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THE EVOLUTION OF EUROCODE 8 – PART 3: MAIN CHALLENGES AND KEY CHANGES

Andreas KAPPOS[1,2]
On behalf of the Project Team SC8.T3

ABSTRACT

The systematic review process of EN 1998-3, launched by CEN/TC250 in 2014, resulted in a relatively limited number of comments, which cover well some issues like the definition of limit states, the methods of analysis, and the equations for predicting strength and deformation capacities of reinforced concrete and unreinforced masonry members, but did not even touch upon other important issues like assessment and retrofit of steel members, assessment and retrofit of bridges, and use of passive systems as a retrofit technique.

The Project Team (SC8.T3 or simply 'PT3') that was charged with the drafting of the new EN 1998-3 (EC8-3 hereafter), started working in October 2015. A daunting challenge that PT3 faced was to extend the scope of EC8-3 (whose current version is strictly limited to buildings) to the assessment and retrofit of bridges. Other major challenges were to produce a relatively easy to use code, which at the same time would be transparent to the 'average' engineer, without substantially increasing the size of the Code. This is close to squaring the circle and the PT decided to opt for some increase in size with a view to promoting clarity and ease of use. The new EC8-3 comprises a total of 12 chapters ('clauses'), the new ones among them being the material-related (Specific rules for reinforced concrete, steel and composite, timber, and masonry structures), which in the current EC8-3 are covered in informative annexes, and a final chapter devoted to bridges.

A large number of changes have been implemented in EC8-3, some resulting from the national comments and several others resulting from suggestions made by SC8 and by the PT3 members. Among them one could mention: revised definitions of four limit states; introduction of global (structure–level) verifications; revised definition of Knowledge levels (distinct for Geometry, Details and Materials); new (probabilistically derived) γ_{Rd} factors and abolishment of the existing 'confidence factor'; improved nonlinear static analysis procedures, also covering bridges; new, better calibrated, formulae for strength and deformation of concrete members, both as-built and strengthened; new detailed rules for pushover analysis of masonry buildings and the associated global verifications for both 'in-plane' and out-of-plane mechanisms; and new chapters for timber structures and for bridges. Last and not least the Code has been substantially restructured to correspond to the successive steps a designer would follow in carrying out the assessment and/or redesign of a structure (building or bridge).

Keywords: Eurocode 8; Existing structures; Structural assessment; Retrofit

1. THE SITUATION AT THE BEGINNING OF THE EVOLUTION STAGE

Among the 58 Eurocodes currently in force, EN1998-3 (Eurocode 8 – Part 3) is the only one that deals with existing structures. This has several repercussions and creates a number of challenges, one of them being the direction to be followed for its revision/evolution. A major challenge related to this code has nothing to do with technical issues, and is the extent of its *enforcement*. Unlike all other Eurocodes, EC8-3 (CEN 2005) does not apply to all (existing) buildings but only to those wherein the owner(s), or a competent authority (that sets up a nation-wide or regional retrofitting programme) decide to carry out an assessment of a single building or a category of buildings, or indeed the entire building stock in a

[1] Professor, Civil Engineering Department, Aristotle University of Thessaloniki, GR 54124, ajkap@civil.auth.gr
[2] Director, Research Centre for Civil Engineering Structures, Civil Engineering Department, City, University of London, London, EC1V 0HB, UK, Andreas.Kappos.1@city.ac.uk

certain area. An interesting question is whether in these cases, i.e. when the owner or the authority decides to assess the seismic capacity of the building(s), EC8-3 is the only code that can be used. The answer to this might seem obvious, but the fact is that, because retrofitting of existing buildings is a costly procedure, individuals, as well as governments, prefer to retain the 'do nothing' option, even when application of EC8-3 (or, indeed any other current code for seismic assessment) clearly shows that a building is subjected to an unacceptably high seismic risk and will suffer very heavy damage or even collapse when subjected to the seismic action used for the design of new structures. The current EN1998-3 clearly states that it applies "once the requirement to assess a particular building has been established. The conditions under which seismic assessment of individual buildings – possibly leading to retrofitting – may be required" are beyond its scope.

Due to the fact that, at least at the time EC8-3 was written, existing knowledge on assessment of (realistic) structures was at a lower level than that related to design of new structures, and perhaps as a reflection of the sensitive issues mentioned previously, the level of detail included in this part of Eurocode 8 was lower than that in most other parts. In fact the main (normative) part of the Code is not only brief (25 pages) but clearly insufficient for practical implementation. This serious shortcoming was partly remedied by the inclusion of three annexes (55 pages in total) that provide some guidance on the implementation of the Code to reinforced concrete, steel, and masonry buildings. Importantly, all three annexes are *informative* (as opposed to normative, i.e. compulsory), which is good and bad. It is good in the sense that there is freedom to use alternative, more detailed and more comprehensive, provisions, especially in countries where such documents exist (e.g. Italy has issued in 2005 a comprehensive code for seismic assessment of masonry buildings and monuments, and Greece an analogous one for old reinforced concrete buildings). The negative aspect of this freedom is the obvious risk of a lack of uniformity, whereas further *harmonization* (and reduction in number of national choices) is a stated objective in the Eurocode evolution process. In fact, to the writer's best knowledge, EN1998-3 has not been really implemented yet in any country (except for some pilot studies), one reason being the difficulty that many countries faced in trying to implement EC8-3 based on the limited information included in the current document. Therefore, a critical issue in the EC8-3 evolution was whether it should become more detailed (and, inevitably, voluminous), which seems to be in line with the 'Ease of use' objective of the evolution process, or it should stick to the current format of brevity and of the informative annexes, with the aforementioned shortcomings.

