# A risk-consistent approach to determine behavior factors for innovative steel lateral load resisting systems

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## ABSTRACT

A risk-consistent approach is proposed for the evaluation of behaviour factors that are compatible with Eurocode 8 using nonlinear static and dynamic analysis. The proposed process comprises seven discrete steps, involving hazard assessment and record selection at multiple sites, designing and modelling multiple archetype buildings and assessing their performance vis-à-vis target safety objectives. In all cases, uncertainty is incorporated and propagated to the final results whereby a flexible verification procedure is offered to account for the confidence of the investigator on the data available. The value added goes beyond the current state of art, offering a consistent risk basis for the seismic design of different systems that is compatible with current uniform hazard design spectra and future risk-targeted hazard maps.

**Keywords:** eurocodes, seismic design, behaviour factor, risk

## 1 INTRODUCTION

The application of linear design procedures for seismic loading is based on the approximation of the nonlinear dynamic response of the structure via a linear structural model. To account for the beneficial effects of ductility, which allows trading off damage for lower design forces, EN 1998-1 [1] adopts the behaviour factor q to directly reduce (i.e. divide) the elastic design response spectrum. The same factor is also used to scale up the resulting deformations to approximate their actual value due to nonlinearity. Still, EN 1998-1 [1] only provides values of the q-factor for a very limited number of systems without any guidance on quantifying it for others. In order to introduce new and innovative lateral load resisting systems into the code, researchers have at times proposed corresponding q-values, yet without much consensus: Each proposal comes with its own definition of a safety target and seismic performance assessment method, the latter often reflecting the limited resources available to the researchers. Overall, this uneven process lends little confidence to the proposed q-factors, vis-à-vis the target of achieving a uniform risk level across different systems and sites in Europe. Unlike in the US, where the well-received FEMA P-695 [2] standard has largely settled this debate, Europe has not formulated a standard methodology (barring some recommendations) to define and validate the q-factors. As a direct remedy, the recent EU-funded INNOSEIS project is offering a novel procedure for obtaining consistent values for q based on the definition of a set of structures to represent each class of buildings, the use of nonlinear static and dynamic analysis methods and the incorporation of the effect of aleatory and epistemic uncertainty on the actual systems' performance to reach a uniform level of safety across the entire building population.

## 2 PROPOSED METHODOLOGY

The proposed q-factor estimation methodology is based on the explicit performance assessment of a number of archetype structures using two performance targets defined on a mean annual frequency of exceedance basis. It comprises seven discrete steps, taking the engineer from the site hazard to the final risk-based determination of compliance with the safety standards.

### 2.1 Step 1: Site Hazard

Two different sets of 3–5 sites shall be considered across Europe (e.g., *Fig.1*). The first set will comprise moderate seismicity sites with a peak ground acceleration (or zone factor per EN1998) of approximately $a_g = 0.15g$, mainly geared towards evaluating behaviour factors for Ductility Class Medium (DCM) designs. The second set shall use high-seismicity sites with $a_g = 0.30g$ that can be used to test Ductility Class High (DCH) buildings. In all cases, at least soil type C should be considered, while additional soil types may be of interest. Site selection may be performed according to the EU-SHARE seismicity model [3]. For each set of sites, a single suite of ordinary (non-pulsive, not long duration) records will be selected considering all sites within the set and employing Conditional Spectrum selection [4-5] based on *AvgSa* [6-8], i.e., the geometric mean of 5% damped spectral acceleration ordinates $T_{Ri}$ characterizing the archetype buildings of interest:

$$AvgSa\left(T_{Ri}\right)=\left(\prod_{i=1}^{n}S_a\left(T_{Ri}\right)\right)^{1/n} \tag{1}$$

Periods $T_{Ri}$ can be selected as linearly spaced within a range of $[T_L,T_H]$, where $T_L$ is a low period near the minimum second mode of the buildings to be investigated and $T_H$ is a high period that is near 1.5 times their maximum first mode period. If considerable difference exists among the different first mode periods, one should consider using two different definitions of *AvgSa*, one for low/mid-rise structures (shorter periods) and another for high-rise ones (longer periods), for better fidelity. In any case, ground motion records need to be selected for each definition of *AvgSa* at a given set of sites [9-10], while mean hazard curves are required for each definition of *AvgSa* and at each separate site.

