic part of the chord rotation demand and the yield chord rotation given by Eq. (2), ρ_w is the transverse reinforcement ratio and z is the length of the internal lever arm. Furthermore it is noted that N was taken as the member axial force under gravity loads, as suggested in [2], and the term *(h - x)*, representing the distance between the member compression centres, was assumed equal to *2h/3*.

3. DEFINITION OF ADDITIONAL DATA FOR PROBABILISTIC ASSESSMENT

Probabilistic safety assessment of the selected structures was carried out to obtain fragility values corresponding to the deterministically assessed safety levels. Fragility values were computed using NDA results only. Seismic demand was considered to be defined by the same groups of accelerograms used in the deterministic assessment. Three groups of results were therefore obtained for each LS and each KL: results obtained using artificial spectrum-compatible accelerograms, using real ground motion records scaled for the PGA and using real ground motion records now scaled for the 5% damping $S_a(T_f)$. Based on these results, the probabilistic demand estimation due to ground motion record-to-record variability was able to be represented by lognormal distribution functions fitted to the demand values of each LS and KL, using the maximum likelihood estimation method. From the

results, it was seen that, in general, the lognormal CDF adequately fits the demand data. As expected, the 5% damping $S_a(T_f)$ scaled real records were also seen to yield chord rotation demands with larger variability than the artificial records, as the ground motion intensity increases. In terms of shear force demand, the influence of the type of record is less evident.
Randomness of the material properties was not considered in the probabilistic characterization of the demand. However, randomness of the concrete compressive strength and yield steel strength was considered for the definition of the probabilistic distributions of the LS capacities. Assuming that these properties follow normal distributions with a given mean μ and coefficient of variation (COV), the following values were set for both structures: concrete compressive strength with mean $\mu_{f_c} = 28\,MPa$ and $COV_{f_c} = 0.18$, and yield steel strength with mean $\mu_{f_y} = 440\,MPa$ and $COV_{f_y} = 0.06$. Considering the capacity models presented in Section 2.4, LS capacities were simulated for each KL, using 200 values of the material properties f_c and f_y sampled from their probabilistic distributions and combined using the Latin Hypercube Sampling method. Normal and lognormal distribution functions were then fitted using the maximum likelihood estimation method. Fitting results indicated that both normal and lognormal distributions adequately fit the computed capacities of the several LSs.

