REVIEW OF CONFIDENCE FACTOR IN EC8-PART3: A EUROPEAN CODE FOR SEISMIC ASSESSMENT OF EXISTING BUILDINGS

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Abstract: The built environment, both historic and of recent construction, is exposed to high level of seismic risk due to increasing level of seismicity around the world. Thus, it is necessary to assess the existing building performance to current level of seismicity in order to perform the cost-effective interventions. EC8-Part 3 is devoted to seismic assessment/retrofitting of existing buildings. This document introduces an adjustment factor to account for epistemic uncertainty, called “confidence factor (CF)”. CF is based on the level of knowledge of the structural properties such as geometry, reinforcement layout and detailing, and materials. This solution, plausible from a logical point of view, cannot yet profit from the experience of use in practice, hence its soundness needs to be investigated in real applications. This paper proposes a probabilistic based method to calibrate CF, which can simulate the entire assessment procedure conditional on the acquired knowledge. The method is then applied to six-storey three-bay reinforced concrete frame to assess the role of CF. The obtained CF's values are then critically examined and compared with code-specified ones. The critical problems of using a single factor in seismic assessment of existing buildings are pinpointed.

Keywords: Reinforced concrete, Testing and Inspection, Reinforcement details, Material properties, Modelling assumptions, Analysis method, Knowledge level

1. Introduction

A detailed seismic assessment (or evaluation) of an individual building is required to determine the need for seismic retrofitting, and also to identify the particular weaknesses and deficiencies to be corrected. For this reason, during the past two decades considerable work has been done in the direction of developing seismic assessment methodologies, usually under the auspices of national or international organizations. The first document aligned with the modern anti-seismic philosophy can be considered to be the NEHRP guidelines, prepared in 1997 under the sponsorship of the FEMA (FEMA, 1997), followed in 2000 by the FEMA 356 (FEMA, 2000). In the same years work started on Eurocode 8 Part 3 which was finally approved in 2005 (CEN, 2005). Ofcourse, it could not be asked of these documents to provide a knowledge that did not exist and, given the relatively short period during which they were developed, it could also not be expected that they were validated through a sufficiently long experience of application. As a result, they should still be looked at as experimental and subject to further progress.

This paper focuses on one particular aspect of the assessment procedure put forward in EC8-3: the so-called confidence factor (CF), analogous to the knowledge factor in FEMA 356. Actually, the role of this factor is central in the context of the overall procedure. The paper discusses the inadequacy of the present format due the associated uncertainties in structural properties as well as the degree of freedom left to the analyst. Next, the proposed rational calibration procedure is explained and applied to the six-storey three-bay RC frame. The results seem to indicate that the CF format currently specified in the code requires modification.

2. Assessment procedure in EC8-3 and role of CF

The assessment procedure starts with the information base initially available about the existing structure. From this point on, at each step of the procedure given in EC8-3, analysts are faced with a number of options that, as it will be discussed below (with reference to Fig. 1), cannot but lead to different outcomes of the state of the structure.

First of all, the analysts may choose to attain three different knowledge levels, for which different minimum amounts of tests are required by the code (Fig. 1a). For the same target KL, the same percentage of tests per floor may be obtained with different test types and locations (Fig. 1b). Each test type involves a different measurement error and, for indirect tests, a different dispersion in the associated correlation equation. Further, once the results have been collected, these have to be integrated with the initial data set (Fig. 1c): what to do then if the additional information contradicts the design documents? One analyst might accept the discrepancy, within certain limits, while another may choose to rely entirely on in-situ information adopting a full survey, together with extended test/inspection plans, i.e. moving up in the knowledge scale to KL2. Another issue is related to the two higher levels, KL2 and KL3, for which the two options: “initial information plus verification” and “complete reliance on in-situ information” are given as equivalent alternatives. It is quite likely that they are not exactly equivalent and this represents one further source of difference in the final assessment results.