*Fig. 1.* Potential European sites of moderate-to-high seismicity.

Further pan-European verification of q-factors for final inclusion in the code may require additional record sets to be employed that incorporate near-source pulses or long-duration subduction zone motions. Still, assembling such motions is a process that may be strongly site-dependent and will complicate the assessment process needlessly at this level. Still, it should remain as an important consideration for future improvements.

## 2.2 Step 2: Archetype Buildings

A minimum of three archetype configurations shall be selected. These should preferably comprise vertically regular, square/rectangular-plan, residential/office buildings without torsional issues (e.g., *Fig. 2*): At least one low-rise (2-story), one mid-rise (4-story) and one high-rise (8-story) should be employed, the latter only for systems that are applicable to taller buildings. Generally, the use of more buildings highly improves the fidelity of the approach, essentially needing at least 12-20 buildings to have reasonable confidence in the q-factor estimates obtained. Still, three buildings can still serve as a good sample for evaluating pre-norm values of the behaviour factor.

Each building shall be designed according to EN1993/EN1998 [2] and according to the design guide for the proposed structural system, preferably using the recommended values (rather than any specific country's) for nationally determined parameters. As an initial q-factor for design, one may use either existing estimates from previous research or a trial value of 3 – 6 based on engineering judgment. Two versions of each building shall be created, one for $a_g$ = 0.30g for DCH requirements and another for $a_g$ = 0.15g for DCM, unless the system under consideration is only meant to be used only for one of these two site and ductility combinations.

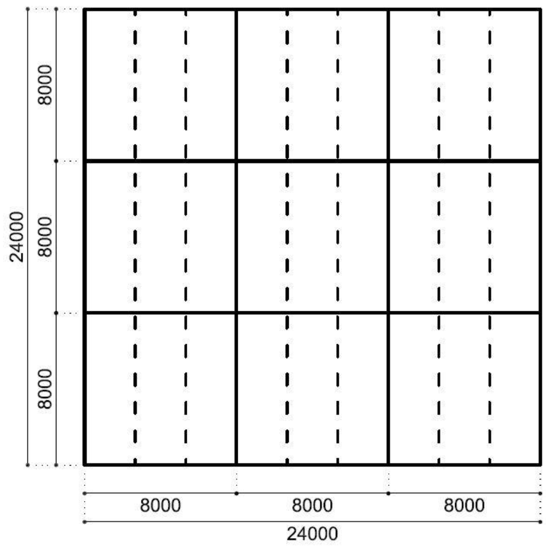


*Fig. 2.* Example plan view of archetype structures (dimensions in mm)

## 2.3 Step 3: Nonlinear Models

A 2D nonlinear model of the structural system of each archetype building shall be created. The model should incorporate accurate hysteresis, including both in-cycle and cyclic degradation, of all system components that may enter the nonlinear range. Optimally, component modelling should be able to accurately reproduce both the monotonic (with in-cycle degradation) and the hysteretic (with cyclic degradation) performance of these elements. Each nonlinear element should also be able to display a clearly defined fracturing deformation (drift, rotation, strain or displacement) whereby it loses all strength and stiffness and ceases to function. *Fig. 3* presents the minimum backbone information each nonlinear element should display. The mass and stiffness of secondary structural and non-structural elements should be incorporated according to state-of-practice approaches, e.g. via a leaning P-Δ column or a full adjacent gravity frame.

