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CURRENT DEVELOPMENTS AND FUTURE PROSPECTS OF THE EUROPEAN CODE FOR SEISMIC DESIGN AND REHABILITATION OF BUILDINGS: EUROCODE 8

Michael N. FARDIS [1]

SUMMARY

The technical contents of the recently approved Part 1 of Eurocode 8: General rules, seismic actions, rules for buildings [1] and of draft Part 3: Assessment and retrofitting of buildings [2] are reviewed, along with the procedure for the upcoming enforcement of the 1st generation of European structural design standards. Possible future technical developments towards the 2nd generation of these parts of EC8 are highlighted.

INTRODUCTION: TOWARDS THE 1ST GENERATION OF EN-EUROCODES

Conversion of about 60 ENV-Eurocodes - published by the European Standards Organisation (CEN) in the early to mid-90s as European prestandards (ENVs) - into the 1st generation of European Structural Standards (EN) started in 1998. It entails a thorough revision of the ENVs on the basis of comments of National Standard Bodies and of recent scientific/technical developments. In the set of 10 Eurocodes, two cover the basis of structural design and the loadings ("actions"), one covers geotechnical and foundation design and five cover aspects specific to concrete, steel, composite (steel-concrete), timber, masonry or aluminum construction. Instead of distributing seismic design aspects to the Eurocodes on loadings, materials or geotechnical design, all aspects of seismic design are covered by Eurocode 8: "EN1998: Design of Structures for Earthquake Resistance" (EC8). This is for the convenience of countries with very low seismicity, as it gives them the option not to apply Eurocode 8 at all.

Eurocode 8 has 6 parts: Part 1 (EN1998-1): "General rules, seismic actions, rules for buildings"; Part 2 (EN1998-2): Bridges; Part 3 (EN1998-3): "Assessment and retrofitting of buildings"; Part 4 (EN1998-4): "Silos, tanks, pipelines"; Part 5 (EN1998-5): "Foundations, retaining structures, geotechnical aspects"; Part 6 (EN1998-6): "Towers, masts, chimneys". In March 2004 Parts 1 and 5 were officially approved as European Standards (ENs) by the 26 National Standard Bodies in CEN. It is expected that in early 2005 Parts 3 and 2 (in that order) will be approved as ENs, followed by Parts 4 and 6 around the end of 2005.

The European Commission considers the EN-Eurocodes as tools for the integration of the construction market and all related services in the European Union (EU) and the enhancement of competitiveness of European designers, contractors, consultants and construction products within and outside the EU. To this end, all parts of the EN-Eurocodes are developed to be fully consistent, user-friendly and seamlessly

[1] Dept. of Civil Engineering, Univ. of Patras, P.O. Box 1424, GR26500, GREECE, fardis@upatras.gr

integrated and to cover fully all types of civil engineering works, except nuclear power plants, off-shore structures and long-span (cable-stayed or suspension) bridges. They will be the basis of technical specifications for public works tenders or public procurements in EU countries and the related engineering services. Nonetheless, at least for the 1st generation of EN-Eurocodes, specification of national regulations as acceptable alternatives in such tenders will also be allowed, as long EU Member States retain them in parallel use with EN-Eurocodes. The European Commission will also expect Member States to promote the EN-Eurocodes in private contracts, instead of any national regulations that may retain in use and will urge them to phase out national regulations in the medium term.

Member States have the right to set the level of safety, serviceability and durability provided by structures built on their territory, taking into account aspects of economy. To allow them to exercise this right without sacrificing harmonisation of structural design codes at the European level and to accommodate geographic, climatic, etc. differences (including seismotectonic differences), "Nationally Determined Parameters" (NDPs) have been introduced in the EN-Eurocodes. NDPs are also used wherever consensus cannot be reached on aspects not related to safety, durability, serviceability or economy. The NDPs are:

- key parameters controlling safety, durability, serviceability or economy (safety factors, the mean return period of the design seismic action, etc.); the normative text of the Eurocode may provide a range of permissible values for them and will normally recommend a value in a non-normative note.
- technical classes (e.g. classes of ductility, classes of importance depending on occupancy or use).
- alternative procedures or methods (e.g. models of calculation).

Alternative NDP-classes and NDP-procedures/methods are identified and fully described in the normative text. A non-normative note may recommend one of the alternative NDP-classes or procedures/methods.

National choice regarding the NDPs will be exercised through the National Annex, to be published by each country as integral part of the national version of the EN-Eurocode. To minimize variation within the EU, the European Commission urges Member States to adopt for the NDPs the choice recommended in the notes. A National Annex may also contain country-specific data (a seismic zonation map, spectral shapes for the types of soil profiles provided in EC8, etc.) that are also NDPs. The decision to adopt nationally or not an Informative Annex of the EN-Eurocode may also be made there. If the National Annex does not exercise national choice for some NDPs, the choice will be left to the designer, taking into account the conditions of the project and other national provisions. What a National Annex cannot do is modify any EN-Eurocode rule or replace it with another – e.g. national – rule. Although discouraged, such a deviation from the EN-Eurocodes is allowed in national regulations other than the National Annexes. However, if the design includes such deviations, it will not be called "design according to EN-Eurocodes", as this implies compliance with all EN-Eurocode provisions, including the national choices for the NDPs. To help the user apply the EN, a National Annex may include reference to national documents for supplementary information, non-contradictory to the rules of the Eurocode. Examples for EC8 may be:

- acceptable values of member plastic hinge or chord rotations for the non-collapse performance level, (as a function of the geometry and the mechanical properties of the member) for design on the basis of nonlinear analysis, with direct verification of deformations, rather than of resistances.
- mechanical models for the (masonry) infill panels in infilled frames, for use whenever explicit modelling of such infills is desired or needed.

A Member State does not need to introduce a National Annex for a Eurocode part considered not relevant for its territory. This may be the case for EC8 in non-seismic or very low seismicity EU countries.

In less than one year after formal approval of an EN-Eurocode, EU Member States will have to translate it into their national language, or adopt one of the three CEN official versions (English, German or French). The deadline for publishing the National Annex – including the national choice for the NDPs – is two years after formal approval of the EN-Eurocode. Member States are expected to calibrate the NDPs during

these two years, so that, for the target safety level, structures designed according to the national version of the EN-Eurocodes will not cost significantly more than if designed with current National Standards. For transparency within the internal market, EU Member States will have to make clearly known to Eurocode users and any other interested party their national choice for the NDPs. Soon after the publication of all National Annexes, the European Commission intends to review the national choices for NDPs. Depending on the outcome, the European Commission may request EU Member States to accept a unique choice for as many NDPs as possible, especially for those ones for which national choices are not very different and/or the final design turns out to be not so sensitive to the national choices of the NDP.

