Assessing existing buildings with Eurocode 8 Part 3: a discussion with some proposals.

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Abstract: the paper contains a few reflections stimulated by the experience of applying EC8 Part 3 to real cases. Some of them are of more conceptual character, such as that on the definition and quantitative evaluation of a limit state for the structure, or that on the treatment of the uncertainty. Their intention is more to raise a discussion than to provide answers ready for application. Others deal with more applicative aspects for which an opinion is offered on possible concrete solutions, such as suitable extension of the codified non linear static analysis method and the verification of members under bidirectional loading.

Introduction

The fundamental difference existing between the two processes of designing a new structure and of assessing an existing one, especially when the latter is an old construction and the assessment refers to its seismic resistance, is well recognised.

Eurocode 8 Part 3 (EC8 Part 3) represents perhaps the only international document to address the issue of the analytical seismic assessment of buildings in a normative way. The decision to take such an approach, a feature proper to the Eurocodes system, has entailed facing, in the phase of drafting the document, severe conceptual and practical challenges, and it is anticipated that difficulties of various nature will be met in its application as well. Some of these difficulties will disappear with the future editions of the document, thanks to the progress made by the intense research activity currently devoted to the subject. It would be vain, however, to expect that some time in the future the seismic assessment of an existing structure will become a matter of routine, similarly to what is possible for the design of a new one.

The unavoidable limitation of knowledge, as much on the structural system as a whole, as on the single structural components, the difficulty in modelling behaviour and capacity of components not intended to resist actions of seismic origin, the necessary use of less familiar and more complex (mostly non linear) methods of analysis and, perhaps most importantly, the measure of personal responsibility involved in the decisions the analyst has to take along the assessment process, are all elements that contribute into making any individual assessment a case of its own.

This paper contains a few, unsystematic observations on certain aspects of conceptual and operational nature deriving from a first experience of application of EC8 Part 3 to various types of buildings.

Performance requirements and compliance criteria

According to the approach now standard for all design situations, EC8 Part 3 defines the performance requirements for the seismic case in terms of a number of limit states (LS).

Reference is made here to the most severe of the three LS's, i.e. the one denoted of Near Collapse (NC), but similar considerations apply to the other two LS's as well.

The definition of the NC-LS reads: "*LS of Near Collapse: The structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non structural components have collapsed. Large permanent drift are present. The structure is near collapse and would probably not survive another earthquake, even of moderate intensity*".

Compliance with the requirement of non exceedance of this and of the other LS's is "*achieved by adoption of the seismic action, method of analysis, verification and detailing procedures contained in this Part 3 of EN 1998 […]*".

Taken to the letter, the compliance criteria would appear not only exceedingly restrictive (with the verifications to be satisfied for all individual primary elements, very few existing buildings would pass the test), but also inconsistent with the LS definition, that describes a scenario of severe damage diffused across the entire structure and such as to bring it close to collapse.

A more plausible interpretation of the general requirement involves the definition, on the part of the analyst, of a number of structural situations that are realistically conducive to collapse. Such situations depend on the structural topology and involve in general both single components as well as specific sets of components. Figure 1 illustrates the concept with reference to a simple frame, by making use of the well known fault-tree representation. One may assume in this example that a state of collapse is reached whenever a single column fails either in shear or in flexural distortion (the latter not shown for simplicity), or at the occurrence of hinging at all columns of one floor, or of other types of mechanism involving hinges forming at both columns and beams (not shown).

Clearly, in the general case there is a distinct subjective component in a proper selection of the collapse mechanisms, and this choice remains under the responsibility of the analyst.