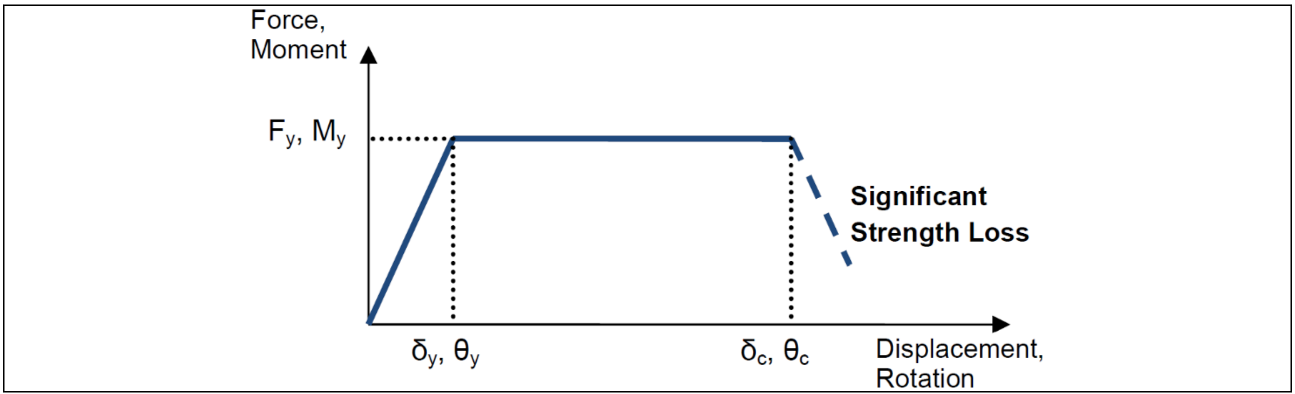


*Fig. 3.* Force/Moment versus displacement/rotation minimum backbone modelling requirements

## 2.4 Step 4: Static Analysis

Nonlinear static pushover analysis shall be performed for each archetype. A preliminary q-factor will be established from the analysis, using the classic product of overstrength Ω and ductility behaviour factor $q_d$. For compatibility with EN1998 [1], overstrength shall be defined as $a_u/a_1$, i.e., the ratio of the maximum base shear strength over the base shear at first yield. The latter is the base shear corresponding to the first plastification of any single (dissipative) element in the structure. Thus:

$$q_{stat} = q_d \cdot \Omega = \frac{\delta_{0.2}}{\delta_y} \cdot \frac{a_u}{a_1} \tag{2}$$

Note that this definition conservatively neglects the overstrength provided by oversizing of members, which is taken into account by US codes and incorporated into the FEMA P-695 [2] standard. If the estimated $q_{stat}$ factor is found to be more than 20% different from the one originally assumed for design for any of the archetypes (Step 2), then a redesign may be required. Still, behaviour factor values obtained via the static approach should only be considered as indicative since they are often found to be less accurate than those estimated via the dynamic approach introduced in the following. The one value that will be of certain use from this step is the overstrength Ω, as it can be employed in the code to offer some flexibility in the definition of the q-factor, as presently done by EN1998.