With regard to specific technical aspects of EC8-3, three issues were arguably the ones to which particular emphasis should be given in future versions of the document:

- Broadening of the scope of EC8-3 to include bridges.
- Performance criteria used for the assessment of the structure in its pristine or damaged condition, as well as subsequent to the application of the selected intervention (strengthening) scheme.
- Modelling of the structure in the above distinct phases (prior and subsequent to strengthening).

Regarding *bridges*, this was a brand new topic as far as EC8-3 is concerned. Nevertheless, EC8-2 covers several aspects that can be applied to assessment of existing bridges, in particular those related to seismic actions and analysis; regarding the latter, key issues are those related to modelling and to performance criteria, discussed below. What was really missing was *strengthening* techniques for bridges; criteria for the verification of strengthened reinforced concrete (R/C) members are provided in the annexes of the current EC8-3 and could be extended to bridges, but other techniques that are much more common to bridges, like the addition of restrainers and/or shear keys had to be added afresh, while seismic isolation and supplementary damping were, in theory, covered in clause 10 of EC8-Part 1, but additional material should be added in EC8-3 to cover aspects related to using passive systems as a retrofit solution, rather than at the initial construction phase.

Regarding *performance criteria*, major issues that deserved particular scrutiny during the evolution process were:

- *Allowable values of deformations* at the critical regions of members related to each performance level (Limit States of Near Collapse, Significant Damage, and Damage Limitation). For instance, in the case of R/C, there were new relationships for the available plastic rotation capacities in the 2010 *fib* (2012) Model Code for Concrete Structures, which are based on more extensive calibration that those included in the (older) EC8-3.
- Relationships for the available *shear strength* of existing, poorly-detailed, structural members; again there are more recent versions of the EC8-3 equations.

Regarding *modelling* of the structure (building or bridge) for linear or nonlinear analysis, a critical issue is the *stiffness* to be adopted for members that are prone to early cracking (R/C and masonry elements), given that the approach to developing the retrofitting scheme was, and will certainly remain, *displacement-based*, hence displacements should not be underestimated. EC8-3 (CEN 2005) properly defines stiffness of reinforced concrete members using an empirical estimate of the secant stiffness at yield, but, again, there are more recent developments in these relationships, on both sides of the Atlantic. When PT3 started its work in October 2015, one of the key problems it had to deal with was that there was a relatively limited number of comments (most of them from Greece, France, Slovenia, and the UK), which covered well some issues like the definition of limit states, the methods of analysis, and the equations for predicting strength and deformation capacities of R/C and unreinforced masonry (URM) members (at their initial or strengthened state), but did not even touch upon other important issues like assessment and retrofit of steel members, timber members, assessment and retrofit of bridges, and use of passive systems as a retrofit technique.

2. OVERVIEW OF PT3 WORK

The composition of PT3 is given in the first column of Table 1. One might be tempted to note a bias towards Southern Europe, as only the UK is represented, but one should not forget that France in particular, and Northern Europe in general, were amply represented in the work of the PT through the SC8 Chairman Philippe Bisch, who (with only one exception) attended at least parts of all PT meetings and, of course, reviewed and commented on all drafts issued so far.
Given the scope of the work and the really tight time-frame (the final draft had to be delivered at the end of October 2017!), it was clear that each member could not possibly distribute his/her contribution evenly among all chapters (clauses). Hence, main and secondary drafters were appointed for each clause (as shown in the second and third column of Table 1), broadly reflecting the primary expertise of each member. It has to be noted that the clause numbers in Table 1 are the first six of the existing (2005) version of EN1998, i.e. 1-General; 2-Performance requirements and compliance criteria; 3-Information for structural assessment; 4-Assessment (mostly focusing on analysis issues); 5- Decisions for structural intervention; 6-Design of structural intervention, whereas 7 corresponds to the existing Annex A (R/C structures), 9 to Annex C (Masonry buildings), and 10 to the new clause on Bridges. Important in the context of the PT3 work is that there was no main, (or even secondary) drafter for clause 9 corresponding to the former Annex B for Steel and Composite structures, due to the lack of such expertise in the PT; this issue is further discussed later in this section.

Table 1. Composition of PT3 and allocation of work

Member	Primary Clause(s)	Secondary Clause(s)
Kappos, Andreas (UK)	1, 5, 6	9
Chrysostomou, Christis (CY)	7	4
Franchin, Paolo (IT)	2, 3	4, 10
Isaković, Tatjana (SL)	4	7
Lagomarsino, Sergio (IT)	9	2, 3
Panagiotakos, Telemachos (GR)	10	6

PT3 has held a total of 12 meetings over the 26 months of its work until November 2017, almost evenly distributed among 'physical' (person-to-person) meetings and teleconferences. These were supplemented by thousands of emails and document exchanges through a remote server. PT3 has submitted two intermediate drafts to NEN/CEN, in April 2016 and April 2017 and its final draft in November 2017. It is perhaps worth mentioning that although the contract of the PT members started nominally on 1/1/2015, the actual work started, as already mentioned, in October 2015. The final PT draft will be further scrutinised by the national groups of CEN/TC250/SC8 and the final comments (to be submitted by 28/2/2018) will be taken into account in finalising the document, as far as PT3 is concerned. Of course, there is still some way to go, as the clause on Steel Structures is still missing.