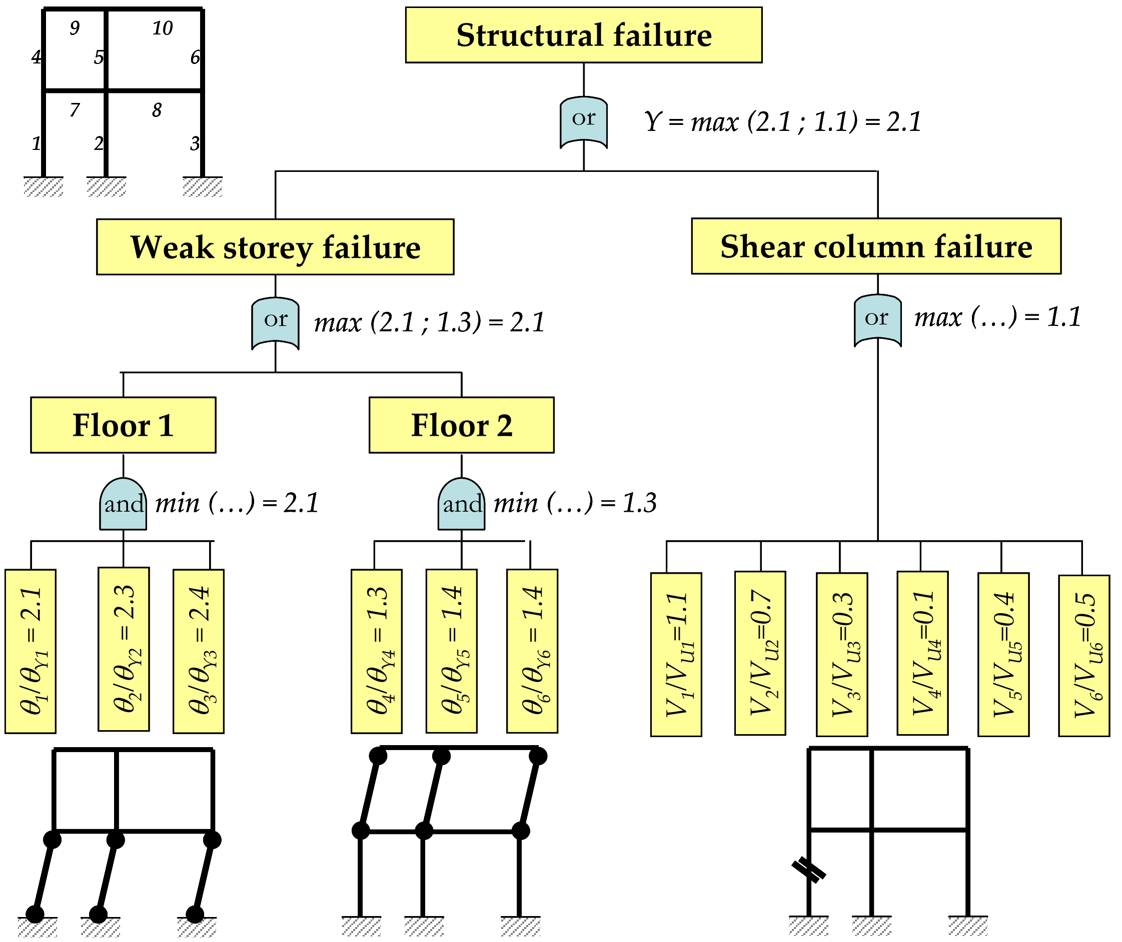

Figure 1. Fault-tree representation of the collapse of the frame in terms of (a sub-set of) its collapse mechanisms.

In the fault-tree representation, the system is described as a serial arrangement (gate OR) of subsystems, some of them made of a number of components working in parallel (gate AND). In the example shown, failure of the system is assumed to occur due to either to the formation of weak storeys mechanisms, or to shear failures in the columns (first serial connection). In turn, failure due to a storey mechanism can occur at any floor (second serial connection). On the other hand, formation of a storey mechanism requires yielding of all the columns of the storey (parallel connection), and so on for the additional mechanisms one could consider.

This representation of the system allows determining its state through the use of a scalar quantity: Y defined as:

$$Y = \max_{i=1,N_s} \min_{j=1,Ni} R_{ij} \quad (1)$$

where R_{iij} is the ratio between demand and capacity at the j-th component of the i-th subsystem, N_i the number of components of the i-th subsystem and N_S the total number of considered subsystems. Demand and capacity can be expressed in terms of shear force, chord rotation, joint principal stresses, etc.

From its definition, when the variable *Y* reaches unity the structure attains the LS under consideration, hence for the verification to be satisfied it must be $Y \leq 1$.