4. SEISMIC SAFETY ASSESSMENT RESULTS FROM THE DETERMINISTIC APPROACHES

Seismic safety assessment results obtained from the different methods of analysis (pushover and NDA using different types of ground motions), different LSs and different KLs are presented in the following. For a given control section i, results are expressed in terms of D_i/C_i ratios where a value below or equal to 1.0 represents a safe situation. The presentation of the results is initially divided according to the method of analysis followed by a comparative assessment of the different approaches. For conciseness sake, only a few sample figures of the results are presented herein. In these figures, the chord rotation LSs of DL, SD and NC are simply termed DL, SD and NC while the shear force LS of NC is simply termed V.
In terms of results obtained from pushover analyses, those presented for a given LS and KL, expressed in terms of D/C ratios, correspond to the most unfavourable results between the several patterns and loading directions.
Fig. 3 a) presents the D/C results for the control sections of frame *TF1* for the 3 chord rotation LSs and considering KL3 while Fig. 3 b) presents the D/C results only for the NC chord rotation LS but considering 3 KLs. Fig. 3 c) presents the D/C results only for the NC shear force LS but considering 3 KLs. Observation of the D/C results enabled to conclude that frame *TF2* is less safe than frame *TF1* due to its higher flexibility, which was induced by changing the orientation of the columns, and that DL seems to be the dominant deformation LS. It can also be seen that the influence of the KL is considerably different for chord rotation and shear force capacities, the latter being more sensitive to the different KLs.
The presented results obtained from NDA intend to provide a general overview of the effect of the type of accelerogram used for the analyses on the safety assessment results. Since 7 ground motions were considered for all 3 types of records, the D/C ratios presented are mean values over the 7 results obtained for each record set.
Figs. 4a) and b) present the D/C results for the control sections of frame *TF1* considering the 3 types of accelerograms for the DL and SD chord rotation LSs and considering KL3. Figs. 4c) and d) present the same type of results for the control sections of frame *TF2*, now for the DL and NC chord rotation LSs. Figs. 4e) and f) present the results for the shear force NC LS for frames *TF1* and *TF2* considering the 3 types of accelerograms.
Observation of these results leads to conclude that the type of accelerogram has a considerable influence on the deformation assessment results, especially for frame *TF1*. Such differences result from the differences observed between the real records response spectra and the code spectrum. As previously noted, PGA scaled records produce response spectra that are more distant from the code spectrum than those scaled using $S_a(T_f)$. This can also be observed in Figs. 4 a) to d) as results of the PGA scaled records are, on average, the more distant ones from the results obtained from artificial accelerograms (which are considered as the reference demand since they match the code spectrum). On the other hand, the V assessment results are seen to be much less sensitive to the record type.
In terms of results based on PGA scaled records, to improve their agreement with those obtained with artificial accelerograms choices are either to choose a different set of records meeting the spectrum matching conditions defined by EC8-3, or numerically alter the selected records (e.g. using wavelets). On the other hand, to improve the results of the $S_a(T_f)$ scaled records for the SD and NC LSs, an alternative scaling period termed T_{inel} was used instead. This period was considered to be representative of the 1st mode inelastic period of the structure and was obtained from the effective period T^* computed from the pushover analysis results. For *TF1* the T_{inel}/T_f ratio is 0.72/0.46 = 1.6 while for *TF2* it is 1.2/0.65 = 1.8. Results of Figs 4 b) and d) include assessment results from the real records scaled for the 5% damping $S_a(T_{inel})$. On average, agreement between results of the artificial accelero-

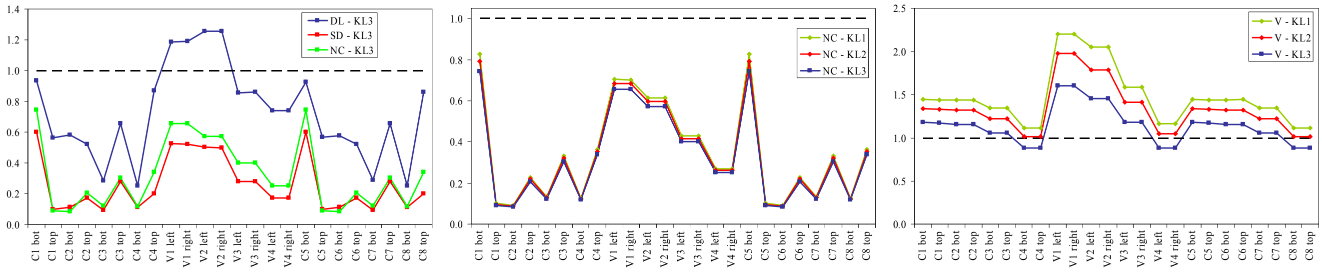

Figure 3. Safety assessment results of frame *TF1* for (a) the 3 chord rotation LSs and KL3; for (b) the NC LS considering 3 KLs and for (c) the V LS of frame *TF2* considering 3 KLs.

grams and those from this new scaling approach improves. In terms of seismic demand definition and in case the spectrum matching conditions proposed by EC8-3 are not met, these results suggest that it is possible to improve the assessment procedure for deformation based LSs for which considerable nonlinear behaviour is foreseen. This improvement comes by selecting a new period T_{inel} ranging between $1.5T_f$ and $2T_f$, instead of T_f, to obtain the spectral acceleration $S_a(T_{inel})$ for which real records should be scaled.

