Reliability Based Code Calibration for Earthquake-Resistant Design

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Abstract
Probabilistic methods are employed for the evaluation of several Eurocode provisions for the design of earthquake-resistant reinforced concrete buildings. The provisions that are examined specifically relate to the strength and ductility of individual members, as well as to the overall structural performance. Whereas further calibration of Eurocode 8 provisions is recommended with respect to member ductility, results concerning the overall seismic risk to which the designed buildings are exposed seem encouragingly low. Structural reliability or seismic risk estimates are enabled using derived vulnerability curves for three frames that vary due to different placement of masonry infill panels. Furthermore, approximation of the actual behaviour ($q$) factors corresponding to failed frames, leads to observations with regard to the calibration of the design value that has a significant influence on the design seismic forces.

Keywords: Eurocodes, infilled RC frames, member capacity, limit state criteria, vulnerability curves

1 Introduction
The probabilistic assessment of structures designed for earthquake resistance is a particularly meaningful means for calibrating relevant design codes as at least some of the many uncertainties involved in the different stages of design and analysis can be accounted for. Unlike standard design whereby the uncertainties in the expected loads are taken into account through safety factors, when designing for earthquake resistance the uncertainty in the nature of the seismic loads additionally contributes to the uncertainty in the magnitude of these loads. Furthermore, it is not yet clear which parameter most appropriately quantifies earthquake severity, even though the peak ground acceleration (PGA) is widely adopted for design, followed by use of the spectral acceleration for the representation of the seismic loads through static, lateral forces.

The current paper focuses on the calibration of Eurocode provisions for the design of reinforced concrete (R/C) buildings. This involves design firstly according to Eurocode 2 (EC2, CEN 1991), which specifies the general provisions for R/C structures, and secondly according to Eurocode 8 (EC8, CEN 1995), which specifies additional provisions for earthquake resistance. Under EC8 design, the following two requirements need to be primarily satisfied:
(i) No collapse requirement: the structure must be able to withstand the design seismic action without partial or total collapse so that structural integrity and a residual load-bearing capacity are retained following the earthquake.
(ii) Damage limitation requirement: the structure must be able to withstand a seismic action that has a shorter return period (i.e. higher probability of occurrence) than the design seismic action, so that the damage sustained does not impose any limitations on structural usage. Both of these aim at ensuring an economical design that should, even in extreme loading cases, offer adequate safety. Account of these requirements is taken through the definition of three limit states, as follows:

(i) Serviceability limit state (SLS): the structure must be able to resist earthquakes of low intensities without sustaining structural damage. This may be achieved by ensuring that all structural members remain elastic during frequent earthquakes.

(ii) Ultimate limit state (ULS): structural elements (and important nonstructural elements such as infill panels) must only sustain light and repairable damage during earthquakes of moderate intensity (i.e. the design earthquake).

(iii) Collapse limit state: the structure must be able to resist earthquakes of high intensity without collapsing.

Whereas for all three limit states the target is to safeguard against fatalities, the first two limit states also aim at minimising disruption and economic losses due to earthquakes of low to moderate intensities.

Herein, calibration of Eurocode provisions for the earthquake-resistant design of R/C buildings is performed through consideration of the three structural systems shown in Figure 1. Firstly, a bare ten-storey frame is considered, which has been properly designed and detailed to EC2 and EC8 for medium ductility class and a design PGA $A_d=0.25g$. The other two buildings involve the same frame infilled with clay brick masonry walls. Although these are nonstructural elements that do not require proper design, they are expected to interfere with the structural response due to their tendency to increase the overall stiffness. A fully infilled frame is therefore considered, as well as a frame with a soft ground storey (the so-called “pilotis” frame system). The latter is expected to be more vulnerable than the bare and fully infilled frames due to the sudden change in stiffness causing a sensitivity in the structural members of the ground storey.

2 Modelling of uncertainties

An important first source or uncertainties that needs to be considered in studies such as the one addressed herein, is material variability. Uncertain material properties can...
significantly affect the strength of structural members as well as their ductility, and they therefore also affect the overall structural response. At the same time, however, further uncertainties arise due to the use of approximate analytical methods and empirical models. Moreover, as previously mentioned, the important uncertainty in the magnitude and nature of the seismic loads cannot be neglected.