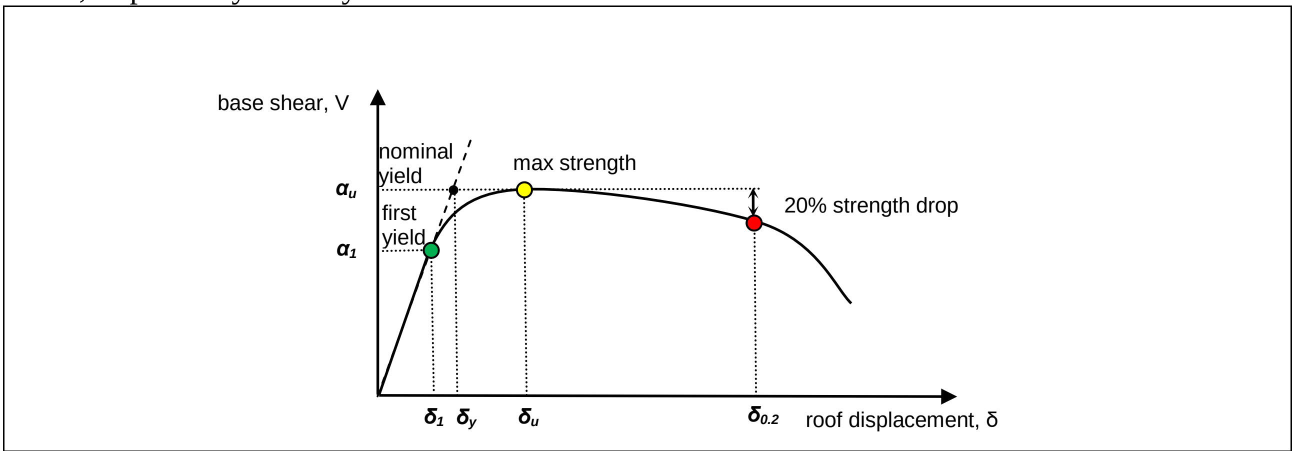


*Fig. 4.* Example of q-factor determination on a static pushover capacity curve

## 2.5 Step 5: Dynamic Analysis

Each archetype will be subjected to the set of records selected for the peak ground acceleration value used for its design. Incremental Dynamic Analysis [11] will be employed, covering the entire post-yield range of response all the way to the first appearance of global collapse in a building, either as global dynamic instability due to simulated modes of failure, or in the form of non-simulated modes of failure introduced in post-processing. For each archetype, the results will be evaluated using *AvgSa* as the *IM*, i.e., the geometric mean of five to ten $S_a$ ordinates linearly spaced within the period range of interest (*Fig. 5*). For simplicity, one may employ the more general definition of the range of periods used in Step 1 that cover several (if not all) building archetypes. Still, when there is considerable variation among first-mode periods across the class, it becomes more efficient to employ $S_a$ ordinates linearly spaced within the range of $[T_2, 1.5T_1]$, where $T_1$ and $T_2$ are the first and second mode of each system investigated. This definition may provide improved results, i.e., lower dispersions and thus better predictive ability, but it also requires a separate estimation of the hazard curve (e.g., *Fig. 6*) and perhaps even a separate record selection for each building, therefore it may severely complicate the process.

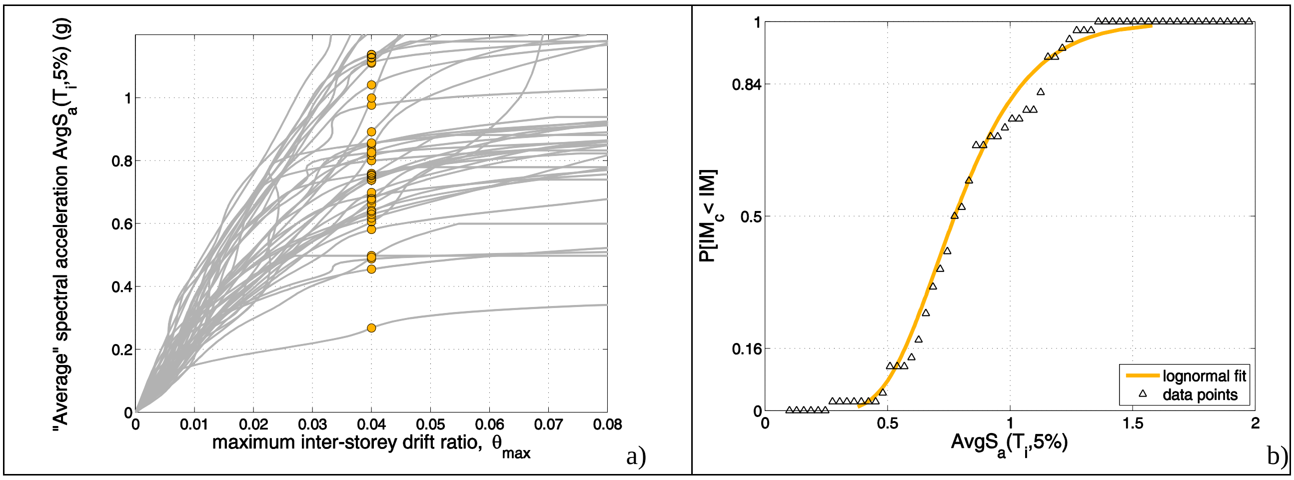


*Fig. 5.* (a) 44 IDA curves for a single archetype building and a "vertical stripe" of *AvgSa* "capacity values" at an inter-story drift level of 4%. (b) Fragility curve corresponding to the vertical stripe