EN Eurocodes will be applied in packages, each package referring to a specific combination of type of structure and construction material (e.g., concrete buildings, steel silos, tanks or pipelines, composite bridges, etc.). For self-sufficiency, each package will include EN1990: "Basis of Structural Design", EN1997: "Geotechnical Design" and Parts 1 and 5 of Eurocode 8, as well as the appropriate parts of EN1991: "Actions on Structures". Part 3 of EC8 will be included in all packages for buildings, Part 2 in all packages for bridges, Part 4 in the packages for "concrete liquid retaining and containment structures" and for "steel silos, tanks and pipelines" and Part 4 in the package for "Steel towers and masts".

Member States must withdraw national standards that compete or conflict with any EN-Eurocode part in a package, five years after the last EN-Eurocode in the package is published by CEN (i.e. three years after the deadline for publication of the National Annex of this last Eurocode part). This will be the beginning of the exclusive use of the EN-Eurocodes in the package as structural design standards in the EU.

A maintenance group will be set up for each EN-Eurocode after its official approval. Its task will be:
- to ensure consistency and co-ordination of on-going work on other draft ENs,
- to collect comments, provide explanations and answers to questions,
- to prepare the next phase of revision and updating of the EN, on the basis of feedback from its application and of scientific and technical developments.

The revision of the 1st generation of EN-Eurocodes and the development of the next one will be a natural continuation of the work of maintenance groups. To prevent confusion, a second generation EN will not be published by CEN, before the package of first generation of EN-Eurocodes it belongs to becomes the exclusive set of European structural design standards and conflicting national standards are withdrawn.

The paper focuses on the parts of EC8 devoted to the design of new buildings - Part 1: EN1998-1: 2004 [1] - or the assessment and retrofitting of existing ones - Part 3: draft prEN1998-3 [2]. Starting with the Introduction the paper summarizes recent developments and presents short- or mid-term prospects, either as these are planned, or as they are foreseen/sensed by the author on the basis of his experience in charge of the conversion of the six parts of Eurocode 8 from prestandards (ENVs) to European Standards (ENs).

PART 1 OF THE EN-EUROCODE 8 FOR SEISMIC DESIGN OF BUILDINGS

Introduction

Although its main object is buildings, Part 1 of the EN-Eurocode 8 includes also the general provisions for the other parts of EC8 to build on: Performance requirements, seismic action, analysis procedures, general concepts and rules applicable to structures beyond buildings. It covers in separate chapters concrete, steel, composite (steel-concrete), timber and masonry buildings, buildings with base isolation.

Performance Objectives: Seismic action and associated performance requirements

Part 1 of EC8 provides for a two-level seismic design, with the following explicit performance objective:
- Protection of life under a rare seismic action, by prevention of collapse of the structure or parts

thereof and preservation of structural integrity and of residual load capacity.

- Limited property loss in a frequent earthquake, via limitation of structural and non-structural damage.

The no-local-collapse performance level is achieved by proportioning and detailing structural elements for a combination of strength and ductility providings a safety factor between 1.5 and 2 against substantial loss of lateral load resistance. The damage limitation performance level is pursued by limiting the overall deformations (lateral displacements) of the system to levels acceptable for the integrity of all its parts (including non-structural ones) and through non-engineered measures for the integrity of (masonry) infills.

Although not explicitly stated, a third objective is the prevention of global collapse in an extremely strong earthquake, of the order of the "Maximum Considered Earthquake" (MCE) of US codes (although it is recognized that repair may be unfeasible or economically prohibitive and that the damaged structure may collapse in a strong aftershock). Resistance to extremely strong seismic actions is pursued by control of the inelastic response mechanism through systematic and across-the-board application of capacity design.

Within the philosophy of national authority on issues of safety and economy, hazard levels corresponding to the two performance levels above are left for national determination. For structures of ordinary importance the recommendation in Part 1 of EC8 is for:

- A seismic action for (local) collapse prevention with 10% exceedance probability in 50 years (mean return period: 475 years). This is the "design" seismic action; the "design" seismic action of structures of ordinary importance on rock is termed "reference" seismic action.
- A 10% in 10 years "serviceability" action for damage limitation (mean return period: 95 years).

Enhanced performance of essential or large occupancy facilities is achieved not by upgrading the performance level for given earthquake level, as in US codes, but by modifying the hazard level for which the performance level is pursued. For essential or large occupancy structures the seismic action at both performance levels should be increased so that its exceedance probability in 50 or 10 years, respectively, is lower than 10%. At the collapse prevention level the recommended value of the NDP-importance factor γ_I (to be applied to the reference seismic action) is 1.4 or 1.2 for essential or large occupancy buildings, respectively. A γ_I-value of 0.8 is recommended for buildings of reduced importance for public safety.

It is meant that the same spectral shape will be used for the seismic action for damage limitation as for collapse prevention, with a single multiplicative factor reflecting the difference in hazard level. The value of this factor should reflect national choice regarding protection of property, but also the regional seismo-tectonic environment. Values of 0.4 and 0.5 are recommended for this NDP-conversion factor, giving at the end approximately the same property protection for ordinary and large-occupancy buildings, less property protection for buildings of low importance (by 15-20% at the level of the seismic action) and higher property protection for essential facilities (by 15-20% at the level of the seismic action), possibly allowing them to operate during or immediately after a frequent event. In buildings, and unless most of the lateral force resistance will be provided by walls, the size of members will be controlled by the limitation of story drift ratio under the 10% in 10 years "serviceability" seismic action, calculated on the basis of the equal-displacement rule. The drift limit is 0.5% if the non-structural elements are brittle and attached to the structure, 0.75% if they are ductile, or 1% if they are not forced to follow structural deformations or do not exist at all. The 1% drift limit protects also structural members from extensive inelastic deformations under the "serviceability" earthquake. As the National Annex will set the level of "serviceability" earthquake, it will also determine to which extent these limits will control member dimensions. With the EC8-recommended values of 0.5x0.8=0.4 to 0.4x1.4=0.56 for the ratio of the "serviceability" to the "design" seismic action, the EC8 limits are 2 to 3 times stricter than in US codes.