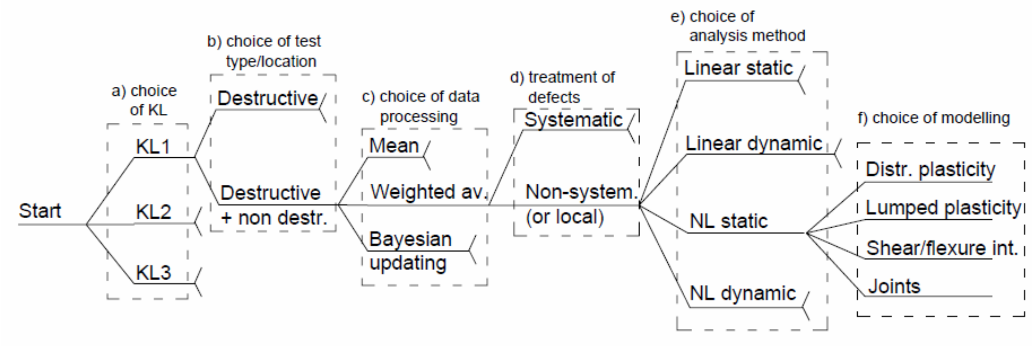

Figure 5. *Degrees of freedom left to the analyst*

The next branching point has to do with the so-called defective details, such as, for example, insufficient anchorage length of rebars, 90° hooks and inadequate diameter/spacing in stirrups, absence of joint reinforcement or wrong detailing of the anchorage of longitudinal bars into the joint, etc. This is a multi-faceted problem. Once a type of defect is discovered, the question arises whether its presence should be considered systematic over the structure or a portion of it only, or as an isolated local feature (Fig. 1d). An informed answer to this question would require extensive and intrusive investigations that are seldom compatible with the continued use of a building.

The next choice of the analysts is the method of analysis to be employed (Fig. 1e), which is intimately related to that of the modelling (Fig. 1f). Obviously, if the selected analysis is linear, the cyclic degradation due to defects cannot be included at this stage. But even if it is nonlinear static, such behaviour cannot be easily included. Exclusion of these defects from modelling may lead to a response quite different from the real one. On the other hand, nonlinear dynamic analysis including behavioural models for defective members would trade the model uncertainty on capacity with that on hysteretic degrading response.

The different sources of uncertainty and multiple choices facing the analysts during the assessment, all contribute to a relatively large dispersion in the estimated state of the structure. The interpretation that is proposed herein for the CF is that of a factor which aims at ensuring that, out of a large number of assessments carried out in accordance with EC8-3, only a predefined, acceptably small fraction of them leads to an unsafe result, i.e. to overestimating the actual safety. Admittedly, the idea that a single factor, with values depending only on the knowledge level, and not on all the aspects recalled above, may achieve the stated objective may appear as unrealistic. The paper represents an attempt to investigate to what extent this idea maintains some value. Further it provides a limited exploration on the magnitude of the CF values needed to reach the stated goal.

3. Proposed procedure for the evaluation of CF values

The proposed procedure consists of a simulation of the entire EC8-3 assessment process with the purpose of quantifying the dispersion in the assessment results due to the many choices/uncertainties described in the previous section.

The starting point of the procedure is to imagine an existing building, with all its properties, including the defects and spatial fluctuation of materials, geometry, etc. completely known. This ideal state of perfect knowledge can never be obtained in practice and it represents a state of knowledge higher (the highest possible) than the state of so-called complete knowledge described in the code (KL3).

In each simulation run choices (knowledge level, type and position of tests, how to process the results, analysis method, modelling options, etc) are made randomly to reflect the arbitrary choices made by different analysts. This obviously requires the spelling-out of all the steps described in the previous section, discretizing the possible choices in a finite number of options and filling the gaps of the code with practices coming from common-sense and experience in real-case assessments. It is imagined that the generic analyst will follow his trail down the procedure arriving at a different evaluation of the safety of the structure. This simulation is carried out without employing the confidence factor (i.e. CF=1). By repeating the process for a sufficiently large number, say n, of analysts a statistical sample (of size n) of the structural safety is obtained and can be used to estimate its distribution.

At this stage the statistical sample of structural states, quantified by the global state variable Y (a *critical* demand to capacity ratio, see Jalayer *et al* 2007) is compared with the *true* state of the structure. It is expected that a portion of the assessments will result in a *conservative* estimate (i.e. in a state worst than the real one) while the remaining will be on the *unconservative* side.

The goal of the last part of the procedure is that of reducing the fraction of un-conservative estimates to an acceptably small value. This is done by re-evaluating the structural state, using the same sets of choices of the previous evaluation, with a value of CF larger than one (i.e. decreasing capacities). If the procedure works as intended the new sample of structural states will have the predefined target fraction of unconservative estimates. The procedure can be split into the following steps:

- Step 1: Generation of the existing and perfectly known structure
  Once all the material properties and possible defects have been assigned a probability distribution, a structure can be generated by sampling a set of parameter values from the above distributions. This structure is by definition completely known and is termed the *reference* structure.
- Step 2: Generation of a sample of imperfectly-known structures from the reference structure
  A number N_{VA} of virtual analysts is given the task of assessing the structure. This step consists of simulating the process of inspection/information-collection, and produces N_{VA} different states of (imperfect) knowledge from the reference structure. These states are the starting point for the assessment by the virtual analysts. In order to reflect the different test plans designed by different analysts, this step requires the randomization of the test locations and test types.
- Step 3: Assessment of the reference structure
  The reference structure is assessed according to the code and the seismic intensity that induces the attainment of the limit-state (LS) under consideration is recorded. The attainment of the limit state is marked by a unit value of the global variable Y=1. This result is considered the true state of the structure.
- Step 4: Assessment of the imperfectly known structures
  The virtual analysts apply the code-based assessment procedure with a unit value of the CF and the same intensity as determined in Step 3. This produces a sample of N_{VA} values of the global state variable Y. This step requires a further randomization, reflecting the freedom left to the code-user in choices such as inclusion/exclusion of defects from modelling and the selection of the analysis method (linear vs. nonlinear, static vs. dynamic).

- Step 5: Statistical processing of the sample states and determination of CF

  Statistical processing of the sample of values of *Y* produces a distribution that exhibits a certain amount of variability around the value Y=1. This is shown in Fig.2 (left). The value of the CF can now be determined by enforcing the condition that a chosen lower fractile of *Y* (say, 10%) is equal to 1, i.e. the *true* state of the structure (as shown in Fig.2 (right)).

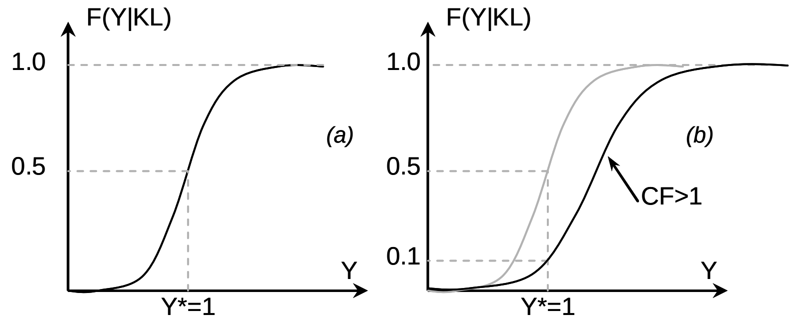

Figure 6. *Distribution of Y conditional on the KL (left), Enforced distribution of Y conditional on the lower fractile of Y (right)*

4. Application

The calibration procedure has been applied to a six-story three-bay RC frame structure (Fig.3). For the purpose of data collection (material tests, reinforcement details etc.) and post processing the structure is considered homogenous, in the sense that the spatial distribution of the properties/defects belongs to a single population.

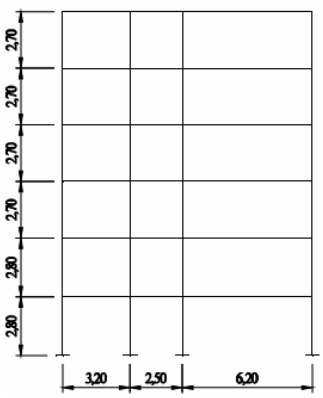

Figure 7. *Six-story, three-bay asymmetric frame*

The assessment has been carried out with the nonlinear static and dynamic methods. CF values have been evaluated both separately with each of the two methods, and jointly, to investigate dependence on the analysis method. For the purpose of dynamic analysis the seismic action is represented by seven recorded ground motions selected to fit on average, with minimum scaling, the EC8-specified spectral shape scaled to a PGA of 0.35g for soil class A (Iervolino et al, 2008). In the case of static analysis, the average spectrum of the recorded ground motions is considered as the demand spectrum.

In terms of modelling the nonlinear degrading response of the structure, account has been taken of flexure-shear interaction and joint hysteretic response. The model is set up in OpenSEES, employing flexibility-based elements for the members with section aggregator to couple a fibre section (flexural response) with a degrading hysteretic shear force-deformation law. Joints have been modelled with a "scissor-model" with a degrading hysteretic shear force-deformation law. Tangent-stiffness proportional damping has been used, calibrated to yield a 5% equivalent viscous damping ratio on the first elastic mode. Since the effect of brittle failure modes, such as shear in members and joints, has been included in the modelling (for both static and dynamic analysis), the structural performance is

checked in terms of deformation quantities only. Hence, $Y = \theta_{max}/\theta_C$, where θ_{max} is the demand peak inter-storey drift ratio, and θ_C the corresponding capacity. Detailed information on the characteristics (geometry, reinforcement, etc) of the structure and the adopted response models can be found in (Rajeev, 2008).
The purpose of generating the *reference structure*, material properties (concrete strength, steel yield stress, and hardening ratio) and structural defects (transverse reinforcement spacing in columns and beams, and column longitudinal reinforcement ratio) are sampled from predefined probability distribution functions (see Table.1).