In the example figure, the critical demand-to-capacity ratio comes from the bottom storey weak-storey mechanism, where the minimum chord rotation ratio is 2.1 and this is the max value over all subsystems, hence the verification would not be satisfied.

In conclusion, the essential concept outlined above is that it should be up to the analyst to properly identify all significant situations corresponding to the attainment of a LS consistently to its definition in EC8 Part 3. This implies also the possibility of eliminating from the verification some of the elements not considered as relevant for the LS. As an example, consideration of the damage to the beams, while relevant for the Light Damage LS, could be omitted in some cases for the NC-LS. At the same time, the above framework allows to include situations not explicitly enforced by the code, such as the weak-storey mechanism ("*low residual lateral strength and stiffness*"). Once the situations leading to the LS attainment are identified, the following step is to represent them by means of a fault-tree, which in turn allows global verification of the structural system through the value of the scalar variable *Y*.

The confidence factor and the treatment of the uncertainty

Incompleteness of knowledge, which to a variable extent is always present in the process of evaluating existing structures, is covered in EC8 Part 3 by means of a single factor, additional to the same material partial factors in use for design, called Confidence Factor (CF). The value of CF depends on the level of knowledge (KL), which in turn depends on the amount of information available on three main aspects: Geometry, (construction) Details and Materials. The range of the states of knowledge is discretized into three levels denoted as Limited, Normal and Full, respectively, with corresponding suggested CF values of 1.35, 1.20 and 1.0. Given a knowledge level, the same CF value is applied for the verification of all LS's.

The function of the CF is two-fold. In evaluating the capacity of the individual elements, be they brittle or ductile, the CF is used as an amplifier of the ordinary gamma-factors of the materials. In case of a linear type of analysis, the CF is also used in evaluating the demand on the brittle mechanisms, in the same way as the so-called “capacity design factors” γ_{Rd} are used in EC8 Part 1. It may be worth recalling that the values of the standard gamma-factors were originally calibrated (incidentally, with reference to non-seismic situations) to account for dispersion of the material properties (additional to the part covered by using fractile values), and, partially, to account for the model uncertainty. Amplifying the gamma-factors by means of the CF allows accounting for larger uncertainties on the material properties, as well as for the higher protection required for the brittle elements.

Is this procedure conceptually adequate for dealing with the kind of (epistemic) uncertainty arising in the assessment of existing structures?

A direct answer can only be negative, since a factor or even a set of factors, cannot compensate for the kind of incomplete knowledge one has normally to face when dealing with existing structures such as, for example, the ignorance on whether a structural detail is present at all.

Structural reliability theory offers standard tools for dealing with this different kind of uncertainty.

Briefly, it would be required setting up a number of different models of the structure, each one associated with a weight (all weights summing up to one) representing the “degree of belief” the analyst, based on his experience and the available evidence, has on the different alternative models. For each model a probabilistic analysis would then be performed to determine its conditional risk (the probability of exceeding the considered LS given the model) and, finally, the unconditional risk would be obtained as the sum of the products of the conditional risks by their respective weights.

Within the deterministic framework proper of EC8, the counterpart of the above procedure would consist in starting with the same first two steps of the probabilistic procedure, i.e. setting up a number of models and assigning a weight to each of them. For each model the corresponding value of Y would then be evaluated, so as to arrive at a final weighted value of Y which would correspond to the best estimate that can be obtained of it.

The procedure outlined above can be regarded simply as the formalisation of the old practice of performing sensitivity analysis on the response of the structure to important model parameters, routinely carried out when these are affected by significant uncertainty. The major difference with the previous procedure is that in sensitivity analysis subjectivity enters in the final adoption of one of the models, while in the former it enters in the definition of the weights.

Analysis: what scope for linear methods?

In the assessment of an existing building, accuracy of the method of analysis is of crucial importance, since a conservative method may indicate unnecessary expensive interventions while a non-conservative one may leave the building exposed to an excessive risk.

Of the five methods allowed in EC8 Part 3, the q-factor approach, with a default value of q=1.5 (for RC structures, the method is not even mentioned for masonry structures) is in most, if not in all cases conservative, hence it should find application when it is a priori apparent that the building is not in need of interventions. Nor it is in general worth attempting to justify analytically the use of larger values of q, unless the ductility properties of the building can be easily and properly documented.