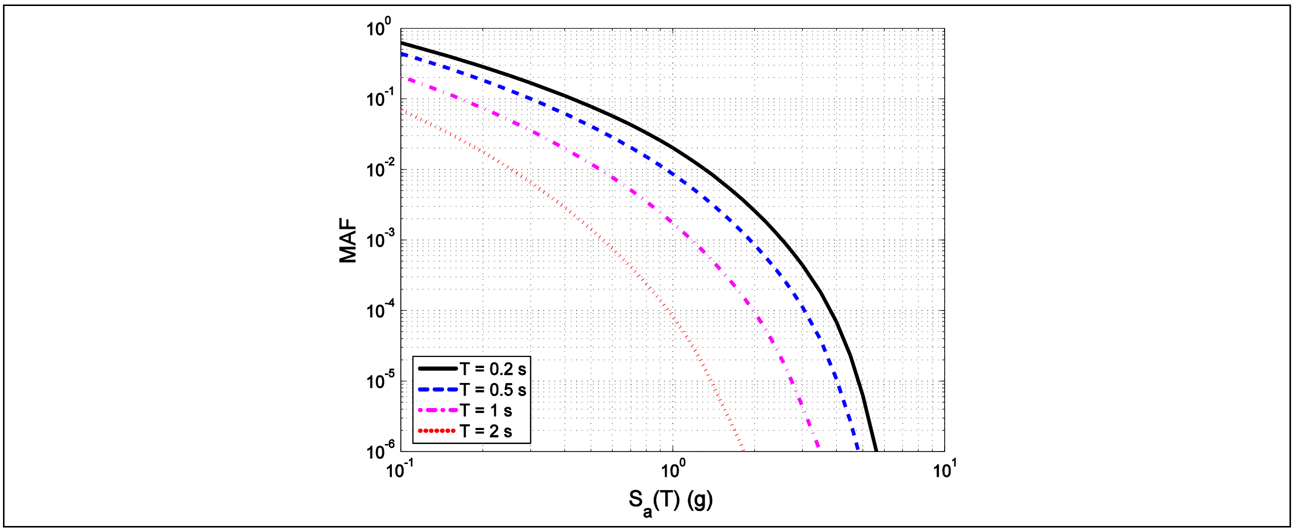


*Fig. 6.* Site hazard curves for $S_a(T)$ and periods of $T$=0.2/0.5/1.0/2.0sec. For use with $AvgS_a$, appropriate hazard curves will need to be generated specifically for this new *IM* for each of the sites of Step 1.

## 2.6 Step 6: Performance assessment

Each archetype's performance will be verified against two performance objectives, namely Life Safety (LS) and Global Collapse (GC). LS will be checked against a mean annual frequency of 10% in 50 years, while GC for the 1 or 2% in 50 years value (to be decided for maximum compatibility with existing EN1998 designs). In general, two types of checks are used in performance assessment. Strength checks are employed to verify that no potential structural element enters a brittle mode of failure (e.g., exceedance of shear or axial strength). These will be deemed to be satisfied automatically thanks to capacity design. For ductile modes of failure, deformation checks shall be applied to verify that no sacrificial (or "energy dissipating") structural element exceeds its plastic

deformation capacity, also known as “capping” deformation, i.e. the deformation that signals the start of the negative stiffness region in monotonic tests.
For our purposes, For LS checking, the approach developed by Vulcu et al. [12] and described below was adopted for deriving acceptance criteria. It is based on provisions of EN 1998-1 [1], ASCE41-13 [13] and FEMA P-795 [14]. The LS seismic performance of the sacrificial components is assessed by identifying component deformation corresponding to two performance levels, namely significant damage (SD), and near collapse (NC), assumed to be characterized by the following description (based on FEMA 356 [15]):

– *Significant Damage:* Significant damage, with some margin against total collapse of the component
– *Near Collapse:* Heavy damage, with low residual strength and stiffness of the component.

Backbone curves are first constructed, for example based on the provisions from FEMA P-795 for cyclic moment-rotation or force-deformation data. In a second step, the rotations/deformations corresponding to the two performance levels were identified. The rotation related to the Significant Damage performance level is considered as corresponding to the drop of force to 0.80 of the maximum one, but not more than 0.75 times the deformation at Near Collapse. The deformation associated with the Near Collapse performance level, is considered as corresponding to a drop of force to 0.2 of the maximum one, but not more than the maximum deformation attained during the test. It is deemed that Life Safety is violated when the first sacrificial element reaches its SD limit-state.