Table 1 lists the random variables adopted herein, alongside their corresponding statistical distributions. As far as the concrete properties are concerned, the peak compressive strength ($f_c$) is taken as a normal variable. Of course, for seismic design whereby ductile behaviour is targeted, concrete confined by transverse reinforcing hoops has a higher compressive strength, which is related to the unconfined strength. Of great importance is also the ductility of the confined concrete described through its ultimate strain $\varepsilon_{cu}$. Due to using empirical models for the prediction the stress-strain behaviour of confined concrete, the uncertainty in $\varepsilon_{cu}$ is accounted for through the model uncertainty factor $X_{m,\varepsilon_{cu}}$ whose statistics have been calculated using experimental results (Kappos et al., 1998). The statistics of model uncertainty factors for the modelling of the masonry infill panels have also been derived using a similar approach. $X_{m,\tau_{\text{max}}}$ and $X_{m,A}$ correspond respectively to the maximum shear stress and area under the shear stress-shear strain ($\tau$-$\gamma$) curve that is capable of allowing for stiffness degradation (Dymiotis et al., 2001). For the steel properties, the yield and ultimate strengths ($f_y$ and $f_u$) are assumed to be fully correlated with a constant strain hardening ratio of 1.15, whereas negative correlation is assumed between the yield strength and ultimate steel strain ($\varepsilon_{su}$). The last variable listed in Table 1 is the critical interstorey drift where, as will be discussed below, exceedance of this value during a time-history analysis implies structural failure under the ultimate limit state (ULS).

### 3 Calibration of code provisions for R/C member design

Prior to considering the overall safety margins associated with the earthquake-resistant design of R/C buildings to the Eurocodes, it is appropriate to evaluate relevant code provisions concerning the ductility and flexural and shear strength of beams and columns. As already mentioned, all members considered herein have been properly designed to Eurocodes 2 and 8, hence attention in this section is primarily given to these specifications. The beam and column cross-sections whose strength and ductility capacities are investigated are illustrated in Figure 2 and represent (i) the largest and most heavily confined members and (ii) the smallest and least confined members in the 10-storey frames of Figure 1.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean</th>
<th>COV (%)</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>$f_c$</td>
<td>28MPa</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>$X_{m,\varepsilon_{cu}}$</td>
<td>1.0</td>
<td>39</td>
</tr>
<tr>
<td>Steel</td>
<td>$f_y$</td>
<td>440MPa</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>$f_u$</td>
<td>506MPa</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>$\varepsilon_{su}$</td>
<td>0.09</td>
<td>9</td>
</tr>
<tr>
<td>Masonry</td>
<td>$X_{m,\tau_{\text{max}}}$</td>
<td>0.875</td>
<td>26</td>
</tr>
<tr>
<td></td>
<td>$X_{m,A}$</td>
<td>0.95</td>
<td>19</td>
</tr>
<tr>
<td>ULS Drift capacity</td>
<td>$\delta_{cr}$</td>
<td>6.6%</td>
<td>31</td>
</tr>
</tbody>
</table>
3.1 Methodology

The variability in strength and ductility parameters due to material uncertainty is firstly assessed using the Response Surface Method (RSM), as described by Kappos et al. (1998). The overall methodology involves firstly the computation of the output response parameters for different combinations of the input random material properties at 5% (lower characteristic), 50% (mean) or 95% (upper characteristic) fractile values. Empirical equations giving the response parameters as functions of the material properties are therefore derived by means of second order regression. The fibre model approach, which is widely recognised to reliably predict response, is adopted for the section analyses.

The derived response surface expressions are of the form

\[ Z = m_Z \left( \beta_0 + \beta_{f_c} \bar{f}_c + \beta_{f_y} \bar{f}_y + \beta_{\varepsilon_{su}} \bar{\varepsilon}_{su} + \beta_{\varepsilon_{cu}}^2 + \beta_{\varepsilon_{su}}^2 + \beta_{f_c} \bar{f}_c + \beta_{f_y} \bar{f}_y + \beta_{\varepsilon_{cu}} \bar{\varepsilon}_{cu} + \beta_{\varepsilon_{su}} \bar{\varepsilon}_{su} \right) \]  

where \( Z \) is the output response parameter, \( m_Z \) is the value for this parameter evaluated for the member under consideration with mean material properties, \( \beta_i \) are regression coefficients and standardised values of the random variables \( f_c, f_y \) and \( \varepsilon_{su} \) are used. For ductility parameters, the right hand side of equation (1) is further multiplied by the model uncertainty factor \( X_{m,ecu} \). It is pointed out that the derived expressions proved to be highly successful, giving almost exact prediction of the response parameters for a wide range of cross-sections that were not considered during the derivation process. The actual statistical distributions of the response parameters can be obtained through Monte Carlo simulation of the random variables and substitution of these in the derived response surface equations. The evaluation of the intrinsic conservatism of code provisions that follows is therefore achieved by means of comparisons of code-implied response values with the derived distributions.