Although most of the new material in the document has been written by PT3, due to the lack of expertise in steel/composite and in timber structures, external assistance was clearly needed and this was sought primarily by the pertinent working groups (WG) of CEN/TC250/SC8. This worked well with WG3 (Timber) which managed to provide a first (incomplete) draft in June 2017 and a more complete (but still missing some sub-clauses) draft just prior to the delivery of the final draft in November 2017. As a result of this valuable contribution the final draft contains a reasonably satisfactory clause (10) on Timber Structures which will be completed and finalised in the version to be submitted in April 2018, after all comments by the national groups and the WGs will be taken into account. Unfortunately, for a number of reasons things did not work equally well with WG2 (Steel), although an ad-hoc group (WG2/TG5) was set up in early 2016. The latter has sent in August 2016 some comments on steel-related aspects in EC8-3, but at the time of writing this paper (Jan. 2018) there was no further input from them. Clearly, addition of a clause on Steel structures is envisaged by SC8 (composite might perhaps be left out as it is hardly a critical problem in existing structures), but the time frame for this is not clear yet. The understanding is that the future clause on steel structures will be less detailed than those on the other materials.

3. KEY CHANGES IN EACH CLAUSE

The new structure of EN1998-3 is the following:

1. Scope
2. Normative references
3. Terms, definitions, and symbols
4. Basis of design
5. Information for structural assessment
6. Seismic action, methods of analysis and verification
7. Design of structural intervention
8. Specific rules for reinforced concrete structures
9. Specific rules for steel and composite structures *(to be drafted)*
10. Specific rules for timber structures
11. Specific rules for masonry structures
12. Specific rules for bridges

Annex A: Preliminary analysis
Annex B: Supplementary information for concrete structures
Annex C: Supplementary information for steel structures *(to be drafted)*
Annex D: Supplementary information for timber structures
Annex E: Supplementary information for masonry structures
Annex F: Flowchart for the application of this standard

It is obvious that the main changes with respect to the current version are the addition of two completely new clauses (10-Timber and 12-Bridges) and the inclusion of the former material-related annexes (R/C, URM and, in due course, Steel) into the main body of the Code. The importance of the latter is significant, i.e. the pertinent clauses become now a mandatory part of the Code and will significantly affect the assessment and strengthening of existing R/C, steel, timber, and URM structures in the future. In the limited space available herein, only the most important (at least in the writer's view) changes in each of the main clauses will be presented and briefly discussed, whenever needed.

3.1 General issues

Changes in the 'general' clauses 1 to 3 more or less reflect those in the other clauses; e.g. the scope of EN1998-3 is extended to cover bridges, and symbols are being harmonised across all Eurocodes (this is a still on-going effort).

An important item wherein more clarity is required is cl. 1.3(2) where it is mentioned that "The provisions of this Standard assume that the data collection and tests are performed by *experienced personnel* and that the *engineer* responsible for the assessment, the possible design of the retrofitting and the execution of work has *appropriate experience* of the type of structures being strengthened or

repaired”. There is certainly a need to provide a definition of ‘experienced personnel’, otherwise a constant point of friction in the application of the Code will be who is eligible to apply it! Having said this, and bearing in mind that the situation is, to a certain extent, different in each country, the detailed description (including qualifications) of the experienced personnel and engineers may be provided by the competent authorities in each country.
An interesting *terminology* issue (cl. 3.2) was raised and thoroughly discussed within the PT, i.e. should the American terms ‘capacity’ and ‘demand’ be used (as done in several clauses of the current EC8-3 that were apparently influenced by the US documents) or should the standard Eurocode terms ‘resistance’ and ‘action effects’ be reinstated along the entire document? It was finally decided to basically retain the Eurocode terminology (e.g. ‘shear resistance’, rather than ‘shear capacity’), with the notable exception of *displacement* and *deformation* related quantities, e.g. ‘displacement capacity’ (or ‘rotational capacity’), as terms like ‘displacement resistance’ sound awkward and confusing.

3.2 Basis of design

This was the part of the Code on which the relatively larger fraction of the deliberations of PT3 was devoted, as it affects all other clauses (and, to a lesser extent, because it was the one that usually topped the agenda of the meetings). The key issues are critically summarised in the following.

