Eurocode 8 Part 3
Assessment and retrofit of buildings

Paolo Emilio Pinto
University of Rome La Sapienza
Italy

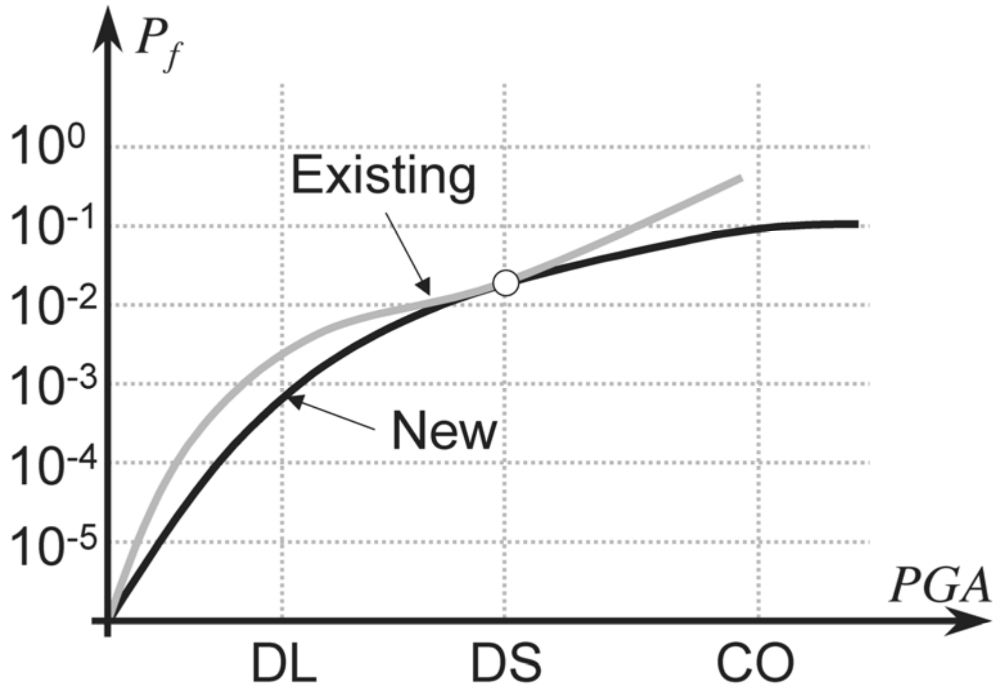
Performance requirements

Hazard (return period of the design spectrum)	Required performance
T_R=2475 years (2% in 50 years)	Near Collapse (NC) (heavily damaged, very low residual strength & stiffness, large permanent drift but still standing)
T_R=475 years (10% in 50 years)	Significant damage (SD) (significantly damaged, some residual strength & stiffness, non-strutural comp. damaged, uneconomic to repair)
T_R=225 years (20% in 50 years)	Limited damage (LD) (only lightly damaged, damage to non-structural components economically repairable)
TR values above same as for new buildings. National authorities may select lower values, and require compliance with only two limit-states	

Motivation for a 3rd, higher level of hazard

Contrary to new, code designed, buildings, existing ones may not have adequate margins to resist seismic actions higher than the design one

The additional "point check" is intended to ensure that "new" and "existing" have the same "total risk"



Compliance criteria

EC8 Part 3, 2.2.1(1)P:

"Compliance with the requirements in 2.1 is achieved by adoption of the seismic action, method of analysis, verification and detailing procedures contained in this Part of EN1998"

Remarks:

- **The criteria are not consistent with the definitions of the LS's. The NC-LS, for ex., is described as a state of severe damage extending over the entire structural system, and such as to bring it close to collapse**
- **If the verifications would have to be satisfied for all individual primary elements, very few existing buildings would be exempted from some form of intervention**

Compliance criteria

A more consistent framework:

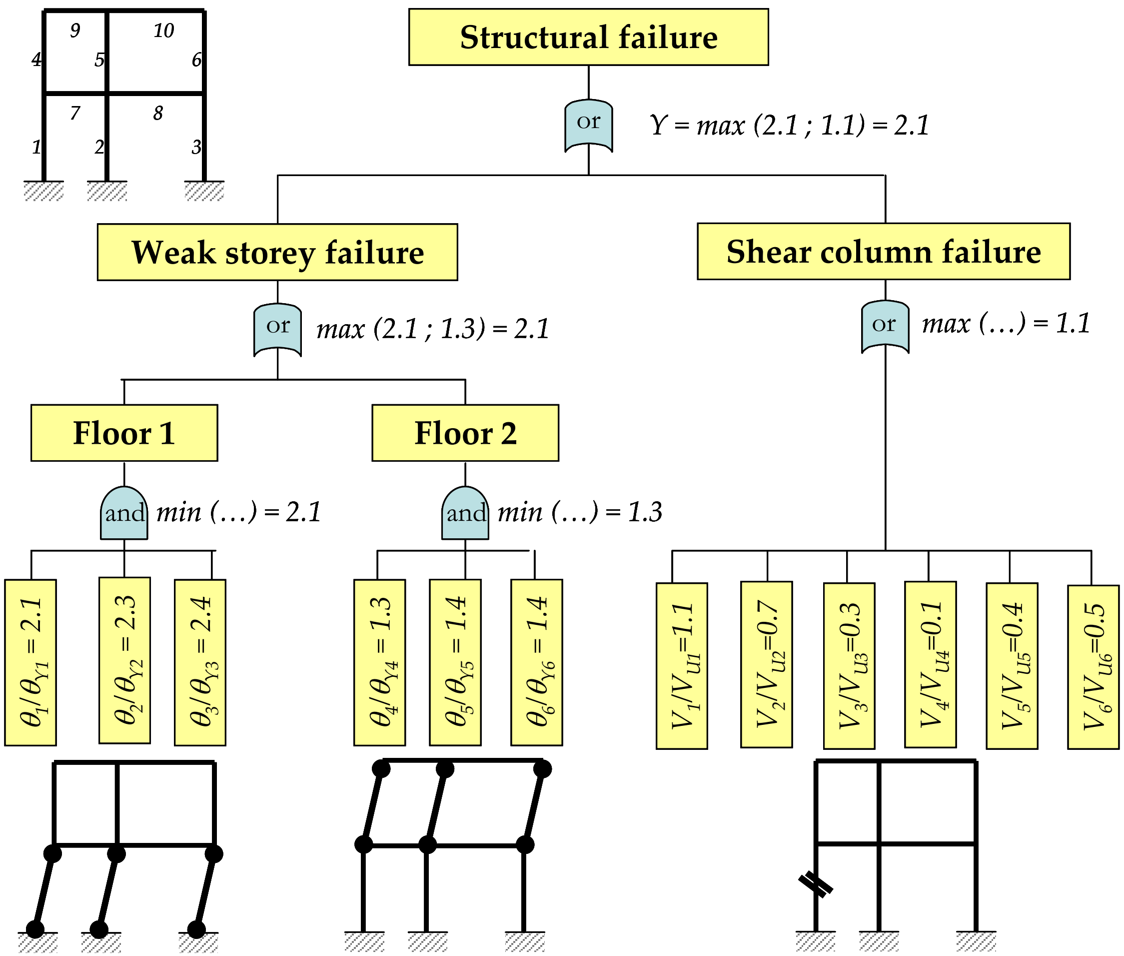
- The analyst should identify a number of structural situations that are realistically conducive to the LS under consideration
- Such situations depend on the building topology and involve in general both single components and specific groups of components

The ensemble of critical situations is conveniently arranged in the classical form of a fault tree. In the fault tree representation the state of the system is described as a serial arrangement of sub-systems, some of which are made of a number of components working in parallel

Compliance criteria

A more consistent framework:

example of a fault tree representation for the NC-LS of a simple frame



Compliance criteria

A more consistent framework:

with reference to a fault tree representation as in the example, the *state* of the system is determined by the value of a scalar quantity defined as:

$$Y=\max_{i=1,N_S}\min_{j=1,N_i}R_{ij}$$

where:

- R_{ij} = ratio between demand and capacity at the j-th component of the i-th subsystem
- N_S = total number of sub-systems
- N_i = number of components in subsystem i