3.2 Member flexural strength

Fibre model analyses are carried out for the four sections of Figure 2, assuming the code-specified material models. The concrete and steel models proposed in EC2 are adopted without modification by EC8. For concrete, in addition to the partial safety factor of 1.5, it is required that the design concrete strength be further multiplied by a
factor $\alpha=0.85$ accounting for unfavourable load effects, thus leading to a strength of 11.3MPa that is almost half the characteristic strength ($f_{ck}=20$MPa). The ultimate strain is specified as 0.35%, which coincides with the value assumed in the initial analyses for spalling of the cover concrete. For steel, both $f_y$ and $f_u$ are divided by the partial safety factor of 1.15, giving design values of 348 and 400MPa, respectively. Both the yield and ultimate moments ($M_y$ and $M_u$) are considered for assessing the conservatism in the likely attained strength of beams, whereas for columns four points on the moment-axial load ($M-N$) interaction diagrams are considered for both yield and ultimate conditions. Yielding is included as an indication of the serviceability of the members, and also because of its importance in inelastic analysis. For both beam sections and both positive and negative bending, the code predicted values are sufficiently conservative when compared to the statistical distributions. The characteristic-to-code ratios for $M_y$ are around 1.1, whereas for $M_u$ these are around 1.3 for all cases. The main contributor to this is the partial safety factor of 1.15 applied to the steel strength, which has been found to govern the variability in $M_u$, with a somewhat smaller effect due to the reduction in the concrete strength.

Figure 3 compares the column interaction diagrams resulting from fibre model analyses for mean and characteristic material properties, with ones obtained using the EC2 material models. Although all EC2 curves are found to lie within the fibre model curves, the safety margins do not appear to be uniform throughout the axial load range. To clarify this, an analysis of the safety margins for the various strength parameters at ultimate conditions is presented in Table 2. The ratios of the simulated characteristic to EC2 values are highly influenced by the code-specified partial safety factors. For the case of $N_{ult}$, the ratio of 1.3 with respect to the lower characteristic arises again from the factor of 1.15 applied to $f_y$ in combination with the influence of strain hardening. For parameters mainly dependent on $f_c$, such as $N_{ucr}$, the ratios are influenced by the partial safety factors on material strength and the coefficient $\alpha=0.85$, as well as the confinement strength factor $f_{cc}/f_c$.

### 3.3 Member shear strength

With regard to shear strength, it is assumed that the overall member shear capacity is given by

$$V_{R3} = X_{m,v_{R3}}(V_{c1} + V_{c2} + V_w)$$

(2)
where $V_{c1}$ is the concrete contribution to shear resistance, $V_{c2}$ is the contribution due to the presence of axial load, $V_{w}$ is the shear resistance contributed by the transverse reinforcement and $X_{m,VR3}$ is an uncertainty factor. Whereas $V_{c1}$ and $V_{w}$ are predicted using empirical and theoretical expressions, respectively, a response surface expression capable of considering the imposed axial load is used for predicting $V_{c2}$ (Kappos et al., 1998). Values given by equation (2) are compared with those given by the code-specified expression $V_{w,d} = V_{w}$. $V_{w}$ is evaluated in the same way as for equation (2) but assuming characteristic (rather than randomly generated) yield strength of the transverse reinforcement. $V_{cd}$, however is taken to be the design shear resistance estimated from

$$V_{cd} = (t_{Rd} k (1.2 + 40 \rho_i) + 0.15 \sigma_{cp}) b w d$$

where $t_{Rd}$ is the basic design shear strength, $k$ is an empirical factor taking into account the curtailment of the bottom reinforcement and $\sigma_{cp}$ is axial load ratio. The comparisons are presented in Table 3 for the two columns. As for the cases of strength and ductility, the material strengths in equation (2) are randomly generated. Moreover, the axial load ratio is taken to be the design load divided by a random ultimate load whose statistical distribution was estimated as outlined in section 3.1. It is found that, in all examined cases, the code estimates are conservatively lower than those given by equation (2).

### 3.4 Member ductility

Curvature ductility is given by

$$\mu = \frac{\varphi_u}{\varphi_y} = \frac{\epsilon_{cu}}{\epsilon_{sy}} \left( \frac{d - c_y}{c_u} \right)$$

where $\varphi_y$ and $\varphi_u$ are ultimate and yield curvatures, respectively, $\epsilon_{sy}$ is the steel yield strain, $d$ is the effective section depth and $c_u$ and $c_y$ are the neutral axis depths corresponding to $\epsilon_{cu}$ and $\epsilon_{sy}$, respectively. An approximation for this equation is suggested in background documents to EC8 (Tassios, 1989a) and is of the form