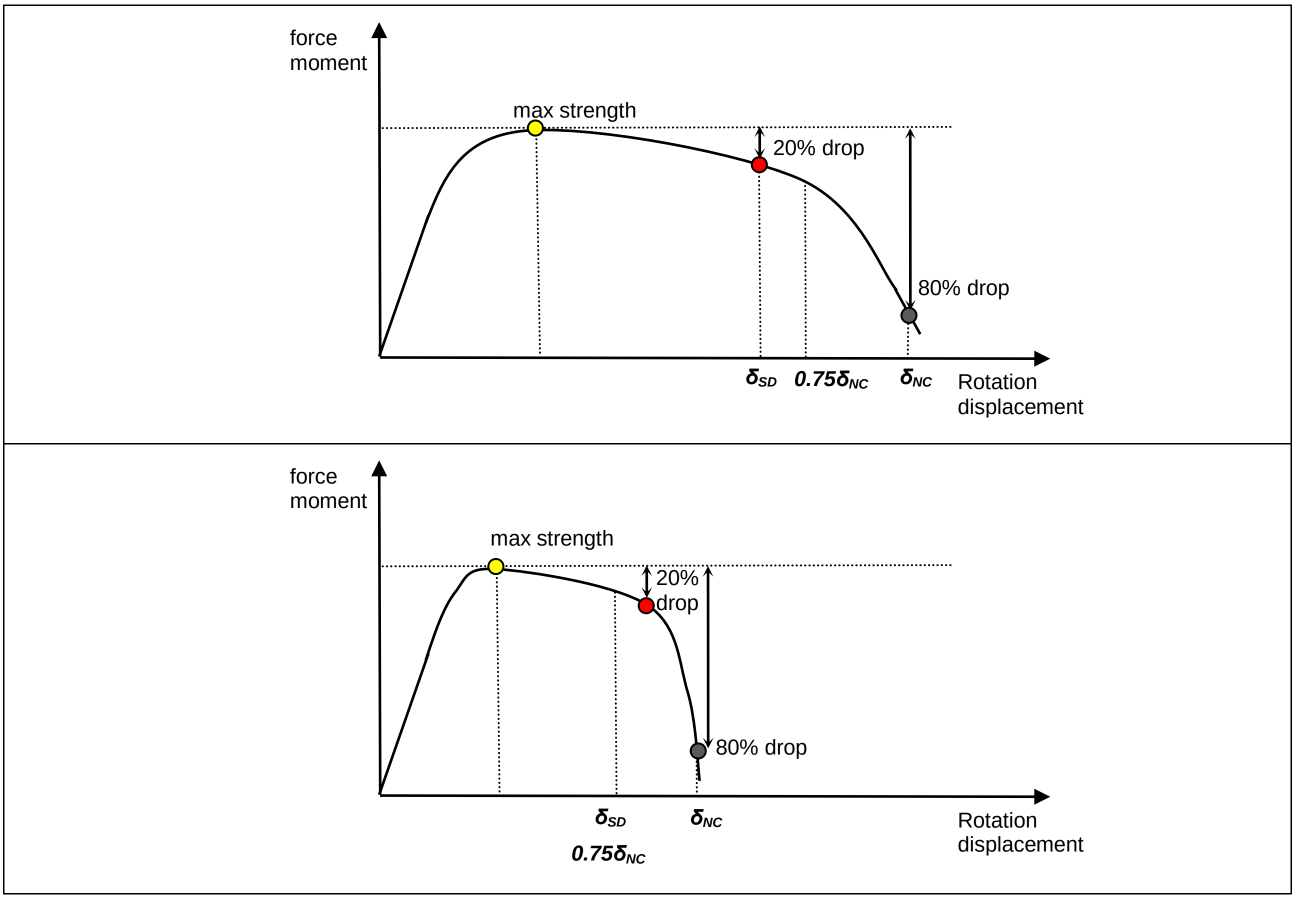


*Fig. 7.* Two examples of SD definition on a component capacity curve. For a component with low in-cycle degradation (top), the SD limit-state is defined by the 20% loss of strength. For a component that rapidly loses strength, (bottom), SD is defined by 75% of the NC state deformation.

GC checking is considered as a numerically more challenging task and it requires a very robust model that is capable of following the behaviour of the building all the way to global collapse.