Table 1. *Distribution type and parameters for random variables*

Random variable	Distribution	Mean or Min	CoV or Max
Column stirrup spacing	Uniform	200 mm	330 mm
Beam stirrup spacing	Uniform	150 mm	250 mm
Reinforcement ratio	Uniform	0.008	0.014
Concrete strength	LN	20 MPa	0.10
Steel Yield stress	LN	275 MPa	0.05
Hardening ratio	LN	0.04	0.25

A value of concrete strength has been sampled at each integration point along a member, while a single pair of values of steel properties has been sampled for all members of each floor. Correlation has been introduced amongst the concrete strength values according to an exponential decay model. The Nataf joint distribution has been adopted for simulation of the concrete strength field values (Liu and Der Kiureghian, 1986).

For the purpose of generating the *imperfectly known structures* (Step 2 of the procedure), the data collection procedure, consisting of tests on material samples from the structure and verification of reinforcement details, is randomized. The *number* of test/inspection locations is determined based on the minimum requirements in the code. These latter are specified as a function of the target KL. Test/inspection levels for KL1, KL2 and KL3 are denominated as *limited*, *extended* or *comprehensive*, respectively, when initial information is poor (relative to each KL requirements).

The actual test location chosen by each analyst is determined by randomly sampling (uniform integer distribution) first the member and then the location within the member (for this purpose each integration point is regarded as a possible test location). At each location, the testing/inspection consists of reading the value of the sought property from the reference structure (value generated during Step 1). Measurement errors are not considered. Since the reference structure is homogeneous by assumption, all the data gathered are *averaged* to obtain the values to be employed in the assessment.

The assumed scarceness of initial information, and in particular the lack of a complete set of construction drawings, influences the knowledge of the geometry of the structure. In particular, this may refer to the presence/absence of elements (a typical case being represented by beams in flat-slab structures) or the actual cross-section dimensions (significant variations in plaster thickness or the presence of cavities for ducts are common and cannot practically be ascertained for all members), or, finally, the precise unit-area weight of the floor system.

To model this kind of "geometrical" uncertainties (denoted as "residual" in the following) two types of additional random variables are introduced: the unit-area weight of floors (one variable per floor typology, e.g. typical floor and roof) and the cross section height of elements (one variable per element type: beams and columns). These random variables are sampled for each imperfectly known structure during Step 2. Detailed information can be found in Rajeev 2008.

5. Result

The assessment of the reference structure has been done with IDA (Vamvatsikos and Cornell, 2002) subjecting the structure to the seven natural ground motions selected to match EC8 spectrum. Consistently with the code indication of using 7 records and taking the average of the maxima, the intensity (PGA of 0.216g) where the mean IDA curve crosses Y=1 is recorded and used in Step 4. The capacity has been set for this structure to the deterministic value of θ_C = 2.5%.

Step 4 of the procedure consists of the assessment by each virtual analyst of its imperfectly known structure (the result of Step 2). As already mentioned, the number of analysts has been set to N_{VA} = 200, and each of them can choose between nonlinear static and dynamic analysis for the assessment. Actually, in this application each analyst has performed both analyses (dynamic and static). The results are first presented separately by method (200 samples each) and then *mixed* (400 samples). This, as anticipated, serves the purpose of investigating the dependence of CF on the analysis method.

The empirical distributions (conditional on KL) of the 200 Y-values obtained are shown in Fig. 4 (top left). In the figure the value Y=1 is marked by a dashed vertical line. For the employed seismic intensity of PGA = 0.216g this is the state of the reference structure. A second vertical (solid) line marks the value Y= 0.79. This is the state of the *mean* structure, i.e. a structure identical in geometry to the reference one, but with spatially homogenous properties equal to the average values of the samples generated in Step 1. As it can be seen, with increasing KL the distributions get steeper (lower dispersion) and closer to the *mean* rather then the *reference* structure. In all cases a large proportion of the analysts overestimate the safety of the structure (i.e. they find Y<1): roughly 40% with KL1, 70% with KL2 and 100% with KL3.

Next, the analysis is repeated with CF-values larger than one in order to reduce the above percentages to the same acceptably low value. For the purpose of this application this value has been set to 10%. Sensitivity of the results to this choice can be found in (Rajeev, 2008). Fig. 4 (top left) shows the corresponding distribution for KL1 only, for clarity (CF=1.34).