### Table 2. Analysis of safety margins for column interaction diagrams

<table>
<thead>
<tr>
<th></th>
<th>C300×300</th>
<th>C500×500</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{uc}$</td>
<td>1.56</td>
<td>4.11</td>
</tr>
<tr>
<td>$N_{ut}$</td>
<td>0.57</td>
<td>1.33</td>
</tr>
<tr>
<td>$N_{ub}$</td>
<td>0.47</td>
<td>1.44</td>
</tr>
<tr>
<td>$M_{ud}$</td>
<td>0.06</td>
<td>0.27</td>
</tr>
<tr>
<td>$M_{ub}$</td>
<td>0.08</td>
<td>0.36</td>
</tr>
</tbody>
</table>

### Table 3. Comparison of code and simulated values of shear resistance

<table>
<thead>
<tr>
<th>Column</th>
<th>Fractile</th>
<th>$V_{R3}$ (EC) (MN)</th>
<th>$V_{R3}$ (sim) (MN)</th>
<th>$V_{R3}$ (sim)/ $V_{R3}$ (EC)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C300×300</td>
<td>50% (mean)</td>
<td>0.176</td>
<td>0.346</td>
<td>1.97</td>
</tr>
<tr>
<td></td>
<td>5% (char.)</td>
<td>0.155</td>
<td>0.269</td>
<td>1.74</td>
</tr>
<tr>
<td>C500×500</td>
<td>50% (mean)</td>
<td>0.799</td>
<td>1.454</td>
<td>1.82</td>
</tr>
<tr>
<td></td>
<td>5% (char.)</td>
<td>0.731</td>
<td>1.132</td>
<td>1.55</td>
</tr>
</tbody>
</table>
\[
\mu_\varphi = \frac{\varepsilon_{cu}}{\varepsilon_{sy}} \left( \frac{0.60 - 0.5\nu}{1.35\nu} \right) 
\]  
(6)

with the axial load ratio \( \nu \) defined as

\[
\nu = \frac{N}{f_c b h} \geq 0.1 
\]  
(7)

where \( b \) and \( h \) are the section width and depth, respectively.

The value of \( \varepsilon_{cu} \) in equation (6) corresponds to \( f_{cu} = 0.85 f_c \) along the descending branch of the \( \sigma-\varepsilon \) curve for confined concrete proposed for EC8 (Tassios, 1989b). This model has been found to be an appropriately conservative one (Kappos et al., 1998). In order to examine equation (6) in combination with the proposed \( \sigma_c-\varepsilon_c \) law, curvature ductilities are calculated for both columns and then related to the corresponding simulated distributions as illustrated in Figure 4. The presented distributions account for both model and material uncertainties. It is clear that the degree of conservatism in equation (6) decreases with decreasing axial load. Furthermore, mean material properties give, in some cases, significantly more conservative results than their characteristic counterparts. This is explained by recognising that the mechanical ratio of confining steel,

\[
\omega_w = \rho_w \frac{f_y}{f_c} = \frac{\text{Volume of hoops}}{\text{Volume of concrete}} \times \frac{f_y}{f_c} 
\]  
(8)

that is the main parameter in confinement models, increases by 27% if mean material properties are replaced by characteristic values. Moreover, converting from characteristic to design values would give a further increase of 30%, thus the partial safety factors have an unfavourable effect on the prediction of \( \varepsilon_{cu} \). Another factor that

![Figure 4. Statistical distributions (including model uncertainty) of column ductility](image-url)
may be responsible for the apparently unconservative values for the heavily confined section is the fact that in the fibre model analyses the failure criterion was buckling of the longitudinal reinforcement, a possibility which is not allowed for in equations (5) and (6). Including the distributions that correspond to 0.85$f_c$ in all plots of Figure 4, it is shown that this effect does not influence the aforementioned conclusions.

It is also interesting to note that, whereas for $\nu=0.4$ equation (6) gives more conservative predictions than (5), as the level of axial load decreases the approximate values approach the upper tail of the distributions and become less conservative than those given by equation (5). It hence appears that the approximation suggested in EC8 is reliable only for higher axial load ratios. For C500×500 under no axial loading, fracture of the tensile reinforcement takes place prior to concrete failure and the presented points therefore correspond to steel failure. Analyses for this column assuming a strain hardening ratio $f_u/f_y=1.30$ (close to the maximum permissible value of 1.35) still indicated the validity of the preceding remarks.

Column sections designed and detailed to EC8 for DC'M' are aimed to have a curvature ductility of 9. In order to examine the extent to which this requirement is satisfied, the available curvature ductilities for these two sections corresponding to their design axial loads are evaluated. The design axial load ratio is given by equation (7) assuming a design value of $f_c$.