$Y=1$ implies attainment of the LS under consisderation

Treatment of uncertainty

Knowledge levels (KL) and Confidence factor (CF)

Knowledge Level	Geometry	Details	Materials	Analysis	CF
KL1		**Simulated design in accordance with relevant practice** *and* **from** limited ***in-situ*** **inspection**	**Default values in accordance with standards of the time of construction** *and* **from** limited ***in-situ*** **testing**	**LF-MRS**	CF_{KL1}
KL2	**From original outline construction drawings with sample** visual **survey** *or* **from** full **survey**	**From incomplete original detailed construction drawings with** limited ***in-situ*** **inspection** *or* **from** extended **in-situ inspection**	**From original design specifications with** limited ***in-situ*** **testing** *or* **from** extended ***in-situ*** **testing**	**All**	CF_{KL2}
KL3		**From original detailed construction drawings with** limited ***in-situ*** **inspection** *or* **from** comprehensive ***in-situ*** **inspection**	**From original test reports with** limited ***in-situ*** **testing** *or* **from** comprehensive ***in-situ*** **testing**	**All**	CF_{KL3}

The confidence factor

Given a KL, the corresponding CF value applies for the verifications of all LS's

Uses of CF:

- In the evaluation of the member capacities, as an amplifier of the ordinary °-factors of the materials, to account for the additional uncertainty

- In the evaluation of the demand on the brittle mechanisms, in the same way as the "capacity design factors" $°_{Rd}$, to account for the higher protection required

The confidence factor: limits of the approach

An additional partial material factor, as the CF, is appropriate to cover the (generally) larger uncertainties on existing structures

Uncertainties due to lack of knowledge such as, for example, the ignorance on whether a structural detail (or even a structural member) is present at all, cannot be accounted for by means of a CF

Structural reliability theory offers standard tools for dealing with this different kind of uncertainty (epistemic uncertainty)

Probabilistic approach to epistemic uncertainty

Steps of the procedure

a) Establish a set of alternative possible models of the structure

b) Based on experience and available evidence, assign a *weight* to each of the models: $\sum w_i = 1$

- Weights represent the *degree of belief* the analyst has on each of the models

c) For each model perform a seismic risk analysis (probability of exceeding the considered LS): $P_{LS,i}$

d) Calculate the *final unconditional risk* as the weighted sum of the above conditional risk:

$$P_{LS} = \sum w_i ¢ P_{LS,i}$$

Deterministic approach to epistemic uncertainty

Steps of the procedure

a) and b) same as for probabilistic procedure

c) For each model calculate the value of the state variable Y_i according to the rules of EC8 Part 3

d) Calculate the *best estimate* of *Y* as the weigthed sum of individual Y_i and check:
$\sum Y_i \cdot 1$

Comment

The above procedure is just one order of rigour higher than customary sensitivity analysis, where the subjective judgement enters in the final selection of just one model and acceptance of the results it gives, while in the above procedure subjectivity enters in the assignment of the weights

Methods of analysis: linear

q-factor approach

- The method is applicable to reinforced concrete (q=1.5) and steel structures (q=2) without restrictions
- Higher values of q are admitted if they can be analytically justified (a rare situation in practice)
- With such small values of q the method is generally quite conservative (it may indicate the need for unnecessary interventions), hence it should find application for buildings having a visible overcapacity relative to local seismic hazard)
- *No mention is made of this method for masonry structures*

Methods of analysis: linear

Linear analysis with unreduced elastic seismic action (1/2)

- **Lateral force and modal response spectrum**
- **Usable subject to a substantial uniformity, over all ductile primary elements, of the ratio between elastically calculated demand and corresponding capacity, i.e.**

$$\max(D_i/C_i)/\min(D_i/C_i) \cdot 2.5 \text{ (suggested, but no >3)}$$

- **Limited practical experience indicates that when the above condition is satisfied the results from elastic multi-modal analysis compare well with those from non linear**
- **This proves that the above condition represents a true physical quantitative definition of *regularity* of a structure from a seismic point of view, a definition that should supersede the semi-quantitative and rather arbitrary definitions given in EC8 Part 1**

Methods of analysis: linear

Linear analysis with unreduced elastic seismic action (2/2)