3.2.1 Limit States (LS)

In lieu of the three LS adopted in the current EC8, four are adopted in both parts of the new EC8 (this was a typical case of harmonisation between Part 1 and Part 3), namely: *Near Collapse* (NC), *Significant Damage* (SD), *Damage Limitation* (DL) and *Fully Operational* (OP); the latter is the new LS, introduced with a view to important structures that should remain operational after a strong earthquake.
The key issue here (one of major importance in the writer’s opinion) is *how many verifications* of LS should be required by the Code. One should recall in this respect that the basic concept of performance-based design and assessment is that it should not be taken for granted that checking for one LS ‘automatically’ guarantees that verifications for all other LS will be (at least more or less) satisfied, and all PBD documents include ‘matrices’ of performance objectives, wherein a number of seismic hazard levels are associated with a number of different *performance objectives* (or LS in European terminology). So, in principle, the new EC8-3 should comply with this ‘philosophy’ and ask for multiple verifications. There is bad and good news in this respect: The bad news is that, in the name of simplicity and ease of use, the compromise had to be made that *only one* verification was made mandatory, hence, strictly speaking, EC8-3 is *not* a performance-based document, from this perspective. The good news is that PT3 made an effort (jointly with PT1, the EC8-1 team) to make this single verification as correct and effective as possible. It is recalled here that in the current EC8 the reference LS is SD, while NC is assumed to be satisfied through an appropriate level of ductile detailing etc. Clearly, this is not the case with old structures, hence the new EC8-3 adopts NC as the primary LS verification, allowing for the exception (cl. 4.1(2)) that “the LS of Significant Damage may be checked in lieu of the LS of NC, irrespective of seismicity, if a preliminary analysis indicates a low demand of inelastic deformation”. For NC it is stated that “When […] exceeded it should be always reported whether the loss of bearing capacity of an element has the potential to escalate into a global collapse or it is deemed to remain confined in a partial localised collapse […]”; while, for SD it is stated that “If […] checked in lieu of the NC one […], then the resistance for SD cannot exceed the resistance for NC divided by the ratio of the seismic action for the verification of NC to the seismic action for the verification of SD”. This ratio is another parameter that is better defined in the new Code, as, instead of the inappropriate 3/2 adopted in US documents or the 4/3 in the current version of EC8-3, a proper estimation is made as the ratio of the spectral accelerations $S_{s,TNC,IC}/S_{s,TSD,IC}$ where T_{NC} and T_{SD} (specified in Part 1) are the return periods of the seismic actions associated with the NC and SD LS, respectively, and IC is the Consequence Class.

3.2.2 Partial safety factors

An important issue addressed in a new (and more consistent) way in EC8-3 is that of safety factors to be applied to resistances, noting that in many cases verifications are carried out in terms of deformations,

rather than forces. It is notable that, although this was not requested in the mandate to PT3, significant work has been carried out by P. Franchin and his co-workers at Sapienza University of Rome (Franchin 2018) who calibrated resistance factors (γ_{Rd}) accounting for epistemic uncertainty in the resistance model, aleatoric uncertainty on variables input to the model, statistical uncertainty on these variables stemming from limited available data (the experimental database compiled and processed by the University of Patras group was used, see Grammatikou et al. 2015), and target safety. These factors are given in the clauses for R/C and URM structures. The partial factor γ_{Rd} should apply to the resistance evaluated using *mean* values of all input variables and taking into account a lower fractile of the resistance distribution, uniform across resistance models.
In addition to γ_{Rd} factors for a given fractile of resistance $k_1 = 0{,}16$ (i.e. mean minus 1σ), the procedure to calculate values for a different fractile (k_2) is also provided in cl. 4.2.2, i.e. $\gamma_{Rd,k_2} = \gamma_{Rd,k_1}^{\kappa_2}$ where γ_{Rd,k_1} is the value corresponding to k_1 and $\kappa_2 = \Phi^{-1}(k_2)$ where Φ is the standard normal cumulative distribution function.
Some improvement is also introduced in the partial safety factor for the action effects γ_{Sd}, which has been made dependent on the state of the structure, being equal to 1,0 for undamaged structures, and 1,15 otherwise.

3.2.3 Global verification and element types

Another major issue where significant advances have been made in the new EC8-3 is the introduction of the option to use *global verification*, alongside conventional local (member-level) ones. Very briefly, this means that in lieu of checking each individual member, the verification can be made on the basis of the resistance curve (pushover curve) of the structure, using a specified drop in strength (recommended value of 25%). Obviously, for such a verification to be feasible, nonlinear modelling of members should allow for strength deterioration. Such an approach should be supplemented by local verification for all failure modes not captured by the model. This is an important development, not only in an 'academic' or even conceptual context, but also in a very practical one, as it is particularly relevant and useful for structures like *masonry* buildings, and R/C (or steel) frames with *masonry infills*, where loss in strength of some masonry members does not correspond to a state of near collapse for the entire structure. Such structures will invariably be found to be unacceptable (and hence in need of strengthening) if assessed in a 'conventional' local way, even if some of their members are designated as 'secondary'.
A related new concept introduced in EC8-3 is that of "*Non-critical*", as distinct from the "secondary" elements of Part 1; the former can be neglected in modelling and verification, and be heavily damaged as long as they do not endanger life safety and the integrity of other structural elements and allow for the possibility of local repair. A typical example (and the trigger for introducing this type of elements) is a 'sacrificial' backwall in a bridge abutment.

3.3 Information for structural assessment

3.3.1 Knowledge levels

The issue of collecting and assessing information on the properties of an existing structure is one of the thorniest in structural assessment. Equally thorny is how the outcome of this effort will be translated into a 'knowledge level' (KL) and then into a 'confidence factor' that will affect the safety factors to be used in the assessment of the structure. Given the attention that this issue has received since the launching of EC8-3 (2005), it is not surprising that there are important changes in the pertinent clauses. First, in contrast to the single KL in the current EC8-3, the new Code defines three distinct KLs, one for Geometry (KLG), one for (construction) Details (KLD), and one for Material properties (KLM). These need not to be unique over the entire structure, which gives flexibility to the engineer dealing with the assessment, who may choose to strive for a higher KL in regions that are critical for the performance of the structure. Each of these KLs can be judged as: (i) Minimum; (ii) Average, or (iii) High, based on how thorough the corresponding *survey* is. The latter can be categorised as L (limited), E (extended) or C (comprehensive); e.g. a Limited survey of Geometry involves checking correspondence between the