Table 4 presents the levels of axial load and characteristic values from the corresponding distributions of curvature ductility. The latter are calculated by linearly interpolating between the statistics of the simulated distributions for the discrete values of $\nu$ considered (0.0, 0.1, 0.2 and 0.4). As shown, the 5% characteristic values of ductility are significantly less than 9 in both cases. The last column of this table shows the probability of not achieving the code specified target ductility. Whereas for C300×300 a reasonable safety margin exists, for the heavily confined column the mean value of the distribution is only slightly higher than the required ductility and thus the probability of exceedance approaches 0.5. However, defining ductilities on the basis of $\varepsilon_{cu}$ corresponding to 0.85$f_c$ along the descending branch is often a conservative approach. Distributions of curvature ductility corresponding to the 0.5$f_{cc}$ criterion are also produced and as shown in Figure 4, these give more conservative code values, especially for higher axial load ratios.

### 4 Structural reliability

The three frames of Figure 1 are initially assessed by means of vulnerability curves. These do not give overall failure probabilities but probabilities dependent on the occurrence of an earthquake of given intensity and are therefore cumulative distribution functions (cdf’s). The methodology adopted for deriving the vulnerability curves is that formulated by Dymiotis et al. (1999) and summarised in Figure 5. Whereas emphasis is given to the uncertainties concerning the actual structural systems and modelling, the uncertainty in the imposed seismic forces is treated as simply as possible. This is achieved by selecting an as small as possible set of natural accelerograms that cover a wide range of seismic parameters such as

<table>
<thead>
<tr>
<th>Column</th>
<th>$\nu$</th>
<th>$N/N_{ult}$</th>
<th>$(\mu_{\phi})_m$</th>
<th>$(\mu_{\phi})_{0.05}$</th>
<th>$P(\mu_{\phi}\leq9)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C300×300</td>
<td>0.0713</td>
<td>0.0465</td>
<td>15.12</td>
<td>5.71</td>
<td>0.14</td>
</tr>
<tr>
<td>C500×500</td>
<td>0.351</td>
<td>0.215</td>
<td>9.21</td>
<td>3.67</td>
<td>0.48</td>
</tr>
</tbody>
</table>
magnitudes and PGAs of the non-scaled record, soil characteristics, durations and epicentral distances. For the frames considered herein, three such carefully selected accelerograms have been found to be adequate for the ULS, whereas for the SLS use of five accelerograms is preferred.

The computer program employed for the dynamic, inelastic, time-history analyses is DRAIN-2000, (Kappos & Dymiotis, 2000), a modified and enhanced version of DRAIN-2D (Kanaan & Powell, 1973). Specifically, use of the earlier described response surface functions is incorporated into this program, in order to enable prediction of member strength and ductility during the analysis. Furthermore, beam failures are identified during the time-history analysis and accounted for through strength and stiffness adjustments. Collapse of the system is defined through exceedance of overall drift criteria and/or column failures. This program is employed within a Monte Carlo simulation based on Latin Hypercube Sampling (LHS) for variance reduction.

4.1 SLS vulnerability

The serviceability limit state (SLS) is rather difficult to define quantitatively, as different response parameters may govern depending on the target performance of both structural and nonstructural members. Furthermore, design codes do not indicate any possible procedures for quantification of the SLS. A series of “failure” criteria are therefore considered herein, which should ideally lead to comparable SLS vulnerability. Firstly, a limiting interstorey drift ($\delta_{cr}$) of 0.5% is considered, based on the average value of those suggested in EC8 and is also in accordance with fairly recent US recommendations (Wen et al., 1996). Another SLS consideration is the degree of structural damage, hence the criterion $\theta > \alpha\theta_y$ is adopted, where $\alpha \geq 1$ and in the absence of detailed information, vulnerability curves are plotted for $\alpha = 1$ and $\alpha = 2$.

Resulting vulnerability curves are presented in Figure 6(a)-(c) for each of the above criteria and for all three frames. The curves obtained for $\delta_{cr} = 0.5\%$ imply significantly lower failure probabilities than those obtained for $\theta > \alpha\theta_y$. In fact, the curves corresponding to $\theta > \alpha\theta_y$ show that the exact value of $\alpha$ in the range $1 \leq \alpha \leq 2$ has a relatively small effect on the results. Focusing on the serviceability earthquake which for EC8 has an intensity $A = 0.10g$, the local damage criterion clearly implies a high likelihood of violation of the SLS ($P_f = 0.76$ and 0.38 for $\alpha = 1.0$ and 2.0, respectively).
As anticipated, the irregular pilotis frame appears more vulnerable than the fully infilled and bare frames. In addition to the damage sustained by structural members, masonry panels in both infilled frames also suffer some damage as the post-cracking region may be entered prior to the violation of the SLS. As such a violation is normally associated with necessary repair to nonstructural elements only, the encounter of $\gamma > \gamma_{cr}$ is acceptable, whereas $\gamma > \gamma_{max}$ should be avoided. Nevertheless, sometimes $\gamma > \gamma_{max}$ in critical regions, especially for fully infilled frames as these generally fail at higher intensities.