Optimally, this will be performed by checking only for simulated modes of failure, in our case ductile modes of failure that are explicitly incorporated in the model. In case capacity design is not guaranteed to prevent the appearance of brittle modes of failure after some sacrificial elements have reached their ultimate fracture ductility (and ceased to offer strength or stiffness to the building), non-simulated modes of failure may also be introduced in postprocessing of the results. In both cases, a single global collapse point will be established in each individual IDA curve, using the flatline for simulated modes (*Fig. 5a*) and the earliest occurring non-simulated mode, whichever comes first, to assess the collapse fragility. In cases where the model is not capable of displaying global collapse, a more conservative check may be performed for ductile modes of failure, whereby global collapse shall be assumed to occur when the first ductile element reaches its ultimate (fracturing) deformation.

### 2.7 Step 7: Acceptance or rejection of q-factor

Assessment will be performed according to the Cornell et al. [16] fragility-hazard convolution approach (*Fig. 8*) to determine $\lambda_{DS}$, i.e., the mean annual frequency (MAF) of exceeding the damage state (*DS*, being either LS or GC) of interest:

$$\lambda_{DS} = \int_{IM} \mathrm{P}[D > C|IM] |\mathrm{d}\lambda(IM)| \tag{3}$$

The intensity measure (*IM*) is *AvgSa* and $\lambda(IM)$ is the MAF of exceeding values of the IM, i.e., the hazard curve derived in Step 1. The engineering demand parameter (EDP) is a response parameter that can be used to determine exceedance of either LS or GC. Both demand (*D*) from the nonlinear dynamic analysis and capacity (*C*) are expressed in terms of the EDP. For GC this is always the maximum interstory drift, considering all stories, while for LS it is usually the response parameter that best expresses the exceedance of SD (See Step 6) by the first sacrificial element in the building. This can also be a maximum interstory drift variable if reliable means are found to relate its values with the failure of the element in question.

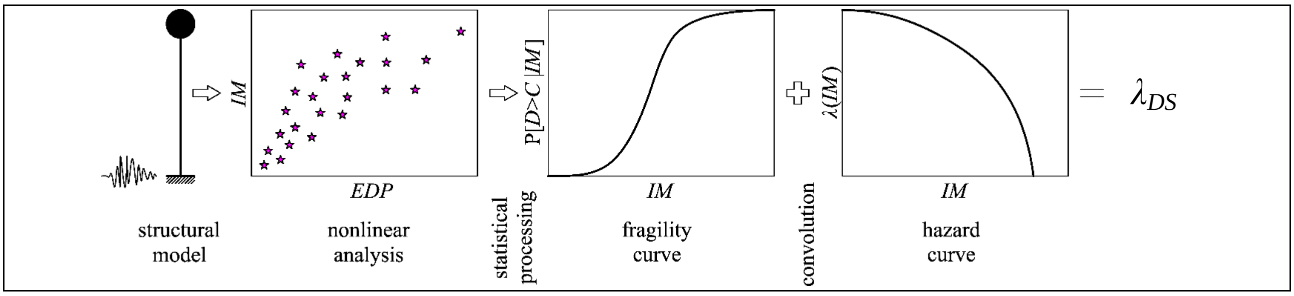


*Fig. 8.* The concept of performance assessment according for a given damage state (DS), by extracting the fragility curve from nonlinear dynamic analyses and convolving with the hazard curve over all values of the IM.

There are considerable uncertainties that need to go into the estimation of $\lambda(DS)$ via *Eq. (3)*. Comprehensively taking them into account is no simple task. Even if we assume that the use of the mean hazard adequately takes into account the uncertainty inherent in the hazard assessment itself, there are considerable demand and capacity uncertainties derived from modeling, analysis, sample, element tests and even the archetype sample size employed. Rather than further complicating the estimation of $\lambda_{DS}$ by including them therein, we chose to employ the Cornell et al. [16] demand-capacity factored design formulation that allows the introduction of uncertainties in a simple checking format:

$$\frac{\lambda_{DS}}{\lambda_{DSlim}} > \exp(K_x \cdot k \cdot \beta_u) \tag{4}$$