Of the two other linear methods of analysis, the lateral force and the multi-modal, given the advancement in the graphical interfaces of available analysis software, use of the former, which is obviously less accurate, is difficult to justify, since it does not involve any economy in setting-up and in analysing the model.

The applicability of these linear methods, however, is conditional to a substantial uniformity, over all ductile primary elements, of the ratio between the elastically evaluated demand due to the elastic seismic action and the corresponding capacity. Verification of this condition is clearly onerous, since it can be carried out only after the analysis of the structure and of its capacity is completed. It is to be noted, however, that the above condition represents the true quantitative definition of "regularity" of a structure from a seismic point of view, one that supersedes and, in case of contrast, should prevail over the semi-quantitative, common-sensical but rather arbitrary definitions given in EC8 Part 1 for the design of new buildings, and unnecessarily enforced also in EC8 Part 3 (4.4.2 and 4.4.3).

Unfortunately, it is not to be expected that many existing buildings can satisfy such a strict condition as suggested in EC8 Part 3: $\max(D_i / C_i) / \min(D_j / C_j) \leq 2.5$, the consequence of which is that non-linear methods of analysis will be likely employed in the vast majority of cases. This is considered to be even more true for masonry structures. Actually, for these structures several additional conditions must be fulfilled for the application of linear methods, such as vertical continuity of all walls, rigid floors, maximum stiffness ratio between walls at each floor less than 2.5, etc. All these conditions are such that in practice a tiny percentage of existing masonry buildings, especially those of old construction, could be assessed using a linear method. Further, it is noted that for masonry structures, the condition on the D/C ratios is not of straightforward application, as in the case, for example, when the structure is modelled using finite elements.

Non-linear static methods

Regarding the non-linear methods of analysis, only the static approach (pushover analysis) is discussed in the following, since the use of non-linear dynamic analysis is unrestricted and, to some extent, EC8 leaves to the analyst the responsibility of making the proper choices for accurate results.

EC8 Part 3 basically refers to the version of pushover method contained in Part 1, hence reflects the state of advancement of this technique in the early 2000s. Much progress and experience have been gained in the meantime, in part also from the applications made in assessing real buildings.

The version presented in Part 1 (the N2 method, Fajfar and Gaspersič 1996) was originally devised for planar, single-mode dominated structures, and makes use of two structure-independent load patterns. Its extension to unsymmetrical buildings consists of a rather hybrid procedure, whereby the applied loading pattern is still planar and structure-independent, and, to account for the dynamic amplification due to torsion, the displacements on the stiff-strong side as obtained from the pushover are increased by a factor based on the results of a spatial modal analysis.

Several more direct proposals are now available in the literature that can account for multiple modes contribution, including of course torsional modes, and recourse to such methods is explicitly allowed in a note of EC8 Part 3 (note at 4.4.4.5). One of these methods, due to Chopra and Goel (2002, 2004), in spite of its inherent approximation which is common to all multi-mode methods (i.e., making use of superposition of effects in the non-linear range, and also of the modal combination rules valid for elastically responding structures), has shown to provide acceptably accurate results and offers the advantage of being a rather straightforward extension of the original N2 method.

In this method a set of fixed loading patterns is considered, each one given by the product of the mass matrix by one of the selected mode shapes (hence a spatial loading pattern). A pushover analysis is carried out for each pattern with the maximum displacement obtained by using an appropriate inelastic response spectrum. All desired response quantities (member chord rotations and forces, joint principal stresses, etc) are then calculated mode by mode and combined using the SRSS (or CQC) rule. The SRSS rule can also be applied for combining the maxima due to the two horizontal components of the seismic action, leading to the final expression for the generic scalar response quantity R:

$$R = R_G + \sqrt{\sum_{i=1}^{N} \left(R_{i,E_X} - R_G\right)^2 + \left(R_{i,E_Y} - R_G\right)^2} \qquad (2)$$

where the summation is over the N considered modes, R_{i,E_X} and R_{i,E_Y} are the values of the response quantity for mode i due to the X and Y component of the seismic action, and R_G is the response under gravity load. This latter must be subtracted from those due to the seismic action, since all the pushover analysis start after the application of the gravity loads. In general, the modal responses in equation (2) must be evaluated for both signs of the load patterns, since $R_{E_X} \neq -R_{-E_X}$.