The comparative evaluation of the different analysis methods aims to verify if similar assessment results are obtained, thus validating the use of pushover analysis. Figs. 4 a) to f) also include assessment results based on pushover analysis. Observation of these results leads to conclude that for deformation based LSs, correlation between pushover and NDA results is best when considering artificial records. Nonetheless, there are some differences for low intensity seismic demand such as that of the DL LS, namely for the beams of frame *TF2*. When considering real records, agreement is best when considering the alternative scaling method previously suggested. With respect to the lack of agreement when using PGA scaled records, comments made in the previous Section are applicable. In terms of the shear force NC LS, agreement between NDA and pushover results is much better. Nonetheless, artificial accelerograms and $S_a(T_f)$ scaled records yield slightly larger assessment ratios in beams.

In the overall, with the exception of some control sections for the shear force NC LS and for the deformation LSs when considering, either the proposed alternative scaling method or the artificial accelerograms, pushover results can be seen to be on the safe side. In addition, it should be noted that selection of real records based on moment magnitudes and epicentral distances criteria does not appear to be an adequate approach for safety assessment based on EC8-3. Also, when using NDA with real records that do not meet the EC8-3spectrum matching conditions, a larger record-to-record variability of the results is expected. In such case, it should be emphasized that there is no clear justification for the use of 7 records only and that the ground motion scaling method plays a fundamental role. In such cases, considering the mean of the response may lead to unsafe demand estimations as the mean is a poor central tendency estimator, given its high sensitivity to variations in the data values.

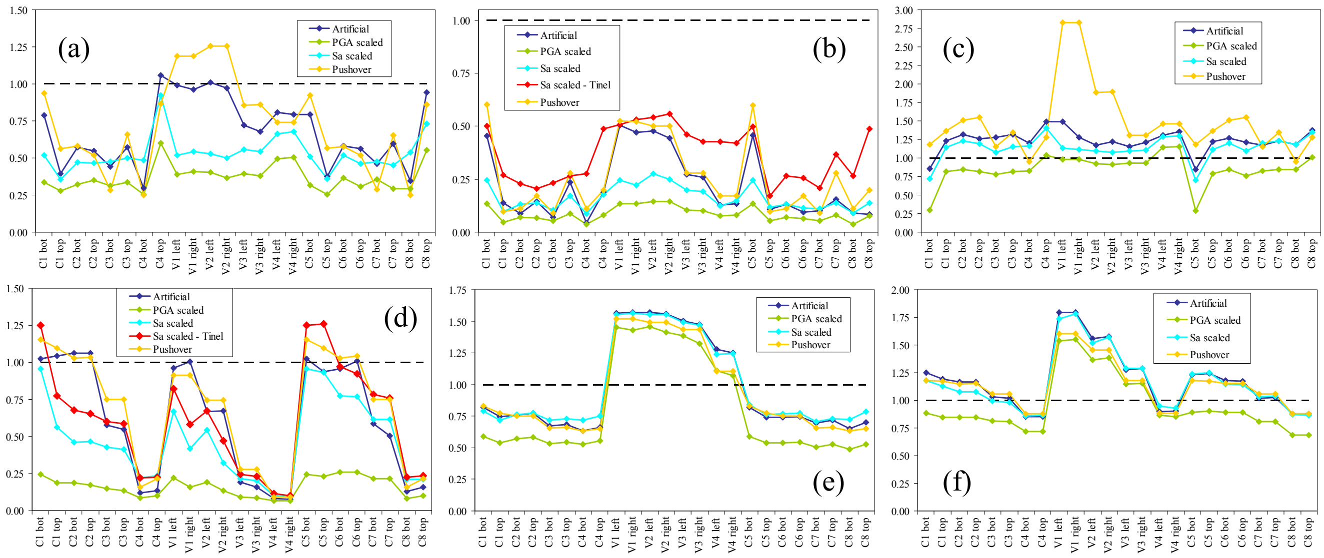

Figure 4. Assessment results considering pushover analysis, NDA with 3 types of accelerograms and KL3 of frame *TF1* for (a) the DL LS and for (b) the SD LS including $S_a(T_{inel})$ scaled records; of frame *TF2* for (c) the DL LS and for (d) the NC LS including $S_a(T_{inel})$ scaled records; for (e) frame *TF1* and (f) frame *TF2* for the V LS.