The elastic response spectrum of the "design" seismic action is anchored to the "reference" ground

acceleration on rock, α_{gR}, to be mapped in national zoning maps. The spectrum includes regions where spectral acceleration, pseudovelocity or displacement is constant. The extent of each of these regions and the corresponding spectral amplitudes are taken to depend on the top 30m of the ground (which may be rock or various types of soil) and on the geologic features underneath. The standard ground types are:

- Type A: rock, with a lower limit on the average shear wave velocity in the top 30m, $v_{s,30}$, of 800m/s;
- Type B: very dense sand or gravel, or very stiff clay, with $v_{s,30}$ from 360 to 800m/sec;
- Type C: medium-dense sand or gravel, or stiff clay, with $v_{s,30}$ from 180m/sec to 360m/sec;
- Type D: loose-to-medium sand or gravel, or soft-to-firm clay, with $v_{s,30}$ less than 180m/sec;
- Type E: 5m to 20m thick soils with $v_{s,30}$ less than 360m/sec, underlain by rock.

Special studies are needed for the definition of the spectrum over soils containing at least 10m of soft clay/silt with high plasticity index and high water content, or over liquefiable soils, sensitive clays, etc.

The entire elastic spectrum is anchored to the mapped "reference" acceleration on rock multiplied by: a) the importance factor γ_I; b) a (NDP) factor S≥1 reflecting the effect of ground conditions on spectral values; and c) a correction factor for damping ζ other than 5%, equal to $\sqrt{10/(5+\zeta)}$.

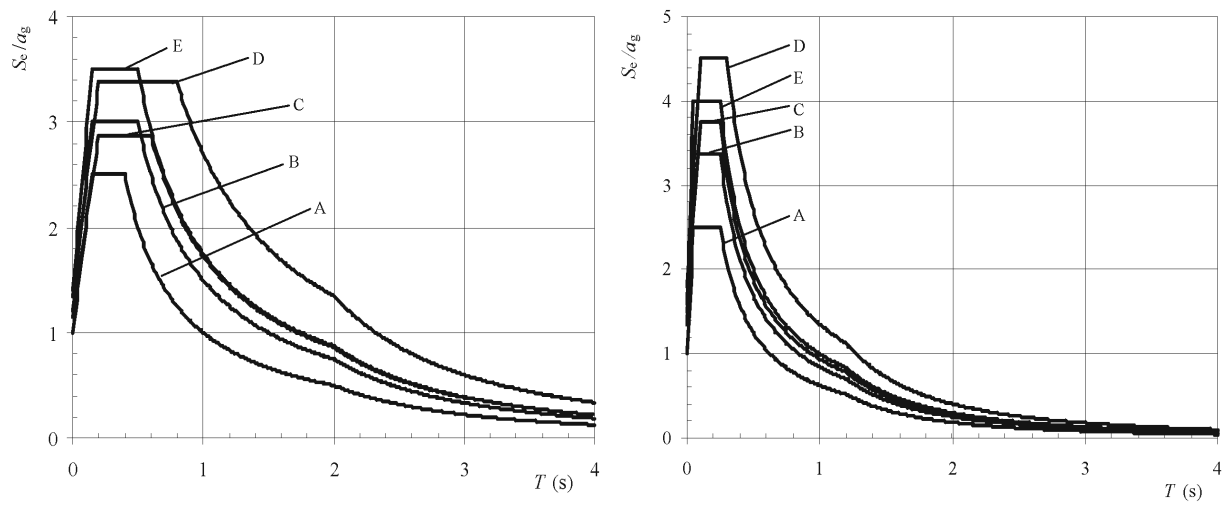

Figure 1. Elastic spectra of Type 1 (left) and 2 (right) recommended for the 5 standard ground types

The definition of spectral shapes for the standard ground types is left to National Annexes, depending on the magnitude of earthquakes contributing most to the hazard. In principle this allows a National Annex to introduce spectral amplification factors over each of the 3 regions of the spectrum – constant acceleration, pseudovelocity or displacement – which decrease with the absolute magnitude of the spectral value, due to the soil nonlinearity effect. This is the approach already taken in the USGS Seismic Hazard Maps to be used with all recent nationally applicable US documents. Instead of spectral amplification factors that decrease with increasing design acceleration (spectral or ground) as in recent US codes, the (non-binding) recommendation in Part 1 of EC8 is for two constant spectral shapes:

- Type 1 for moderate to large magnitude earthquakes;
- Type 2 for low magnitude ones (e.g. with surface magnitude less than 5.5) at close distances (producing over soft soil a motion rich in high frequencies).

Figure 1 shows the two recommended spectra types for ζ=5%, γ_I=1 and a PGA on rock of 1.0g. These normalized spectra were developed from a fairly large database of available records from Southern Europe and the Mediterranean area. They are believed to be currently more representative of the seismic hazard of that region than the more refined description of elastic spectra in recent US codes. Nonetheless, they are clearly not representative of intermediate depth earthquakes from the Vrancea region in Romania or the Aegean. Noteworthy is the rapid decay of the spectra in Figure 1 for long periods; the importance of

this decay is limited though by the lower limit of 20% of acceleration at zero period recommended for design spectral accelerations (i.e. after division by the behavior factor, q).

The 2nd generation of the EN-Eurocode 8 will most likely see a redefinition of the standard ground types, with possible subdivision of some of the current types, e.g. of C. It may also see seismic hazard maps for the entire European area, seamless at national boundaries. The maps should be in terms of 5%-damped elastic spectral values (for the constant acceleration, pseudovelocity or displacement regions), given at various different mean return periods, to let each European country exercise the right to choose the seismic action for the various performance levels foreseen in the different parts of EC8 (currently two in Part 1 or three in Part 3). It remains to be seen whether such maps will be included - along with the definition of the seismic action in terms of the response spectrum - in a special part of EN1991 ("Actions on Structures"), like the parts on snow or wind loads including the European snow and wind maps, or whether EC8 will remain the sole Eurocode covering aspects of seismic design, including the loading.

Design of buildings for controlled inelastic response and energy dissipation

The standard procedure in the 1st generation of the EN-Eurocode 8 is force-based design on the basis of the results of linear analysis for the 5%-damped elastic spectrum reduced by the "behavior factor" q.