A problem arises with the use of equation (2) for the determination of member forces, since the contribution of all modes are summed up with positive signs, and this may lead for ex. to unrealistic demands in terms of bending moments as well as to shear force values that are not in equilibrium with the bending moments at the member ends.

Equally unsolvable in rigorous terms is the problem of shear verification of columns, due to the uncertainty in the evaluation of the normal force. A larger axial force increases the flexural strength at the end, hence the shear demand (through equilibrium); on the other hand, it increases also the shear capacity with ensuing uncertainty on the value of the ratio D/C.

An approximate solution to the last problem, in analogy with the definition of some damage indices or the Miner's rule for fatigue, consists in evaluating the D/C ratio (i.e. the ratio $V_i(N_i)/V_R(N_i)$) for each mode (conserving signs and not violating equilibrium or constitutive laws) and in using the modal combination rule on these ratios. The verification would then be:

$$\sqrt{\sum_{i=1,N} \left(V_i(N_i)/V_R(N_i)\right)^2} \leq 1 \qquad (3)$$

In practice the difficulties discussed above is often made less severe by the fact that for many structures the response is predominantly governed by just one mode for each direction of the seismic action, in which case the summation in equation (2) is little affected by the contribution of higher modes. In the limiting case where only one mode would be significant for each direction of the seismic action equation (2) would reduce to:

$$R = R_G + \sqrt{\left(R_{i,E_X} - R_G\right)^2 + \left(R_{j,E_Y} - R_G\right)^2} \tag{4}$$

Verifications

A problem not explicitly dealt with in EC8 Part 3 is how to carry out verification of both ductile and brittle elements under bi-directional loading. This is the normal condition under which members are due to the simultaneous application of multiple components of the seismic action, and the lack of guidance is the direct result of the lack of knowledge (theoretical as well as experimental) on the biaxial deformation and shear capacities at ultimate.

With reference to the deformation capacity, a limited experimental evidence (Fardis, 2006) supports the use of an "elliptical interaction" domain at ultimate. Proceeding as for equation (3) on a mode by mode basis, the bidirectional demand to capacity ratio (BDCR) would read:

$$BDCR_i = \sqrt{\left(\frac{\theta_{2i}}{\theta_{2u,i}}\right)^2 + \left(\frac{\theta_{3i}}{\theta_{3u,i}}\right)^2} \tag{5}$$

where θ_{2i} and θ_{3i} are the contributions of the *i*-th mode to chord-rotations in planes 1-2 and 1-3 (axis 1 being the longitudinal one), and $\theta_{2u,i} = \theta_{2u}\left(N_i\right)$ and $\theta_{3u,i} = \theta_{3u}\left(N_i\right)$ are the corresponding uniaxial capacities at ultimate. Using the SRSS rule to combine the modal contributions the verification consists in checking that $\sqrt{\sum_{i=1}^{N} BDCR_i^2} \le 1$.

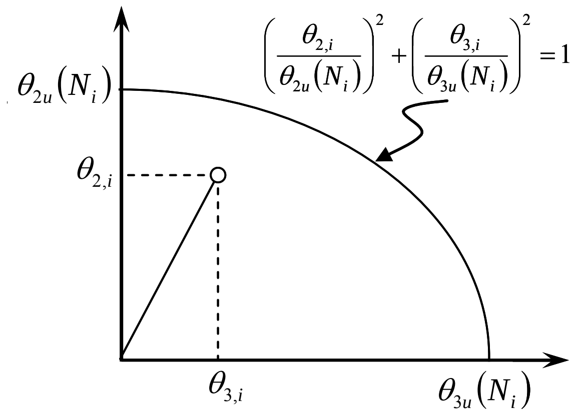

Figure 2. Elliptical interaction diagram for chord-rotation at ultimate.

No comparable experimental evidence exists with regard an interaction domain for biaxial shear. It is proposed to adopt a similar format as that of equation (5).

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