Finally, a brief comment is made with respect to the influence of the KL in the assessment results. Since demand is based on the analysis results, the effect of the KL is only felt on the capacity side. After evaluating the several LS capacities for the different KLs and for all the members of both frames, and averaging the increase in capacity that is gained by moving from one KL to another, the values presented in Table 1 were obtained. These allow for a global view of the influence of the KL. As can be seen, from the practical point of view, going from KL1 to KL2 or from KL2 to KL3 produces limited changes in the capacity values. Thus, the need for an increase in knowledge about the materials must be carefully thought out due to the increase in work and costs that may be implied.

Table 1 – Increase in capacity of the several LSs by increasing the KL

	DL	SD	NC	V
KL1 to KL2	8%	3%	3%	10%
KL2 to KL3	14%	5%	5%	17%
KL1 to KL3	24%	9%	9%	29%

5. SEISMIC SAFETY ASSESSMENT RESULTS FROM THE PROBABILISTIC APPROACH

Based on the probabilistic demand and LS capacity characterization presented in Section 4, fragility values for each control section, LS and KL combination were obtained by standard demand-capacity convolution, considering NDA results only. Although direct comparison with deterministic results is not possible, they were still compared to assess if similar D/C ratios lead to similar fragility values. This comparison enabled to determine if a correlation could be established between D/C ratios and the expected fragilities.

For conciseness sake, only a sample of the comparison results is presented. Figs. 5 a) and b) show the comparison between fragility values and deterministic D/C ratios for the DL chord rotation and the NC V LSs of frame *TF1*, considering the 3 KLs and $S_a(T_f)$ scaled records. Fig. 5 c) shows the comparison between fragility values and deterministic D/C ratios for the SD and NC chord rotation LSs of frame *TF2*, considering the 3 KLs and artificial accelerograms. For easier comparison, values of the D/C ratios above *1.0* were set to *1.0* in both figures. The presented fragility values were computed using the fitted lognormal LS capacity distributions. Negligible differences were obtained when considering normally distributed capacities.

Observation of the full range of the obtained results leads to conclude that there is a considerable variability of the fragility values for similar deterministic D/C ratios. This variability was found to be dependent of the LS, the type of demand (chord rotation or shear force) and on the type accelerogram. Nonetheless, the overall results still allowed for the definition of estimated ranges for the expected fragility values, given a set of ranges of the deterministic D/C ratios. These expected fragility ranges are presented in Table 2.

Observation of the proposed fragility ranges shows that, with the exception of the last range, no lower bound is