EC8 allows neglecting the seismic action, if it is considered so low that design for the other actions provides also the necessary earthquake resistance. It is recommended (as NDP) that seismic design is not needed if the reference ground acceleration at the ground surface, Sa_{gR}, times the importance factor, γ_I, is less than 0.05g. EC8 allows also providing earthquake resistance by just proportioning members according to the other Eurocodes, for internal forces from an analysis for the seismic action with forces reduced by a behavior factor (q) of 1.5 (or 2 in some cases of steel or composite buildings) considered available due to overstrength, without any measures for local or global ductility. For non-base-isolated structures this simplification of design is recommended only for low seismicity regions. Although the designation of a region as one of low seismicity is left to the National Authorities as a NDP, a threshold value of 0.1g is recommended for the product of γ_I times the design ground acceleration at the ground surface ($\gamma_I S\alpha_{gR}$). Building structures designed with just the material Eurocodes and a q value of 1.5 (or 2 steel or composite) are termed "low-dissipative" or of Ductility Class (DC) low (L).

The majority of structures designed with EC8 will be designed for "energy dissipation". For buildings of the most common materials in earthquake-resistant construction, namely concrete, steel or composite (steel-concrete) buildings, two ductility classes (DCs) are foreseen for "dissipative" structures: Medium (M) and High (H) ductility. DC M and H buildings are entitled values of the behavior factor q for force reduction well above the value of q=1.5 (or 2 for steel or composite) attributed to overstrength. The global energy dissipation and ductility capacity needed for values of q (well) above 1.5 is ensured via:

- measures to control the inelastic response mechanism, so that concentration of inelastic deformation in a small part of the structure (mainly a soft story mechanism) and brittle failure modes are avoided;
- detailing of the plastic hinge regions for the inelastic deformations expected to develop there under the design seismic action (the one for which the no-local-collapse requirement applies).

Concentration of inelastic deformations and soft story mechanisms are avoided by configuring and proportioning the lateral-force resisting system so that vertical members remain practically straight – i.e. elastic – above their base. Concrete wall or dual systems and braced steel or composite frames are promoted and are capacity-designed to ensure that yielding takes place only at base of walls, only in the tension diagonals of concentric braced frames, or in the special "seismic links" of eccentric braced frames. In concrete, steel or composite moment frames, columns are capacity-designed to be stronger than the beams, with an overstrength factor of 1.3 on beam design flexural capacities in their comparison with that of columns. Concrete beams, columns and walls are capacity-designed against (brittle) shear failure.

Once the global inelastic response mechanism is controlled, the behavior factor q for reduction of elastic forces is related to the global displacement ductility factor, μ, via the Vidic et al [3] q-μ-T relation:

$$\mu_\delta=q \quad \text{if } T_1 \geq T_C, \qquad \mu_\delta=1+(q-1)T_C/T_1 \quad \text{if } T_1<T_C \tag{1}$$

where T_1 = building fundamental period and T_C = period at the upper limit of the constant acceleration spectral region. μ_δ can ultimately be related to the local inelastic deformation demands in structural members; e.g. for concrete members through the following approximation gives the local curvature ductility factor, μ_ϕ: $\mu_\phi=2\mu_\delta-1$. Member detailing aims at providing the local deformation capacity to meet these demands. For instance, the deformation capacity of flexural plastic hinges in concrete members is controlled through the confining reinforcement in columns or in boundary elements of walls, and through the compression reinforcement in beams, determined from μ_ϕ, which depends in turn on the q-factor

$$\mu_\phi=2q-1 \quad \text{if } T_1 \geq T_C \qquad \mu_\phi=1+2(q_o-1)T_C/T_1 \quad \text{if } T_1<T_C \tag{2}$$

Eq.(2) is based on Eq.(1) and on the approximation $\mu_\phi=2\mu_\delta-1$. So the spectrum of q-values gives rise to detailing that also varies continuously. The provisions of EC8 for proportioning and detailing of concrete members for energy dissipation in buildings of DC M and H are described in detail in Fardis [4].

The two upper Ductility Classes represent two different possible balances of strength and ductility, more or less equivalent in terms of total material costs and achieved performance at the collapse prevention level. DC M is slightly easier to design for and achieve at the site and may provide better performance in moderate earthquakes. DC H may give better performance under motions (much) stronger than the design seismic action. Unlike US codes, EC8 does not link selection of the ductility class to seismicity of the site or the importance and occupancy of the building, nor puts any limit to their application. The choice is left to the National Annex, which may in turn leave it to the designer, depending on the particular project.

Unless the Country objects through its National Annex, design directly on the basis of nonlinear analysis is allowed, with members verified by comparing directly deformation supplies to demands. The definition of acceptable member deformation limits is left to National Annexes, in the form of supplementary non-contradictory information. To ensure a minimum global and local ductility for the so-designed buildings, EC8 requires that they meet all DC M rules (i.e. those for member detailing, strong columns-weak beams in frames, capacity design of concrete members in shear, etc.). By allowing design directly on the basis of nonlinear analysis with member verification on the basis of deformations, the 1st generation of EN-Eurocode 8 paves the way for fully displacement- and deformation-based design in the 2nd generation. This approach is rapidly gaining acceptance in Europe, along with nonlinear analysis for the evaluation.

In the 2nd generation of EN-Eurocodes, the Section of Part 1 of EC8 on concrete buildings may see extension of its scope to cover prestressing in members with plastic hinging and energy dissipation, as well abandoning the curvature ductility factor as the link between global inelastic deformation demands (cf. Eq. (2)) and member detailing for ductility, in favor of more physically meaningful approaches.

Analysis procedures for the design of new buildings

Part 1 of EC8 includes the following analysis options for building design or assessment of performance:

- Linear static (termed “lateral force” method).
- Linear modal response spectrum analysis.
- Nonlinear static analysis (“pushover”).
- Nonlinear dynamic (response time-history).

Unlike US codes, where linear static analysis is the reference, in EC8 the linear modal response spectrum method is the standard procedure, applicable to all types of buildings. Countries are allowed to limit the use of nonlinear analysis methods through the National Annex.

The lateral force procedure may be applied if the effects of higher modes are not significant, i.e. only if:

- In both horizontal directions the fundamental period T_1 is less than 2 sec and 4 times the transition period T_C between the constant-acceleration and the constant-pseudovelocity regions of the spectrum.
- There are no significant irregularities in elevation.

Regarding irregularity, unlike US codes [5], [6], which set quantitative – albeit arbitrary – criteria based on the heightwise distribution of story mass, stiffness and strength, EC8 introduces qualitative criteria, easy to check at the preliminary design stage prior to any analysis:

- Lateral force-resisting systems continuous to the top of the – relevant part of the – building.
- Story mass and stiffness that is constant or reduces gradually and smoothly to the top.
- Individual setbacks less than 20% of the underlying storey if they are provided symmetrically on both sides of the building, or 10% if they are unsymmetric, etc.
- – In frame buildings – smooth variation of story overstrength with respect to the design story shear.