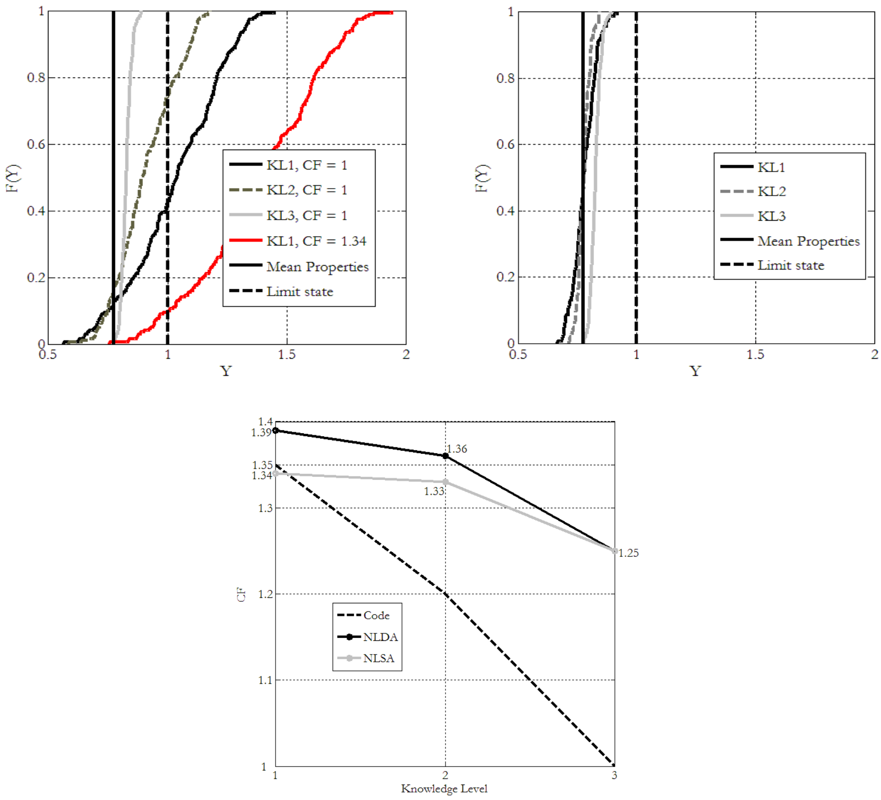

Figure 8. *Distribution of N_{VA} Y-values obtained by static analysis (top left), distribution by static analysis neglecting residual geometric uncertainty (top right), CF-values and analysis types (bottom)*

The relevance of the residual geometric uncertainty can be appreciated by comparing the curves in Fig. 4 (top left) with those in Fig.4 (top right), obtained disregarding this contribution (the difference between the structures analysed by the virtual analysts is only due to material properties and construction defects). The CF values obtained for the considered structure are summarized in Fig.4 (bottom). The figure reports separately the values obtained by static (grey) and dynamic (black) analysis, together with the code-specified values. It can be observed how the dependence of CF on KL is in all cases milder than that specified in the code, and that CF depends on the analysis method.

6. Findings and Conclusions

Based on the results, the most relevant findings are:

- In the code CF values are specified as a function of KL only, implying that KL is the single most important factor influencing CF. Results appear not to clearly support this expectation of the code. The dependence is found to be generally mild.
- The code does not differentiate CF values with respect to the analysis method, implying that epistemic uncertainty has the same effect with all analysis methods. Results appear again not to clearly support this assumption. When considered separately (i.e. assuming that all analysts will chose the same analysis method) nonlinear static results show a much reduced dispersion than those obtained by nonlinear dynamic analysis. This, according to the proposed procedure, leads in general to smaller values of CF to be employed with static than with dynamic analysis.
- The code specifies that the geometry of the structure must be completely known before setting up a model for the analysis. Experience with real-case assessments shows that it is usually not possible to obtain accurate measurements over the entire structure and that even when member centrelines are known, a residual uncertainty on the cross-section dimensions is unavoidable. This source of uncertainty has been modelled in the applications. Results show it to be, for the examined case, at least of the same order of importance of that associated with material properties and defects.

In conclusion, within the limits of the analyses carried out, it appears that current CF-based format of Eurocode 8 Part 3 doesn’t show to be entirely adequate for its purpose, and should be improved since:

- CF values are not differentiated with respect to analysis method/modelling options;
- CF values are not differentiated with respect to structural type (size, regularity, construction material, load-resisting system, etc);
- the so-called complete knowledge (KL3) does not actually correspond to a state of perfect knowledge, hence, it should be penalised with a CF value larger than one.

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