actual geometry of the structure with the available outline structural drawings, by means of sample geometry measurements on selected elements; similar guidance is given for all other cases.
A well-known critical and somehow controversial issue is that of the minimum percentage of structural elements that should be surveyed for attaining a certain KL. The % is given by the (new) relationship

$$p = p_1 n^{-c} \leq 100 \qquad (1)$$

where n is the total number of elements of the type under consideration (beams, columns, etc.) in the structure. The coefficients p_1 and c are given in tables for each type of KL; as an example, Table 2 gives the coefficients for the case of geometric survey (dimensions of elements). Note that for, say, 20 columns and 40 beams at a certain storey, 42.8% of the columns, but only 27.5/2= 13.7% of the beams have to be surveyed, applying equation (1) in the case of Extended (E) survey. Values for p_1, c for KLM are material-dependent and hence are given in the pertinent clauses (8 to 11).

Table 2. Minimum requirements for different levels of geometric survey (vertical elements*)

Level of survey	***p_1***	***c***
Limited (L)	221	0,86
Extended (E)	291	0,64
Comprehensive (C)	236	0,44

* For horizontal elements, the percentage is halved.

Based on the above, the KL in each case (G, D, M) is defined as 1 (Minimum), 2 (Average) and 3 (High). Table 3 provides an example, i.e. how KLs are defined in the case of construction details; the role of availability of detailed structural drawings is clear, as when the set is complete, high KL (KLD3) is assumed even after a Limited survey, whereas if they are not available, a Comprehensive survey is required to achieve the same level (KLD3).

Table 3. KLD on Construction Details as a function of collected information

Original design documents (detailed structural drawings)	**Survey level***		
	L	E	C
Not available	KLD1	KLD2	KLD3
Incomplete set	KLD2	KLD3	
Complete set	KLD3		

Clause 5 also provides for the beneficial role of a '*preliminary' analysis* (this is linear elastic for reinforced concrete, steel frame structures and timber structures, and nonlinear static for masonry structures), which is not mandatory, but clearly encouraged, as a means for setting up an effective survey programme. If a preliminary analysis is carried out, further investigations on construction details (cl. 5.4.3) and material properties (cl. 5.4.4) may be limited to, or focus mainly on, the identified critical portions, in which case the total number of elements n should refer to the number of elements in the identified critical portions.
It is worth noting that the new EC8-3 abandons the (rather controversial) concept of the confidence factor; instead, in each material-dependent clause (8 to 11) the partial safety factors for resistance (γ_{Rd}) are simply defined as a function of the pertinent KLs.

3.3.2 Representative values of material properties

Long discussions took place within PT3 on another sensitive and somewhat controversial issue, that of the representative values of material properties to be used in the assessment, specifically whether verifications should be carried out in terms of mean or characteristic values of the properties. The following were finally adopted:

- *Mean* values should always be used for *existing* materials (in calculating resistance); these may be different in different areas of the structure. For KLM1 mean values may be obtained from standards

in force at the time of construction (for reinforcing steel and timber) or from information given in Annex E (for masonry).
- On the contrary, *characteristic* values should be used for new materials, in the spirit of treating them consistently with all other Eurocodes, in particular with EC8-1 for the design of new structures.

The above lead to the problem of how one should treat the case of members that include both existing and new materials, which is quite common in strengthening (e.g. R/C or FRP jackets). The final solution given to this thorny issue is that in the cases of *added* materials, design values of material properties for calculating resistances to be used in local verifications, should be defined as the *mean*. In this case the mean values of the added (new) materials are calculated from the corresponding characteristic values (specified in the pertinent codes); e.g. for concrete $f_{cm} = f_{ck} + 8$ MPa, while for steel and timber, the mean values are calculated using appropriate standard deviation values. The recommended values for *standard deviation* are as follows:

- Infill walls: 0,20 to 0,40
- Concrete: 0,10 to 0,20
- Reinforcing steel: 0,05 to 0,10
- Structural steel: 0,05 to 0,10
- Timber: 0,15 to 0,25
- Masonry: 0,20 to 0,30

3.4 Seismic action, methods of analysis and verifications

Clause 6 is strongly related to Part 1, hence harmonisation was a major issue right from the beginning and its structure has been modified to harmonise the chapter with cl. 6 of Part 1, as well as to ensure ease of use. In the writer's opinion, the most difficult issue (for the PT as well as for SC8) was to decide what should be left in Part 3, given that seismic actions are, of course, specified in Part 1 and the same, basically, applies for analysis methods as well. Hence, in principle, Part 3 should only contain any modification to the seismic action to be used in existing structures (no such modification was decided, but, of course, this is ultimately a national choice), and, mainly, the verification criteria to be used for existing ('old') members. Given that the expertise in structural engineering was much stronger in PT3 which included structural engineers only (while PT1 also had several experts in engineering seismology and geotechnical engineering), a significant amount of work was done in PT3 on nonlinear analysis methods, especially those for bridges that were not covered in the existing EC8-3. Pending further discussion on the issue, it was decided to proceed as follows:

- Details of *linear* methods of analysis are included in Part 1, and Part 3 makes reference to them.
- Details of *verifications* associated with *nonlinear* methods of analysis (static/dynamic) are included in Part 3, as these methods are inherently suitable for existing structures (nonlinear analysis of a structure can be carried out only when all its properties are known). Part 1 describes the procedures for applying the nonlinear methods, but for the nonlinear modelling and the verification of members made of specific materials (R/C, URM), in particular their deformation capacity, Part 3 has to be used. The same applies for the global verification procedure (see §3.2.3), which is meaningful mainly for existing structures.