In the lateral force method of analysis the base shear is computed by assigning the total mass to the 1st translational mode in the horizontal direction of interest. It may use values of the fundamental period estimated through empirical expressions – the same as in the SEAOC '99 requirements (e.g. $T_1=0.085H^{3/4}$ for steel moment frames, $T_1=0.075H^{3/4}$ for RC frames or eccentric braced steel frames, $T_1=0.05H^{3/4}$ for all other systems, with H = height from base in m). Such expressions represent lower (mean minus one standard deviation) bounds to values inferred from measurements on buildings in California in past earthquakes and give low values of the period, especially for buildings with lower required earthquake resistance and hence less stiff. Use of a period calculated from mechanics – e.g. via the Rayleigh quotient – is encouraged (as preferred in European design practice), without a check or a limitation of the value with respect to the empirically calculated one. Moreover, if the fundamental period is less than twice the transition period T_C of the spectrum, in buildings with more than two stories the total lateral force is reduced by 15% - to account for the difference between the 1st mode mass and the total. The objective is to bring the results of the lateral force method closer to those of modal response spectrum analysis, and not the other way around as in US codes. The total lateral force is distributed to the stories following a 1st mode pattern of response accelerations, which may be taken as inverted triangular.

In the response spectrum analysis modal results are combined via rigorous application of the SRSS or CQC rule, i.e. at the level of the final seismic action effects of interest (displacements, internal forces, etc.). This is unlike the US approach, where modal response spectrum analysis is cast to look like linear static, in that story lateral forces are calculated for each mode (from modal story accelerations) and combined via the SRSS (or CQC) rule, for the structural system to be then analysed under the resulting lateral forces as in the linear static procedure. A drawback of the rigorous EC8 approach is that when the SRSS or CQC rule is applied to modal internal forces, signs are lost and, with them, physical meaning too.

Pushover analyses should be performed under two lateral load patterns: one corresponding to uniform lateral accelerations and another similar to the lateral forces used in linear static (lateral force) analysis, if applicable, or derived from a modal response spectrum one. The target displacement for pushover analysis is derived via the N2 procedure by Fajfar [7], given in an informative annex. The key elements in this procedure are: (a) the fitting of an elastic-perfectly plastic curve to the base shear-top displacement curve of the building; (b) establishment of an equivalent SDOF system on the basis of a lateral displacement pattern consistent with the postulated lateral load pattern and the calculation of its elastic stiffness as that of the elastic-perfectly plastic curve; and (c) estimation of the target displacement from the 5%-damped elastic spectrum, through the equal displacement rule, as modified for $T_1<T_C$ via Eq.(1).

Nonlinear response-history analysis should use as input at least 3 artificial, recorded, or simulated records (or 3 pairs of different records, for analysis in 3D), the mean elastic spectrum of which should no-where fall below that of the design seismic action by more than 10%. Results are averaged, if at least 7 such analyses are performed; otherwise the most unfavourable ones among the analyses performed are used.

In planwise regular buildings the analysis for the two horizontal components of the seismic action two may use independent 2D models. Before the analysis a building can be characterized as regular-in-plan if:

- it has rigid diaphragms, nearly rectangular in plan (re-entrant corners reducing floor area by not more than 5% each), with aspect ratio less than 4.
- the eccentricity between the story centers of mass and stiffness is less than 30% of the corresponding story torsional radius (square root of ratio of torsional to lateral stiffness, with stiffness parameters estimated in most cases from the moments of inertia of vertical elements), and
- at each story the torsional radius in each direction is less than the radius of gyration of the floor mass.

Two separate 2D models may also be used for buildings of ordinary importance with: (a) height less than 10m or 40% of the plan dimensions; (b) story centers of mass and stiffness approximately on (two) vertical lines, and (c) in both horizontal directions torsional radius not less than the SRSS of the radius of gyration of the floor in plan and of the projection of the eccentricity between centers of mass and stiffness in that direction. If conditions (a) and (b) are fulfilled, but not condition (c), then two separate 2D models may still be used, provided that all seismic action effects from the 2D analyses are increased by 25%.

Regardless of whether they are computed via a single 3D or two separate 2D models, seismic action effects due to the individual horizontal components are combined through the SRSS or the 1:0.3 rule. Maximum values of action effects estimated individually through the SRSS rule may be conservatively assumed to take place at the same time. Nonetheless, more accurate and less conservative rules may be introduced in the National Annex for the estimation of the likely simultaneous values of action effects due to the different components of the seismic action. In buildings which are regular in plan and have independent lateral-force-resisting systems in the two horizontal directions consisting solely of walls or bracing systems, the effects of the two horizontal components do not need to be combined.

Accidental eccentricity is taken equal to 5% of the perpendicular plan dimension, without amplification due to torsional-lateral coupling. Its effects may be calculated statically, by applying story torsional moments to a 3D structural model, even when the modal response spectrum method is used in the analysis of the response to the two horizontal components. The effects of accidental eccentricity may be accounted for in a simpler and conservative way by amplifying the results of the – linear static or dynamic, or nonlinear static – analysis for each horizontal component by $1+0.6x/L$, where x denotes distance of the element of interest from the center in plan and L the plan dimension, both normal to the direction of the seismic action. This factor is derived assuming that torsional effects are fully resisted by the lateral-force-resisting-elements in the horizontal direction considered and that such elements are uniformly distributed in plan. In planwise regular buildings analysed with two separate 2D models, factor 0.6 is replaced by 1.2.

Unlike what happens in some US codes, amplification of eccentricities between centers of mass and stiffness is not required. This is convenient, as in most cases story stiffness centers cannot be uniquely defined; moreover, determination of their position at a level of accuracy and sophistication consistent with the dynamic amplification of natural eccentricity, requires tedious additional analyses.

In all types of analysis the model should include only elements considered as part of the lateral-force-resisting-system, termed “primary seismic elements”. “Secondary seismic elements” should collectively account for less than 15% of the total lateral stiffness and should be distributed regularly and uniformly in plan and elevation, so that they don’t affect the regularity classification of the structural system.