The above findings suggest a need to re-consider SLS drift values. In particular, the 0.4% drift value recommended in EC8 for buildings with nonstructural elements of brittle materials attached to the structure is thought to be a more reasonable option, with levels of damage expected to be close to those for the $\theta > 2\theta_y$ criterion.

### 4.2 ULS vulnerability

The ULS generally corresponds to the “no collapse requirement” and is defined herein using a dual criterion, namely column failure or failure due to a large interstorey drift. The first criterion is based on the damage index

$$I_{d,el} = \max \left\{ \frac{\theta_p\text{ req}}{\theta_p\text{ avail}}, \frac{V_{R2}\text{ req}}{V_{R2}\text{ avail}}, \frac{V_{R3}\text{ req}}{V_{R3}\text{ avail}} \right\} \geq 1.0$$

(9)

where $\theta_p$ is the plastic rotation and $V_{R2}$ and $V_{R3}$ are the shear resistances to web crushing and diagonal compression failure, respectively. The subscripts “req” refer to parameter values required during the time-history analyses whereas the subscripts “avail” refer to the section capacities calculated as recommended by Kappos (1991). The second criterion is the critical interstorey drift, which as mentioned earlier is treated as a random parameter (see Table 1) rather than adopting the traditionally used deterministic value of 3%.
Figure 6(d) shows the ULS vulnerability curves for all three frames. Whereas for the SLS the fully infilled frame is generally associated with the lowest failure probabilities, this appears significantly less reliable than the bare frame for the ULS, indicating an unfavourable effect of the infills, as also suggested in EC8. For the bare frame with average material property values the fundamental period $T = 0.93$ s, which is significantly higher than the fully infilled and pilotis frame values of 0.60 s and 0.65 s, respectively. As may be seen from Figure 7, much higher spectral accelerations are therefore associated with the two infilled structures, hence the bare frame is subjected to lower seismic forces. The lower failure probabilities obtained for the bare frames are thus reasonable, as has been justified by considering an additional motion that gave a lower spectral ordinate for the bare frame (Dymiotis et al., 2001). With regard to the adequacy of reliability levels, these are addressed below where the seismic hazard is also taken into account for independent reliability estimates.

4.3 Seismic risk

This section deals with the quantification of the actual seismic risk to which a structure, situated at a certain site, is exposed. As already mentioned, vulnerability curves do not allow for the uncertainty concerning earthquake occurrence (seismic hazard). Since earthquakes are random processes, such an uncertainty always exists and a structure located in a seismically active region will be exposed to some earthquake loads during its lifetime. Neither the magnitude, nor the exact nature of these, however, can be predicted a priori. An appropriate description of the seismic hazard needs to be incorporated, enabling a conceptually meaningful estimation of structural reliability. In this context reliability refers to the probability that the structure will resist any seismic loads during its lifetime, and results therefore need to be obtained assuming a fixed period of exposure ($t_d$). As remarked by Hamburger (1997) the counterpart of the vulnerability curve is the hazard curve, and integration of these two curves is a meaningful description of structural performance. As hazard curves are site-specific, in the same way as vulnerability curves are structure-specific, the former need to be selected in order to adequately represent an area of seismicity comparable to that for which the structure has been designed.

Crespellani et al. (1997) considered the long-term seismicity of a site in Central Italy, leading to the derivation of hazard curves for different scenarios, including one that gave a “mean plus one standard deviation” PGA, associated with a probability of
exceedance of 10% in 50 years, of $A=0.27g$. This compares well with the design scenario adopted for the frames considered herein, whereby $A_d=0.25g$ also corresponds to a 10% probability of exceedance in 50 years. Associated hazard curves shown in Figure 8 are therefore used in the current study. It is pointed out that these correspond to mean PGAs and also that some of the curves have been extrapolated assuming that the time-dependent probability of exceedance follows the Poisson distribution.