- **The lateral force method is less accurate and not computationally advantageous: it might well be dropped**
- **Modal response spectrum is accurate when the conditions for applicability are satisfied but the percentage of buildings complying with them is anticipated being not very large**
- **Application of linear methods to <u>masonry structures</u> is problematic due:**
  - The condition related to D/C ratios is not of clear application, especially in case of a FE modelling of the structure
  - There are additional strict conditions to be fulfilled: vertical continuity of all walls, rigid floors, maximum stiffness ratio between walls at each floor less than 2.5, etc

<u>The above remarks point towards a generalised recourse to non linear methods</u>

Methods of analysis: non-linear static 1/3

- **The reference version of the pushover method in EC8 Part 3 is the same as in Part 1**
- **This versione provides satisfactory results when:**
  - The structure is essentially symmetric and torsionally rigid
  - The effects of the higher modes are negligible
- **The case of unsymmetrical (but still single-mode dominated) buildings is treated in EC8 Part 1 by means of an hybrid procedure whereby:**
  - The loading pattern is still planar (*uniform* or *modal*)
  - The displacements of the stiff/strong sides of the building obtained from the pushover analysis are amplified by a factor based on the results of spatial modal analysis
- **In EC8 Part 3 a note is added in 4.4.5 saying that when $T_1 \geq 4T_C$ or T_1>2s the effects of higher modes should be taken into account (not a 'P', hence not obligatory)**

Methods of analysis: non-linear static 2/3

Multi-modal pushover: a convenient proposal (Chopra and Goel, 2002)

- **Use several (spatial) lateral load patterns, corresponding to all *significant* modes:** $F_i = MÁ_i$
- **Perform a pushover analysis and evaluate the desired response quantities R, for each modal pattern and for each of the two horizontal components of the seismic action E_X and E_Y and for the two signs ($R_{E_X} \neq -R_{E_{-X}}$)**
- **Combine the results from the above analyses according to the SRSS rule**

$$R = R_G + \sqrt{\Sigma_\iota} \, (R_{i,E_X} - R_G)^2 + (R_{i,E_Y} - R_G)^2$$

Methods of analysis: non-linear static 3/3

- **Problem with modal combination of member forces (absolute value)**
  - Unrealistically high normal forces and bending moments
  - Shear forces not in equilibrium with bending moments
- **Shear verification of columns: influence of the value of N both in the demand V(N) and in the capacity $V_R(N)$**
  - Approximate solution: evaluate the D/C ratio mode by mode $V_i(N_i)/V_R(N_i)$ (same sign of N_i on both D and C) and then check:

    $\sqrt{[\sum_{\iota} (V_i(N_i)/V_R(N_i))^2]} \cdot 1$

    (damage variable analogy)

Members verifications: demand quantities

- **Ductile members (beam-columns & walls in flexure):**
  the demand quantity is the chord-rotation at the ends, as obtained from the analysis, either linear or non-linear

- **Brittle mechanisms (shear):**
  the demand quantity is the force acting on the the mechanism
  - Linear analysis: the ductile transmitting mechanisms can be:
    - below yielding: the force is given by the analysis
    - yielded: the force is obtained from equilibrium conditions, with the capacity of the ductile elements evaluated using mean values of the mech. prop.'s multiplied by the CF
  - Non-linear analysis: forces as obtained from the analysis

Members verifications: capacities

- **Ductile members (beam-columns & walls in flexure)**
  - expressions of the ultimate chord-rotations are given for the three performance levels, the values of the mech. properties are the mean values divided by the CF.

- **Brittle mechanisms (shear)**
  - expressions for the ultimate strength are given for the NC-LS, the values to be used for the mechanical properties are the mean values, divided by both the usual partial °-factors and the CF