In linear analysis the elastic stiffness should be the secant stiffness to yielding, which for concrete and masonry may be taken as half of the uncracked stiffness of the gross section. Nonlinear analysis may use this value as pre-yield stiffness (pre- and post cracking stiffnesses may be considered also, if so-desired), and may neglect the effect of strain hardening on post-yield stiffness (post-yield stiffness should be taken

negative, if there is significant strength degradation). Hysteresis models for nonlinear response-history analysis should reflect realistically the energy dissipation within the expected range of displacements. Nonlinear element models should use mean material properties, which are higher than nominal values.

For linear analysis displacements are calculated in general on the basis of the equal displacement rule, i.e. by multiplying linear analysis results by the q-factor. Allowance is made for more accurate calculation, including the q-μ-T relation of Eq.(1). Story drifts determined in this way are used to estimate the ratio of P-Δ effects to 1st-order effects. If the ratio $\theta=P\delta/Vh$ exceeds 0.1 in any story, 1st-order analysis results are divided by $1-\theta$. P-Δ effects are calculated on the basis of the secant-to-peak-drift story stiffness, instead of the elastic stiffness used in US codes, influencing analysis results more often and to a larger extent.

Non-engineered masonry infills producing irregularities in plan or elevation should be taken into account in the analysis of steel or composite moment frames of DC H, as well as in DC H concrete buildings with walls providing less than 50% of lateral force resistance. Normally, irregular distribution of infills in plan may be considered by doubling the effects of accidental eccentricity. Very irregular arrangements, as e.g. when infills are concentrated mainly on one or two sides of the perimeter, require instead a 3D model that explicitly includes the infills, and a sensitivity analysis of the effect of their properties and position (including removal of one out of 3 or 4 infill panels from the model). Attention is called to the possible effects on structural elements furthest away from the sides of the plan where infills may be concentrated. A reduction in the amount of infills relative to the story above (as in open ground-stories) should be taken into account by amplifying seismic action effects from the analysis so that the deficit in infill shear strength in the story is compensated by the increase in resistance of vertical members of the frame there.

The current trend in Europe, is towards nonlinear analysis methods: the traditional response-history version and the more fashionable pushover analysis. In the draft Part 3 of EC8 pushover analysis is the reference method for the assessment of existing buildings and the evaluation of their retrofitting. If the future is the continuation of the past and present, in the 2nd generation of EC8 nonlinear analysis methods will assume greater importance for the design of new buildings as well. This evolution will depend not so much on developments in nonlinear models for members under monotonic or cyclic loading, but on the emergence and wide acceptance of member verification criteria based on deformations. To be established as the prime workhorse for seismic design of new concrete buildings, nonlinear analysis will have to overcome a difficulty specific to concrete: that key parameters of nonlinear member models (e.g. the pre-yield stiffness and the yield moment) depend on the amount of reinforcement, which is not known prior to detailed design. Ad hoc procedures may be used to overcome this difficulty: e.g. beam reinforcement may be first proportioned for gravity loads, that of columns may then be obtained from the strong column-weak beam rule, accounting for minimum reinforcement, to provide an estimate of member pre-yield stiffness and yield moments for the (first round of) nonlinear analysis, that will give the deformation demands for which members will be verified and their transverse reinforcement proportioned and detailed.

Another point where there is room for improvement of the present EC8 rules is in the analysis methods and approximations for the inelastic torsional response due to natural or accidental eccentricities in plan.

Behavior factor q for reduction of the forces from elastic analysis

In structures designed for energy dissipation the behavior factor, q, that reduces the forces in the elastic structure, is linked, directly or indirectly, to the ductility and deformation demands on members and connections, and hence to the corresponding detailing requirements. The value of q depends on the structural material, the type of lateral-force-resisting-system and the ductility class selected for the design. The values quoted here are the ones adopted by EC8 for the most common structural materials in earthquake resistant construction, namely for concrete, steel and composite (steel-concrete) buildings.

Overstrength due to materials is considered to account for a q-factor of 1.5 (or up to 2 for steel or composite buildings), even in buildings without energy dissipation. In buildings with energy dissipation and structural system redundancy the q-factor incorporates a factor α_u/α_1 (denoted here for convenience by α_R) that accounts for (structural) system overstrength. The α_R factor is the ratio of the seismic action at development of a full plastic mechanism (i.e. of a fully yielded structure) to the one that causes formation of the first plastic hinge in the system – in both cases with the gravity loads acting simultaneously with the seismic action. Default values of α_R in buildings regular in plan (defined in the previous Section) are:

- 1.0 for: concrete wall systems with just two uncoupled walls per direction;
- 1.1 for: (a) one-story moment frames, (b) wall concrete systems with more than two uncoupled walls per direction, (c) steel or composite frames with concrete infills, (d) wall concrete systems with encased steel sections as wall boundary elements, (e) systems of concrete walls - possibly with encased steel sections as boundary elements - coupled by steel or composite beams;
- 1.2 for: (a) one-bay multistory moment frames, (b) dual concrete systems with walls providing 50% to 65% of lateral force resistance, (c) coupled wall concrete systems, (d) eccentric-braced steel or composite frames, (e) steel or composite systems consisting of concentric-braced and moment frames;
- 1.3 for multistory multi-bay frames, (f) systems of walls with steel or composite boundary elements and web consisting of a steel plate with concrete overlay on one or both sides.

In buildings that are not regular in plan, the default α_R-value is the average of 1 and the default value listed above. Values higher than the default may be used (up to a maximum of 1.5 for concrete, or of 1.6 for steel and composite), if they are confirmed through a pushover analysis of the so-designed structure.

Table 1. q-factors for structural systems regular in elevation

Material	Lateral-load resisting structural system	DCM	DCH
steel or composite	inverted pendulum systems (≥50% of mass in upper third of height, or with energy dissipation at base of single element)	2	$2\alpha_R$
steel or composite	moment-resisting frames, or eccentrically-braced frames	4	$5\alpha_R$
steel or composite	frames with concentric diagonal or X-bracings (with only the tension diagonals considered for seismic resistance)	4	4
steel or composite	frames with concentric V-braces (with both tension and compression diagonals considered for seismic resistance):	2	2.5
steel or composite	moment-resisting frames combined with concentric (diagonal, V or X) bracings	4	$4\alpha_R$
steel or composite	moment-resisting frames with concrete or masonry infills in contact with frame, without positive connection	2	2
composite	1) steel or composite frames with connected concrete infills; 2) concrete walls with encased steel sections as boundary elements at the edges; or 3) walls with steel or composite boundary elements and web of a steel plate with concrete overlay(s)	$3\alpha_R$	$4\alpha_R$
composite	1) concrete walls of the type (2) above; or 2) concrete walls coupled through steel or composite beams	$3\alpha_R$	$4.5\alpha_R$
concrete	inverted pendulum systems (defined above for steel or composite)	1.5	2
concrete	torsionally sensitive (radius of gyration of floor mass > torsional radius in one or both directions: i.e. lateral-load-resisting system concentrated near plan center)	2	3
concrete	system of walls with resistance ≥65% of seismic base shear and with >50% of wall resistance from uncoupled walls (: coupling reduces wall base moments by <25%)	3	$4\alpha_R$
concrete	all concrete systems other than above	$3\alpha_R$	$4.5\alpha_R$