Among the new material that was included in EC8-3, the writer considers as important the possibility to apply multimodal nonlinear static procedures to the assessment of bridges (this, should, in fact be also allowed for buildings). Specifically, it is mentioned that "In bridges that are irregular according to EN 1998-2:2005, 4.1.8, or where the modal participating mass ratio of the predominant mode (in the considered direction) is less than 70% of the total vibrating mass, higher mode effects in each principal direction should be taken into account. Unless nonlinear response history analysis is used, this requirement may be satisfied *using an appropriate multimode nonlinear static analysis*".

3.5 Design of structural interventions

The new clause 7 resulted from the merger of two fairly short clauses of the 2005 version (5 and 6), dealing with decisions and criteria for structural intervention. It was discussed, more than once, within the PT3 whether these clauses should be expanded to include detailed guidance on the selection of the intervention scheme, which would, in principle, be beneficial for the engineers dealing with the assessment and retrofit of an existing structure. Finally, the view initially expressed by the SC8 Chair was adopted, i.e. that the details of the selection of the strengthening scheme should be left to the judgement of the engineer, who should not be forced to follow predetermined procedures ('recipes') as to how to proceed with the strengthening scheme, but should have the freedom to apply other solutions that s/he would consider as more appropriate. The Code should provide specific criteria for *verifying* that, whatever the solution adopted, the resulting structure complies with the performance requirements specified in the Code.

In the light of the above, Clause 7 provides a clear description of the principles to be followed in the design of the intervention scheme, as well as an indicative list of 'conventional' intervention types (e.g. repair and strengthening of existing structural elements, addition of new structural elements, modification of the structural system, transformation of existing non-structural elements into structural elements, etc.) which were already in the 2005 version, as well as the introduction of *passive protection* devices through dissipative bracing or other dissipative devices, or seismic *isolation* at an appropriate level. For the isolation and dissipation techniques no further information is given in Part 3 and reference to the pertinent clauses of Parts 1 and 3 is made; recall that the new Part 1 contains detailed information on these issues, which, in principle, can also be applied to existing buildings, although it has to be admitted that the procedure to be followed is not identical in both cases. Combinations of the aforementioned procedures are allowed, but the choice of procedure is left to the Engineer.

3.6 Specific rules for reinforced concrete structures

This is a voluminous clause (the second longest after the one on URM) which replaces the existing Annex A, without actually changing its conceptual framework. Importantly, though, all R/C aspects that are currently considered as mature enough become part of the normative clauses of the Code, while a few issues that have not reached a sufficient degree of maturity (namely the prediction of ultimate chord rotation at the end of columns with section consisting of rectangular parts, without or with lap-splices and/or FRP) are allocated to Annex B of the new EC8-3. It has to be stated here (mainly for the record) that this important decision was far from easy to make, as there was fairly strong opposition to it, primarily from some of the 'low seismicity' countries that insisted on retaining all the details of the R/C deformation capacity etc. in an informative annex. Finally, the strong position of not only the PT3 but also of several other members in SC8 (including the 'senior management') prevailed and Clause 8 is expected to be one of the key parts of the new EC8-3, which will strongly influence the assessment and strengthening of R/C structures in the future.

While the concepts have not been changed, i.e. the deformation capacity of R/C members and their resistance with regard to non-ductile mechanisms (shear and inadequate anchorage) remain the core of this clause, most of the actual formulas have been updated. This was a more or less obvious requirement (also of national groups such as those of UK and Greece), the 2005 equations having become obsolete, as new ones were derived by the same groups, mainly that of University of Patras (e.g. Grammatikou et al. 2015) using more extensive experimental databases than those used in the past. Moreover, some formulas for FRP strengthening were taken from the recent Italian Code for FRP Strengthening of Existing Structures (CNR 2013a), while improved expressions were introduced for the monolithicity factors to be used in R/C jacketed columns (Thermou & Kappos 2016), accounting for the important parameter of the axial load level.

Besides including all the updated formulas, the new Cl. 8 has also a more rational (and user-friendly) structure. It starts with Identification of geometry, details and materials in existing R/C structures, followed by provisions for Structural modelling (with focus on stiffness); recall that, with the exception of the q-factor method, the stiffness of R/C members should be calculated on the basis of their yield moment – appropriately reduced if shear prevails - and their yield rotation. The Clause then describes in detail the Resistance models for assessment (also including lap splices regions and beam-column joint cores), and then, in a separate sub-clause, specific criteria for the Verification of Limit States are provided. Clause 8 concludes with important provisions for Resistance models for strengthening; of course, these cover only the commonly used techniques like R/C and Steel jacketing and FRP strengthening (jackets or strips).

3.7 Specific rules for timber structures

This is a totally new clause prepared by members of the SC8/WG3 (Timber), restructured for harmonisation by the writer and revised to a normative style by the SC8 Chairman. The clause follows essentially the same structure as the R/C and URM clauses (see §3.6 and 3.8), the main exception being that the 4th sub-clause (10.4) covers Resistance models for both assessment and strengthening (it will be split in due course). Currently (Jan. 2018) missing parts include force and displacement limitations for Timber frames, models for Single-step joints with tenon-mortise, and Dowel-type joints; there will also be an addition (regarding timber diaphragms) to the pertinent informative Annex (D).