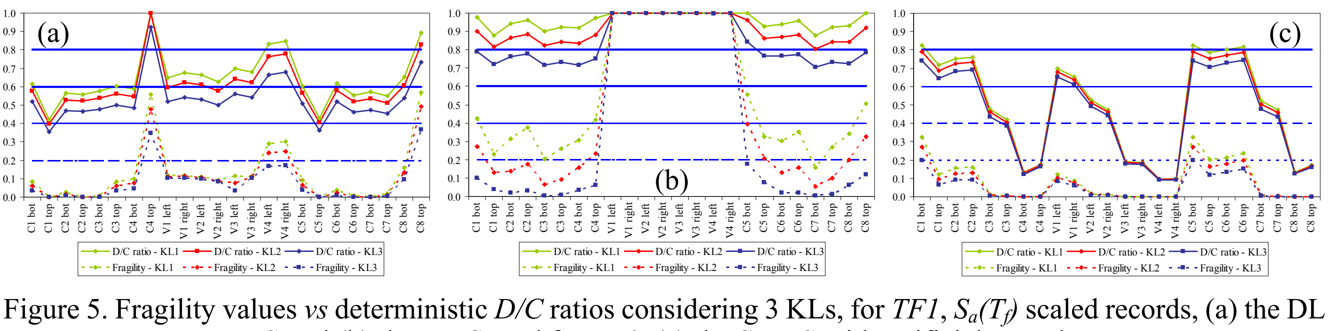

Figure 5. Fragility values *vs* deterministic *D/C* ratios considering 3 KLs, for *TF1*, $S_a(T_f)$ scaled records, (a) the DL LS and (b) the V LS; and for *TF2*, (c) the SD LS with artificial records.

Table 2 – Fragility estimated ranges based on deterministic D/C ranges

D/C range	Fragility *f* range	*D/C* range	Fragility *f* range
$D/C < 0.2$	$f \approx 0$	$0.6 < D/C < 0.8$	$f < 30\%$
$0.2 < D/C < 0.4$	$f < 5\%$	$0.8 < D/C < 1.0$	$f < 50\%$
$0.4 < D/C < 0.6$	$f < 15\%$	*D/C* much larger than 1.0	$f > 50\%$

generally proposed. This is due to the referred variability of the fragility values for similar deterministic D/C ratios. The fragility ranges may thus be seen as maximum upper bounds and, for a given D/C range, do not eliminate the possibility of obtaining a much lower fragility. Due to the lack of lower bounds, the proposed ranges also indicate that as the D/C ratio increases, the variability of the fragility values is also expected to increase.

6. CONCLUSIONS AND FINAL OBSERVATIONS

As little comparative applications address the assessment and validation of the EC8-3 code deterministic procedures for seismic safety assessment, an application of such procedures was presented herein, and complemented using a probabilistic approach. The study addressed the application of the deterministic procedure for the safety assessment of two reinforced concrete one-bay-four-storeys planar frame structures of similar geometry. The seismic safety of the structures was assessed for both deformation and strength based LSs. For each LS, the 3 EC8-3 KL conditions were also considered for safety assessment. Safety assessment for each LS and KL combination was performed using pushover and NDA. Results of the deterministic assessment lead to conclude that DL seems to be the dominant deformation LS. The influence of the KL was also seen to be considerably different for chord rotation and shear force capacities, the latter being more sensitive to the different KLs.
With respect to NDA results, these lead to conclude that the type of accelerogram has a considerable influence on the deformation assessment results. Such differences are a direct result of the differences observed between the real records response spectra and the code spectrum. On the other hand, the shear force assessment results are seen to be much less sensitive to the record type. To improve the results obtained from real records scaled for spectral acceleration ordinates, for the SD and NC deformation LSs, an alternative scaling period was considered, providing a better agreement between results of the artificial and real records. Comparative assessment between pushover and NDA safety assessment results leads to conclude that for deformation based LSs, correlation between pushover and NDA is best when considering artificial records. When considering real records, agreement is best when considering the alternative scaling period previously referred. In terms of the shear force NC LS, agreement between NDA and pushover results is much better, irrespective of the ground motion type. In the overall, with the exception of some sections, pushover results are on the safe side, when compared to NDA results.
In terms of the KL influence on the assessment results, it was found that going from KL1 to KL2 or from KL2 to KL3 produces limited changes in the capacity values. Therefore, the need for an increase in knowledge about the materials must be carefully thought out due to the increase in work and costs that may be implied.
In terms of the probabilistic approach, observation of the obtained fragility values for the several LSs and KLs shows that there is a considerable variability of the fragility values for similar deterministic D/C ratios. This variability was found to be dependent of the LS, the type of demand (chord rotation or shear force) and on the type accelerogram. Nonetheless, the overall results still allowed for the definition of estimated ranges for the expected fragility values, given a set of ranges of the deterministic D/C ratios.

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