For concrete, steel or composite buildings that are regular in elevation, the q-factor for Ductility Class M

or H is given in Table 1. This is the "basic" value, q_o, of the q-factor, used to estimate the ductility demands in element regions intended for energy dissipation and detailed accordingly. In buildings with vertical irregularities, seismic action effects from linear analysis are calculated from a q-factor equal to 80% of the "basic" value, q_o, but not less than the value of 1.5 for concrete, or of 2 for steel or composite, which is considered available due to overstrength alone. A building may have different q-factors in the two main horizontal directions, depending on the structural system and its vertical regularity classification in these directions, but not due to Ductility Class, which is the same for the whole building.

Design of the foundation and of its elements

Unlike US codes [5], [6], which allow reduction of overturning moment at the base due to uplift by 25% for linear static analysis or by 10% for a response spectrum one, in DC M and H buildings verification of the foundation soil and proportioning of foundation elements are also based on seismic action effects derived from capacity design. This is due to the importance of the foundation for the integrity of the whole and the difficulty to access, inspect and repair damaged foundations. Capacity design seismic effects, to be superimposed to those due to gravity loads, are the – factored by 1.2 – seismic action effects which occur at the intended plastification of the component of the superstructure with the highest influence on the foundation element in question (i.e. the base of a frame column or wall in the direction which yields first in bending). For common foundations of more than one vertical element (e.g. rafts, foundation beams, etc.) plastification in the element with the largest design seismic shear is considered in these capacity-design calculations. Foundation elements (including piles) proportioned for the above capacity-design load effects, follow the detailing etc. rules of DC L structures, i.e. design according to Eurocode 2 alone. Alternatively, they may be proportioned for seismic action effects derived from the analysis; then they should meet all the proportioning and detailing rules of the corresponding DC for members of the superstructure (e.g. capacity design in shear for foundation- or tie-beams).

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Levels of Performance (Limit States) and Hazard Levels (Seismic action)

Part 3 of EC8 [2] adopts a fully performance-based approach for the evaluation and rehabilitation of existing buildings; three performance levels (termed in the European tradition "Limit States") are defined:

- "Near Collapse" (NC), similar to the "Collapse Prevention" level in US codes [7], [8]. The structure is heavily damaged, may have large permanent drifts, retains little residual strength and stiffness against lateral loads, but vertical elements can still carry the gravity loads. Most non-structural elements have collapsed. The building may not survive another earthquake, not even a strong aftershock. Repair may not be technically feasible and is not economically justified.
- The "Significant Damage" (SD) level, corresponding to the "Life Safety" level in the US [7], [8]and to the local-collapse prevention performance level for which new structures are designed according to Part 1 of EC8. The structure is significantly damaged, may have moderate permanent drifts, but retains some residual lateral strength and stiffness and certainly its vertical load-bearing capacity. Non-structural components are damaged, but have not collapsed. Repair may be uneconomic.
- The "Damage Limitation" (DL) level, corresponding to the "Immediate Occupancy" level in the USA [7], [8]. The structure is only lightly damaged, has no permanent drifts and its elements have no permanent deformations, retain fully their strength and stiffness and do not need repair. Non-structural components may have distributed cracking and can be easily and economically repaired.

To allow national determination in all safety-related issues, the "Seismic Hazard" levels for which the three "Limit States" will be required are NDPs. For ordinary buildings, the recommendation in EC8 is for a 225-year earthquake (20% exceedance probability in 50 years), a 475-year event (10% in 50 years) and a 2475-year one (2% in 50 years), for the DL, the SD and the NC "Limit State", respectively.

Performance differentiation of essential or large occupancy buildings from ordinary ones is effected as in new buildings: by multiplying the seismic action with the "importance factor", which has the recommended values given in Part 1. The same spectral shape applies at all "hazard levels". It is noteworthy that as the SD "Limit State" conceptually corresponds to the local-collapse prevention level for which new structures are designed according to Part 1 of EC8, Parts 3 and 1 recommend the same seismic action for both, without differentiation of the "Performance Objective" between new and existing buildings. Any such differentiation comes through the "compliance criteria" specified in Parts 1 and 3 for this single performance level. National Authorities are free to relax this requirement for existing buildings (by specifying in the National Annex a lower seismic action for the SD "Limit State"), to reduce the number of buildings that need retrofitting and/or make retrofitting more attractive from the economic point of view. National Authorities will also specify whether all three "Limit States" will have to be verified under the corresponding "Seismic Hazard" level, or whether verification of one or two "Limit States" for the corresponding "Seismic Hazard" level may suffice.

Analysis methods for the calculation of deformation demands and criteria of applicability

Part 3 of EC8 has adopted a fully displacement-based approach. So the objective of the analysis for the seismic action of interest is the estimation of deformation demands in structural members.

Part 3 provides the same four analysis options of Part 1 of EC8. In all analysis methods the seismic action is given by the 5%-damped elastic response spectrum or the quantities of interest derived from it: the target displacement for nonlinear static analysis, or acceleration time-histories for nonlinear dynamic analysis. The spectrum is anchored to the PGA corresponding to the "hazard level" chosen for the "Limit State" for which analysis results will be used, with multiplication by the "importance factor" appropriate for the building. All types of analysis, except the nonlinear dynamic, employ the equal-displacement rule, at the level of member deformations, e.g. chord-rotation demands for the first two types of analysis, or of the displacement of an equivalent SDOF system for pushover analysis.

If linear analysis is applied, internal forces in "brittle" members are estimated as in "capacity-design", i.e. from equilibrium assuming that "ductile" locations delivering force to them develop their force capacity, or the force demand from the analysis, whichever is less. Force capacities are estimated from expected values of material strengths derived from the available information, multiplied by a "confidence factor" greater than 1 that depends on the amount and reliability of information on the as-built structure.