If the above inequality is verified for both limit-states, all buildings and at all sites, then the q-factor value by which the buidings were designed is deemed acceptable. Note that $\lambda_{DSlim}$ is the maximum allowable MAF limit whose exceedance signals violation of the damage state. $K_x$ is the standard normal variate associated with a confidence level of $x$%, $K_x=\Phi^{-1}(x)$, e.g. $K_x\approx 1$ for $x$ = 84%. $k$ is the local slope of the hazard curve in log-log space and $\beta_u$ is the total dispersion due to uncertainty, assuming lognormality holds:

$$\beta_u = \sqrt{\beta_{TD}^2 + \beta_{DR}^2 + \beta_{AS}^2 + \beta_C^2} \tag{5}$$

where the dispersions combined are $\beta_{TD}$ due to test data quality rating, $\beta_{DR}$ due to design rules quality rating, $\beta_{AS}$ for archetype sample size and $\beta_C$ due to element capacity test dispersion. $\beta_{TD}$ and $\beta_{DR}$ are based on expert opinion. Where no further guidance is available, one may use pertinent values from FEMA P-695 [2]. Therein, values of 0.50, 0.35, 0.20 and 0.10 are suggested for Poor, Fair, Good and Superior ratings. For $\beta_C$ one should employ the natural dispersion observed in tests of the sacrificial element type, if LS is tested, while for GC one should use a value that conveys the uncertainty in the assessment of collapse given the maximum interstory drift. Some guidance on selecting dispersion values may also be found in FEMA P-58 [17], if good data is not available. An accurate estimation of $\beta_{AS}$ is beyond the scope of this study as it heavily depends on the characteristics of the system evaluated. In general, the more archetypes one uses and the better they cover the expected building population to incorporate the system investigated, the lower $\beta_{AS}$ should become. As a general rule of thumb, and to avoid using small samples of archetype structures, we recommend using the following ad hoc formula:

$$\beta_{AS} = \frac{0.7}{\sqrt{N}} \tag{6}$$

where $N$ is the number of distinct structural configurations employed. Obviously, this is purposefully designed to make sure that using 3-5 archetypes will make $\beta_u$ quite large, and correspondingly require a large margin of safety in terms of *Eq. (4)*. In essence this will only allow employing safe q-values that do not come close to exhausting the ductility of the system. To validate higher q-values that come closer to fully utilizing the actual ductility capacity, one will require at least 20 structural configurations to force the safety margin in *Eq. (4)* to come closer to unity.

In cases that the verification has a wide margin of success, one may choose to redesign the archetypes with a higher q and restart from step 3. If the verification has failed, the q-factor will need to be reduced and another cycle of verification attempted. As a final note, it is important to stress that the q-factors vetted by this approach already incorporate the overstrength Ω. Similar to what has been done for, e.g., moment-resisting frames in EN1998, one may opt to separate the effect of overstrength and allow tuning it on a case-by-case basis via a static pushover analysis, or permanently incorporate it. In the former case, the q-factor should be divided by the value of Ω estimated in Step 4 and appropriate guidance should be provided vis-à-vis limitations and required system characteristics so that the user can safely re-introduce it when needed.

## 3 FURTHER CONSIDERATIONS

One may introduce any number of improvements to the basic procedure outlined above, each bringing in its own additional complexity. Perhaps the most obvious improvement concerns the site and ground motion record selection, where one can require sites having both Type 1 and Type 2 spectra, as well as different soil categories, pulsive and non-pulsive records, or short and long duration ones. Obviously this comes at the cost of considerably more computations plus the burden of appropriately selecting records and estimating the hazard related to such ground motions, a capability that is not yet widely available. In most cases it is also quite important to be able to introduce more archetypes of different configurations and dimensions, including perhaps some level of asymmetry and irregularity (within code limits for using the full q-value), always making sure to provide as much coverage of the future population as possible. Another important consideration is the level of safety associated with the Life Safety and Global Collapse targets. Herein, values of 10% and 1-2% in 50 years have been adopted, yet one should make sure to calibrate these to provide at least as much safety as Eurocode 8 offers to traditional lateral load resisting systems.

This is indeed a wide ranging investigation that will offer a solid basis in determining the required performance of any newer system to be added into the code.

## ACKNOWLEDGMENT

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