As demonstrated by, for example Thoft-Christensen & Baker (1982), the probability of failure ($P_f$) of a structural system of resistance $R$ subjected to a load $S$ may be estimated from

$$P_f = \int_{-\infty}^{\infty} f_S(x) F_R(x) \, dx = \int_{-\infty}^{\infty} f_R(x) (1 - F_S(x)) \, dx$$  \hspace{1cm} (10)$$

where for the current case $R$ and $S$ may be represented by the structural vulnerability and seismic hazard, respectively, integrated with respect to PGA. Equation (10) is evaluated for each scenario of structural vulnerability and seismic hazard by numerical integration. Figure 9 illustrates the trend with which the seismic risk increases with $t_d$, obtained using all hazard curves of Figure 8 and the ULS vulnerability curves. Key failure probabilities and corresponding reliability indices ($\beta$) are further listed in Table 5 for both limit states considered herein. Firstly considering the SLS, values of $P_f$ are obtained using the hazard curve for $T_r=50$ years, the return period normally associated with the serviceability earthquake of $A=0.10g$ in EC8. Three values are obtained for each frame, corresponding to the previously considered criteria. In accordance with the vulnerability results, the probabilities corresponding to member yielding criteria are in most cases significantly more conservative than equivalent values for the displacement criterion, which for the bare frame is more or less an order of magnitude lower than the $\theta>\theta_y$ value. The range of estimated values again points out the need to calibrate local as well as displacement-based criteria for SLS design.

Regarding the ULS, $P_f$ values corresponding to $T_r=475$ years as assumed in EC8 for this limit state, range between 0.82 and 2.78% for the three frames. It is interesting to observe that the lowest of these, which corresponds to the bare frame, is comparable to the SLS value obtained for the same structural system with a much shorter
exposure period and the drift criterion. For the two infilled frames, on the other hand, ULS values are closer to those obtained at the SLS for the $\theta > 2\theta_y$ criterion.

In order to assess the appropriateness of the derived failure probabilities, some indicative target values are also included in Table 5. In the absence of European targets concerning seismic reliability, the target reliability indices specified in Eurocode 1 (EC1, CEN 1994) are listed for return periods of 50 and 475 years for the SLS and ULS, respectively. Probabilities are assumed to follow a Poisson distribution so that $\beta_{EC1}$ for $T_r = 50$ years from the code-specified value for $T_r = 475$ years. It is noted that these values refer to target reliabilities under normal, non-seismic loads, and are therefore higher than their counterparts relating to seismic loads. Furthermore, recent US proposals for seismic target reliability (Wen et al., 1996) are listed, again approximating the SLS value assuming Poisson distributed failure probabilities. For the ULS all calculated values are above these US target reliability indices, although appropriately well below the non-seismic EC1 targets. The same conclusion also applies to the SLS values obtained for member yielding criteria, while values based on drift criterion compare favourably with the EC1 indices.

5 Calibration of behaviour factors

In the earthquake-resistant design of ordinary buildings, the design seismic loads are normally represented by static lateral forces, which increase with height. Following the derivation of the elastic acceleration response spectrum, the design spectrum is obtained through a division of the elastic one by a behaviour factor $q > 1$. This

<table>
<thead>
<tr>
<th>Limit state scenario</th>
<th>SLS: $\theta &gt; \theta_y$</th>
<th>SLS: $\theta &gt; 2\theta_y$</th>
<th>SLS: $\delta_{\text{max}} &gt; 0.5%$</th>
<th>ULS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_r$ (years)</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>475</td>
</tr>
<tr>
<td>Bare frame</td>
<td>$P_f$ 0.1139, $\beta$ 1.21</td>
<td>$P_f$ 0.0716, $\beta$ 1.46</td>
<td>$P_f$ 0.0069, $\beta$ 2.46</td>
<td>2.40</td>
</tr>
<tr>
<td>Fully infilled frame</td>
<td>$P_f$ 0.0936, $\beta$ 1.32</td>
<td>$P_f$ 0.0281, $\beta$ 1.91</td>
<td>$P_f$ 0.0110, $\beta$ 2.29</td>
<td>2.01</td>
</tr>
<tr>
<td>Pilotis frame</td>
<td>$P_f$ 0.1175, $\beta$ 1.19</td>
<td>$P_f$ 0.0362, $\beta$ 1.80</td>
<td>$P_f$ 0.0163, $\beta$ 2.14</td>
<td>1.91</td>
</tr>
<tr>
<td>$\beta_{EC1}$</td>
<td>2.45</td>
<td>2.45</td>
<td>2.45</td>
<td>3.80</td>
</tr>
<tr>
<td>$\beta_{US}$</td>
<td>-</td>
<td>-</td>
<td>1.5-1.8</td>
<td>1.55-1.75</td>
</tr>
</tbody>
</table>
reduction takes into account the ability of a structure to dissipate energy whilst behaving inelastically, as aimed by the capacity design approach. The design lateral forces are then determined based on the design spectral ordinate corresponding to the approximated fundamental period of the building.