The criteria for applicability of linear-elastic analysis, static or dynamic, are the following:

- In all "brittle" elements force capacity exceeds demand estimated according to the paragraph above.
- There is a fairly uniform distribution of ductility demands over the entire structure. If the ratio D/C of bending moment at the end of a member from elastic analysis to the corresponding flexural capacity is taken as a measure of the local ductility demand (this ratio is roughly equal to the demand value of the chord-rotation ductility ratio), then the maximum value of the D/C-ratio over the entire structure should not exceed the minimum value of the D/C-ratio over the structure by a factor between 2 to 3. the maximum permissible value of the D/C-ratio within this range is a NDP with a recommended value of 2. In this criterion D/C values less than 1 are taken equal to 1, indicating elastic response.

If both these conditions are met, the relevant criteria of Part 1 of EC8 determine whether linear static analysis ("lateral force" procedure) may be used instead of the modal response spectrum method. No limit is set for the absolute value of the ductility demands of "ductile" elements for linear analysis to be applicable. Nonetheless, the limitation of the ratio of maximum to minimum value of the D/C-ratio below a ceiling between 2 and 3 (with a recommended value of 2) poses a very severe restriction to the applicability of linear-elastic analysis (except at the DL "Limit State", where elastic response is required).

Each one of the four analysis methods should follow the rules specified in Part 1 for new buildings.

According to these rules the elastic stiffness should be the secant stiffness to yielding, which Part 1 allows taking equal to half of the uncracked stiffness of the gross section in concrete or masonry buildings. This rule is meant to provide safe-sided estimates of force demands in force-based design; nonetheless, it underestimates the effective secant-to-yield rigidity of members and provides unconservative estimates of chord rotation demands for verification in displacement-based evaluation and rehabilitation, as in Part 3 of EC8. To avoid this, the secant-to-yield rigidity of concrete members may be determined from the yield moment and the chord rotation at yielding, estimated through semi-empirical expressions given in the informative Annex of Part 3 devoted to concrete buildings.

Compliance criteria for the three "Limit States"

Following Part 1 of EC8, Part 3 distinguishes the elements considered as part of the lateral-force-resisting system ("primary") from those which are not ("secondary"). The restriction in Part 1 that "secondary" elements should collectively account for not more than 15% of the total lateral stiffness is waived. As more relaxed compliance criteria apply for "secondary" elements, the engineer is free to designate elements of the existing or of the retrofitted building as "secondary", depending on the outcome of rounds of analysis and verifications with alternative trial designations of some elements as "primary" or as "secondary". What the engineer is not allowed to do is to deliberately choose the distribution of "secondary" elements in plan and elevation so that the regularity classification of the structural system changes from irregular to regular (as this may affect the method of analysis allowed to be applied).

A distinction is also made in Part 3 between "ductile" and "brittle" elements. In concrete buildings members are characterized as "ductile" in bending and as "brittle" in shear. Compliance criteria of "ductile" elements are in terms of deformations, while those of "brittle" elements are in terms of forces:

- At the NC "Limit State" "ductile" elements are allowed to reach their (expected) ultimate deformation capacity. The Annex on concrete buildings specifies this as the ultimate chord rotation capacity, computed from mean material strengths divided by a "confidence factor" that depends on the amount and reliability of the information available for the as-built structure; moreover an overall safety factor of 1.5 is applied to the ultimate chord rotation capacity of "primary" elements. Force demands on "brittle" elements should remain below their ultimate force resistance, computed from mean material strengths. For primary concrete elements these mean material strengths are divided by the material partial factors times the "confidence factor" that depends on the amount and reliability of information; moreover, the so-computed shear resistance is further divided by a safety factor of 1.15. For secondary elements, factoring of mean material strengths by material partial factors or "confidence factors", or of the shear resistance by an additional safety factor, is not required.
- At the SD "Limit State" deformations (chord rotations at member ends) of "ductile" elements are limited to a fraction of their ultimate deformation capacity. The Annex for concrete buildings specifies this as 75% of the ultimate chord rotation capacity, with the additional safety factor of 1.5 for "primary" elements. In the force-based verification of "brittle" elements the capacity is taken the same as in the NC "Limit State" and as the demand is also (essentially) the same, verification at the SD "Limit State" may be omitted as redundant, if already done at the NC "Limit State".
- At the "Damage Limitation" (DL) level, "ductile" and "brittle", "primary" or "secondary" elements alike, are required to remain elastic (below yielding).

The informative Annex on concrete buildings provides empirical or semi-empirical expressions for the ultimate chord rotation of concrete members, as affected by their geometry, the amount and detailing of (longitudinal and transverse) reinforcement, the presence and length of lap splices, etc. It gives also semi-empirical expressions for their shear resistance, as controlled by yielding of stirrups or (for "short columns", i.e. those with shear-span-ratio less than 2, and for walls) by diagonal web crushing, and as it decreases with the magnitude of post-yield cyclic deformations. Expressions and design rules are also given for the enhancement of member ultimate chord rotation and shear resistance effected by jackets of

concrete, steel or fiber-reinforced polymers (FRPs).

Prospects for the evolution of Part 3 of Eurocode 8 towards the 2nd generation of EN-Eurocodes

Unlike Part 1 of the EN-Eurocode 8, which has evolved from the ENV version of 1994 with extensive changes in rules, addition of many new provisions and significant enlargement of the scope, but without radical changes in the overall philosophy, the draft Part 3 represents a revolutionary change over its 1996 predecessor. Therefore, technical developments towards the 2nd generation of Part 3 of EC8 will depend on how this novel 1st generation EN-Eurocode will be received and applied in European seismic rehabilitation practice. As another difference with Part 1 - which consists almost exclusively of normative text or Annexes albeit with a few dozens of NDPs - Part 3 has a brief and general normative part with very few NDPs, but three long and very detailed informative Annexes - one for each of the three main materials: concrete, steel or composite and masonry. As Member States may accept informative Annexes as source of information or replace them with other (national) alternatives, the form and the technical contents of the 2nd generation of Part 3 of EC8 will strongly depend on how the three Annexes will be received in Member States. If they are accepted as source of information in practically all Member States publishing a National Annex for the EN-Eurocode 8, it is very likely that the European Commission will enrich the limited normative part of Part 3 of EC8 by upgrading (some of) the three Annexes to Normative, even before launching the 2nd generation of EN-Eurocodes.

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