A detailed presentation of all these new provisions is, clearly, beyond the scope of this paper; some general comments will be made instead. The main structural elements addressed in Clause 10 are rather different from those in other clauses; for instance, while the R/C Clause focuses on beams, columns (and, briefly, beam-column joints), and walls, the Timber Clause focuses on diaphragms, frames and two commonly used types of joints (carpentry and dowel-type). For these structural elements detailed information is given on both modelling and calculation of their resistance. The type of description is also quite different, i.e. it is implicitly assumed that the user is not familiar with the elements addressed, and descriptions and even photographs are provided, reminiscent of manuals rather than a code. But such issues are relatively easy to sort out, once the basic information is there. For instance, it is important that the present draft includes all necessary formulas for the calculation of the resistance of the afore-mentioned timber elements for different types of failure (as an example, for carpentry joints, formulas are given for the case of shear crack formation in the tie beam, and crushing at the front-notch surface). Equally important is that, in line with the displacement and deformation-based 'philosophy' of modern codes for structural assessment, the verification criteria are given in displacement terms. An example is shown in Table 4, that provides drift criteria for horizontal timber diaphragms; it is pointed out that such criteria are not restricted to as-built diaphragms only, but also cover all pertinent types of retrofit, i.e.: (a) additional diagonal sheathing; (b) structural wood-based panels; (e) CLT/LVL panels; (f) timber planks and additional diagonal sheathing.

Table 4. Acceptance criteria for horizontal timber diaphragms in terms of drift ratios [%]

	No retrofit	**Type of retrofit (a)**	**(b)**	**(e)**	**(f)**
Near Collapse (NC)	6,0%	2,1%	1,6%	1,5%	2,1%
Significant Damage (SD)	4,0%	1,5%	1,2%	1,1%	1,5%
Damage Limitation (DL)	2,5%	0,8%	0,7%	0,6%	0,8%

3.8 Specific rules for masonry structures

Clause 11 on URM buildings is the longest in the new EC8-3 and constitutes a major step forward with respect to the current Annex C, in more ways than one. It builds on the recent research (analytical and experimental) on assessment of masonry structures using modern analysis tools, with special emphasis on non-linear static (pushover) analysis, which is particularly suited for carrying out the global verification described in §3.2.3. It also benefits significantly from the recent detailed Italian Code on Assessment of URM Structures (Ch. 3 in CNR 2013b). There is new material covering in detail all aspects of modelling, calculation of resistance of masonry elements, and verification criteria. The only sub-clause that does not go to fine detail is the one on analysis and resistance models for strengthening. This is no surprise, given the very different types of strengthening procedures used in URM buildings and the fact that the corresponding research has clearly been much more limited than that on as-built masonry members and structures, hence this issue is not as mature as the previous ones and most of the information on URM strengthening techniques is provided in the Informative Annex E; this includes further valuable information, such as classification of masonry types not conforming with EN1996-1-1:2003 and reference values for the material properties, reference values for the equivalent in-plane stiffness of horizontal diaphragms of different types, drift capacity of masonry elements in the case of hybrid failure modes, and reference values for the material properties of strengthened masonry types.

Regarding modelling (cl. 11.3), the key issues are the following:

- Both in-plane and out-of-plane response of masonry walls are considered.
- In-plane behaviour of a masonry wall is modelled using the equivalent frame approach (line elements with rigid offsets for both piers and spandrels).
- Piece-wise linear force-deformation relations are adopted, with limited deformation; Figure 1 shows typical nonlinear constitutive laws (shear force vs lateral drift) to be adopted for URM piers and spandrels failing in diagonal tension.
- Horizontal diaphragms should be defined as rigid, stiff, or flexible.
- Global model is defined when diaphragms are rigid or stiff; in the case of flexible diaphragms each wall is analysed independently.
- Local out-of-plane mechanisms are considered using equilibrium limit analysis.
- In Nonlinear static analysis: (i) Ultimate displacement capacity δ_u is defined by checking the strength degradation at global level; (ii) If horizontal diaphragms are not rigid, it is also to be checked that the NC limit state is not reached in all piers at the same level of any masonry wall considered relevant.

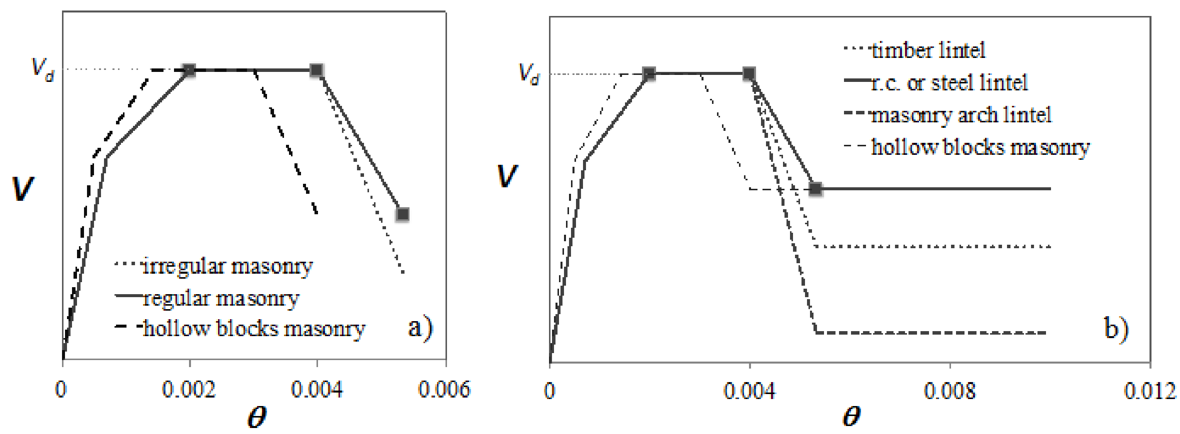

Figure 1. Force-deformation relationship for masonry piers (left) and spandrels due to diagonal cracking