Member verifications: synopsis

<table>
<tr><td colspan="2" rowspan="2"></td><td colspan="2">Linear Model (LM)</td><td colspan="2">Non-linear Model</td></tr>
<tr><td>Demand</td><td>Capacity</td><td>Demand</td><td>Capacity</td></tr>
<tr><td rowspan="7">Type of elment or mechanism (e/m)</td><td rowspan="4">Ductile</td><td colspan="2">Acceptability of Linear Model
(for checking of $\rho_i = D_i/C_i$ values)</td><td rowspan="7">From analysis.
Use mean values of properties in model.</td><td rowspan="4">In terms of deformation.
Use mean values of properties divided by CF.</td></tr>
<tr><td>From analysis.
Use mean values of properties in model.</td><td>In terms of strength.
Use mean values of properties</td></tr>
<tr><td colspan="2">Verifications (if LM accepted)</td></tr>
<tr><td>From analysis.</td><td>In terms of deformation.
Use mean values of properties divided by CF.</td></tr>
<tr><td rowspan="3">Brittle</td><td colspan="2">Verifications (if LM accepted)</td><td rowspan="3">In terms of strength.
Use mean values of properties divided by CF and by partial factor.</td></tr>
<tr><td>If $\rho_i \leq 1$: from analysis.</td><td rowspan="2">In terms of strength.
Use mean values of properties divided by CF and by partial factor.</td></tr>
<tr><td>If $\rho_i > 1$: from equilibrium with strength of ductile e/m.
Use mean values of properties multiplied by CF.</td></tr>
</table>

Capacity models for RC members: flexure

- Mechanically-based models capable of accounting for all internal deterioration mechanisms that develop in inadequately detailed RC members are not available
- Resort has been made to a large database collecting tests made in the past in order to derive empirical expressions.

$$\theta_{um} = 0.01(0.3)^{\nu}\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225}\left(\frac{L_s}{h}\right)^{0.35} 25^{\alpha\rho_{sx}\frac{f_{yw}}{f_c}} 1.25^{100\rho_d}$$

where ν = normalised axial force

ω, ω' = mech. reinforc. ratio of compression and tension reinf.

L_s = shear span

h = net height of the section

α = confinement effectiveness factor

ρ_{sx}, ρ_{dx} = transverse and diagonal reinforcement ratio

Capacity models for RC members: shear

- **The well-known three-terms additive format for the shear strength has been retained. The expressions for the three contributions have been derived using the same database as for the flexural capacity, augmented by test results of specimen failing in shear after initial flexural yielding:**

$$V_R = 0.85\left[\frac{h-x}{2L}\min\left(N;0.55A_c f_c\right)+\left(1-0.55\min\left(5;\mu_\Delta^{pl}\right)\right)\times\right.$$

$$\left.\times 0.16\max\left(0.5;100\rho_{tot}\right)\left(1-0.16\min\left(5;\frac{L_s}{h}\right)\right)\sqrt{f_c}+V_w\right]$$

where x = neutral axis depth

N = compressive axial force (= 0 if tensile)

A_c = cross-section area

μ_Δ^{pl} = plastic part of ductility demand

ρ_{tot} = total longitudinal reinf. ratio

V_w = contribution of transverse steel

$$\frac{V_{\mathrm{R,predicted}}}{V_{\mathrm{R,experimental}}} = 1$$

$$CoV = 15\%$$

Members verifications under bidirectional loading

- **Standard situation due to the simultaneous application of the orthogonal components of the seismic action**

- **No guidance in EC8 Part 3 (lack of adequate knowledge of the behaviour at ultimate)**

- **Limited experimental evidence (Fardis, 2006) supports the assumption of an *elliptical interaction domain* for biaxial deformation at ultimate**

- **Proposal:**
  - For each mode evaluate the *bidirectional demand/capacity ratio* (BDCR)

$$BDCR_i = \sqrt{(\mu_{2i}/\mu_{2u,i})^2 + (\mu_{3i}/\mu_{3u,i})^2}$$

  - Check that $\sum_{\iota} (BDCR_i)^{2} \cdot 1$

Capacity models for strengthened members

The section covers traditional strengthening techniques, such as concrete and steel jacketing, as well as the use of FRP plating and wrapping, for which results from recent research are incorporated.

Guidance in the use of externally bonded FRP is given fo the purposes of:

- **increasing shear strength (contribution additive to existing strength)**
- **increasing ductility of critical regions (amount of confinement pressure to be applied, as function of the ratio between target and available curvature ductility)**
- **preventing lap-splice failure (amount of confinement pressure to be applied, as function of the bar diameters and of the action already provided by existing closed stirrups)**