Due to the importance of $q$-factors in design and the non-theoretical nature of their quantification, calibration of adopted values appears appropriate. As suggested by Kappos (1991), the actual $q$-factor developed in a frame may be estimated from

$$q = \frac{A_{cr}}{A_{d,inel}} = q_d \frac{A_{cr}}{A_{d,el}}$$

(11)

where $q_d$ is the design value, and $A_{cr}$ is the critical acceleration leading to structural failure. Determination of this latter parameter needs to be based on the criteria for the ULS discussed previously. Due to the inclusion of $A_{cr}$, equation (11) may only be evaluated for cases where failure, in terms of violating the ULS, is encountered, thus at any PGA this may be performed for only the percentage of structures that fail.

The two integrals in equation (10) lead to the derivation of two new distributions, conditional upon failure in $t_d$ years. As recommended by Thoft-Christensen & Baker (1982), the two new functions given by

$$f_R'(A') = \frac{f_R(A')(1 - F_S(A'))}{P_f}$$

(12)

$$f_S'(A') = \frac{f_S(A')F_R(A')}{P_f}$$

(13)

describe the maximum loads $R'$ sustained by structures that fail and the loads $S'$ that cause these failures. The two new distributions are described pictorially in Figure 10, where it may be seen that $R'$ and $S'$ are subsets of $R$ and $S$, respectively. Behaviour factors are related to the resistance of failed structures, as $A_{cr}$ is the intensity that causes collapse. The resulting histogram of maximum accelerations sustained by structures which fail ($R'$) derived for the ULS using the 475 years return period hazard curve is therefore used for the calibration of the $q$-factor. Employing the linear transformation of equation (11), which by substituting the design values for the frames considered herein reduces to $q=15A_{cr}$, cdf's of actual $q$-factors for each structural system are obtained. The design $q$-factor of 3.75 is found to be associated with probabilities of non-exceedance as listed in Table 6. The probabilities of non-exceedance for the entire population of structures, obtained from

$$P(q \leq q_d) = P(q \leq q_{d,\text{failure}}) P_f$$

(14)

are also included in this table. Hence, between 0.6 and 2.2% of 10-storey 3-bay R/C frames, depending on the presence of infill walls, designed to EC8 for DC “M”, soil class $A$ and $A_r=0.25g$ are expected to fail whilst developing a $q$-factor less than or

![Figure 10. Resistance and load subsets given by eqns (12) and (13), respectively](image)
equal to the code value due to uncertainties relating to material variability, failure criteria and the seismic excitation. It is pointed out, however, that for the case considered herein the probability of not exceeding the code-specified \( q \)-factor is mostly governed by the seismic risk as, except for very low intensities, the probability of an earthquake occurring is very small compared to the probability of structural failure in such an event.

6 Summary and conclusions

Response surface equations and statistical representation of member response parameters have enabled the evaluation of several Eurocode provisions with regard to the earthquake-resistant design of R/C buildings. It has been found that ductility estimates arising from EC8 provisions assuming characteristic material properties do not coincide with the lower fractiles of the simulated distributions. Further calibration of EC8 provisions would therefore be recommended, unless ductility is assessed based on mean material properties. Nonetheless, for the members considered herein the Eurocodes always provide conservative estimates of flexural and shear strength even though the safety margins are not uniform for the various parameters. This may be partly attributed to the different anticipated failure modes (brittle versus ductile).

Regarding the overall seismic risk, in the absence of target safety margins, outcomes seem encouraging when compared with available data for non-seismic codes or results from previous studies involving other modern seismic codes. Furthermore, approximation of the actual behaviour (\( q \)) factors corresponding to structural systems that fail, led to the derivation of the probability of exceedance of the design value (\( q_d \)). Even though in the rare occasion that a frame fails \( q_d \) is highly likely to be exceeded, the overall probabilities unconditional on such an occurrence are found to be reasonably small. As there is again lack of code-specified target values, simply comparing the derived probabilities with the commonly adopted 5% non-exceedance probability, obtained values for all three frames considered herein correspond to significantly lower fractiles. Although results are encouraging, indicating that there is even a margin for increasing behaviour factors, hence resulting in more economic designs, this conclusion may not be general as it relates to a frame of specific geometry and design assumptions. Calibration of the code-specified \( q \)-factors therefore needs to allow for all aspects affecting the value of \( q \) according to the code. This would require the consideration of buildings designed for other ductility classes and soil categories, as well as other building types and geometries.

Finally, it is pointed out that the methodologies and associated outcomes summarised in this paper are presented in more detail by Kappos et al. (1998), Chryssantopoulos et al. (2000) and Dymiotis et al. (2001).

7 References