Regarding Resistance models and verification of in-plane loaded masonry elements (cl. 11.4.1 and 11.5.1) the key issues are:

- Shear resistance of masonry elements (piers-spandrels) is the minimum among 3 possible alternative failure modes: flexure, shear sliding, diagonal cracking.
- Failure criteria are provided by considering the different behaviour of piers and spandrels.
- Masonry is classified as: (i) regular masonry (arranged through horizontal layers and stair-stepped mortar joints); (ii) irregular masonry.
- Drift limits are provided for all the above-mentioned cases, for damage levels of SD and NC.

Regarding modelling and verification of local mechanisms (cl. 11.4.2 and 11.5.2) the key issues are:

- Out-of-plane failure of portions of masonry walls not well connected to orthogonal walls and horizontal diaphragms is modelled by a kinematic mechanism of rigid blocks.
- Limit analysis provides the peak ground (or peak floor) acceleration that activates the rocking behaviour (DL limit state), and the principle of virtual work is applied.
- By considering the evolution of the mechanism (geometric nonlinearity), the pushover curve is obtained, and SD and NC limit states are defined.
- Safety verification is carried out in terms of displacements.

3.9 Specific rules for bridges

This (Cl. 12) is a new clause, wherein all information needed for the assessment and retrofit of bridges (additional to that given in all other clauses of EC8-3 and also in EC8-2 and the 'material-specific' Eurocodes) is gathered together. A separate clause is needed, not only because the design of bridges is a separate part for all Eurocodes, but also because bridges are, at least in certain respects, quite different from buildings, especially as far as seismic performance is concerned. Having said this, the SC8 did not see the necessity to have a different part of EC8 for Assessment and retrofitting of bridges, hence the mandate of PT3 was to include them in the same code as buildings. So, to make the issue clear through a specific example, if one wants to assess a concrete bridge, s/he will find information on the resistance and deformation capacity of concrete piers in Clause 8 (R/C), while information on how to find the axial load on the pier (e.g. for calculating the rotational capacity) will be found in Clause 12. To put the scope of this clause into context, reference is made to Table 5, which summarises all intervention types used in bridges. Clearly, EC8-3 covers only Types 3, 4, and 5.

Table 5. Intervention types for bridges

Type	Objective	Means
1	Durability	Local Repairs
2	Structural 'Non-seismic'	Various
3	Seismic Upgrading-1	Seismic Upgrading through Seismic Isolation (combined with additional Damping)
4	Seismic Upgrading-2	Seismic Upgrading through Strengthening
5	New Bridge	Replacement by a new bridge

Regarding the compilation of Information for Structural Assessment, six bridge components are identified: Deck, Piers, Foundation, Abutments, Bearings, Connections; a different Knowledge Level can be (and usually is) obtained for each component. The recommended procedure to be followed is:

- Step 1: Collection of information and first inspection
- Step 2: Simulated design (or reliable construction drawings)
- Step 3: Detailed Survey and Investigation

Information on all 3 steps is provided in cl. 12.4.2.

Regarding the Assessment procedure, depending on the bridge type, a different path has to be followed:

- *Single-Span Framed or Box-type Bridges*, wherein the main part of the seismic action comes from earth pressures acting on their abutments that are in contact with the backfill. In these bridges (usually underpasses) the seismic verifications should be based on a *deformation compatibility approach*, also used for other types of underground structures. Limit equilibrium conditions (i.e. Mononobe-Okabe formulation) can be invoked (EN1998-5:2004) for flexible walls, or a linear elastic solution for rigid or semi-rigid walls (see Annex E of EN1998-5:2004).
- *Bridges with two or more spans*: In this case analysis for seismic actions to determine the force and/or deformation effects is carried out according to Clause 6, possibly accounting for soil structure interaction. The resistance of existing, modified or new elements is assessed according to clauses 8 and 9 for concrete and steel/composite bridges, respectively.

Clause 12 also contains valuable information on Retrofitting of bridges. The general procedure for the design of interventions defined in other chapters (for R/C and steel) is also applicable to bridges. Guidance is also provided on the strategy for the intervention on each bridge component.

4. ACKNOWLEDGEMENTS

This paper is certainly not the outcome of the effort of the writer only, although it has been written by him. What is reported here is the collective work of all members of PT3 (see Table 1 for full list) which has greatly benefited from various contributions (oral and written) by the SC8 Chairman Ph. Bisch. The clause (10) on Timber Structures has been originally drafted by members of SC8/WG3, namely Maurizio Piazza and Ivan Giongo (University of Trento), Andrè Jorissen (Eindhoven Technical University), and Jorge Branco (University of Minho).
While a genuine effort was made to make this paper reflect the collective view of PT3, this does not

guarantee that there is agreement of each PT member with everything that is written herein. In fact, the writer is the only one to blame for any omission of important aspects or any possible misinterpretation of the intent of any of the provisions of EC8-3 or of any national or WG comments.

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