# Effects of Design to EN-Eurocodes on the Seismic Fragility of Concrete Buildings

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**SUMMARY:**
Fragility curves are constructed for prototype regular RC frame and wall-frame buildings designed and detailed to EC2 and EC8, using EC8's own seismic performance assessment methods and criteria. The overall conclusion is that the performance goals of EC8 are met in a consistent and uniform way, except in RC walls, which fail early in shear despite their design against it. The slenderness limits and lateral bracing requirements of EC2 for 2nd-order effects pose severe restrictions on the size of columns and walls, which impact their design and seismic fragilities. Reduction in fragility from higher design peak ground accelerations is disproportionately low.

*Keywords: concrete buildings, concrete frames, concrete walls, Eurocode 8, fragility curves*

## 1. INTRODUCTION

In this paper the question of whether EN-Eurocode 8 (EC8) achieves its own seismic performance goals for RC buildings is addressed by using the analysis and evaluation tools provided by EC8 itself in its part on assessment of existing buildings (CEN, 2005). To this end, a portfolio of prototype plan- and height-wise regular RC frame and wall-frame buildings are designed to Eurocodes 2 (CEN, 2004b) and 8 (CEN, 2004a). Attention is paid to the effect of the slenderness and bracing requirements of Eurocode 2 (EC2) on the size of columns and walls and their implications for seismic performance. Seismic performance is assessed here on the member level, at yielding and ultimate. Instead of carrying out a fully deterministic performance assessment, fragility curves are constructed, depicting the probability that a damage state is exceeded, conditional on a seismic Intensity Measure (IM). The fragility curves in the present paper refer to individual prototype buildings and not to classes thereof. Unlike analytical fragility curves constructed without recourse to Monte Carlo simulation, e.g., (Spence, 2007), the present ones are not based on a global dispersion parameter $\beta$ with a prescribed value, but are built point-by-point from the conditional probability of exceeding the damage state of interest. Besides, they account for shear failure, normally ignored in analytical fragility studies.

## 2. SCOPE AND ASSUMPTIONS

Prototype regular RC frame or wall-frame buildings are studied. The parameters considered are:

- the number of storeys: 5 and 8; for frame buildings, 2 storeys as well;
- the level of seismic design:
  - design for gravity loads only (not even for wind), per EC2 alone;
  - seismic design per EC8 for Ductility Class (DC) L (Low), M (Medium) or H (High), for various levels of design peak ground acceleration (PGA) at the top of EC8's soil type C (firm soil) for the Type 1 spectrum, incorporating the soil factor $S = 1.15$;
- in wall-frame systems: the fraction of the seismic base shear taken by the walls.

The geometry of the prototype buildings is as follows: All storeys have the same height, $h_{st} = 3$ m. The plan is rectangular, with the same geometric parameters and member sizes in both horizontal directions. The bay length is the same throughout the plan, $l_b$ = 5 m. Wall-frame buildings have square columns on a 5 m × 5 m grid and two parallel rectangular walls in each horizontal direction per 5×5 bays of the building plan (for simplicity and generality, no beams are considered to frame into these walls, which just share with the frames the same rigid floor displacements). The size of each column or wall and the width of the beams ($b_w$ = 0.3 m) are constant in all storeys of a building. Slabs are 150 mm thick. Permanent loads, including the full dead weight of the structure, its finishings, its partitions and the façades, amount to7 kN/m$^2$. The nominal value of occupancy loads is 2 kN/m$^2$.

Generic members considered here are limited for simplicity to the interior columns and beams in wall-frame or frame systems and the walls of wall-frame systems. The interior columns and the beams of each frame are all of the same size. Beam depths are kept constant in each frame, but in frame buildings they may differ at different storeys. Under these conditions the effect of perimeter columns and beams on the response of interior members may be ignored, if perimeter members have one-half the rigidity of interior ones at the same storey; then: (a) all beam ends in a storey of a frame have the same elastic seismic moments and inelastic chord rotation demands; (b) the same applies to all interior columns in a storey, with the exterior columns developing one-half the elastic seismic moments of same-storey interior ones but the same inelastic seismic chord rotation demands; (c) the axial force variation due to the seismic action may be neglected in interior columns. Vertical elements are taken as fixed at ground level, with negligible bending moments due to gravity loads. Beam-column joints and floor diaphragms are taken as rigid. P-Δ effects due to the seismic action are taken into account.

## 3. MEMBER SIZING TO MEET THE STIFFNESS REQUIREMENTS OF EC2 AND EC8

Both EC2 and EC8 allow ignoring 2$^{nd}$-order effects if they are less than 10% of the corresponding 1$^{st}$-order ones, giving simplified criteria to that end. The EC2 simplified criteria for isolated columns limit their slenderness ratio as: $l_0/i_c \leq 20ABC\sqrt{n}$, where $l_0$ is the effective length, $i_c$ the radius of gyration of the uncracked column section, $n = N_{Ed}/A_c f_{cd}$ its axial load ratio for the axial force $N_{Ed}$ from the analysis for the factored gravity loads ("persistent and transient" design situation) and $A$, $B$ and $C$ coefficients that depend on creep, steel ratio and the 1$^{st}$-order moments at the two column ends. The effective length of the column is derived from its clear height and the rotational restraints at its ends by the beams. The ground-storey column is fixed at the base, but the column of the storey directly above, with its smaller end restraint, may be more critical, despite its smaller axial load. Therefore, the minimum column size satisfying the slenderness limitation is sought among the two lower storeys.

**Table 3.1.** Depths of interior beams and columns in frame buildings

| design PGA (g) | DC | 2 storeys | | | | 5 storeys | | | | 8 storeys | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $h_b$ (m) | | $h_c$ (m) | | $h_b$ (m) | | $h_c$ (m) | | $h_b$ (m) | | $h_c$ (m) | |
| | | EC8 | EC2/EC8 | EC8 | EC2/EC8 | EC8 | EC2/EC8 | EC8 | EC2/EC8 | EC8 | EC2/EC8 | EC8 | EC2/EC8 |
| 0 (EC2) | - | - | 0.40 | - | 0.45 | - | 0.40 | - | 0.55 | - | 0.40 | - | 0.65 |
| 0.10 | L | 0.35 | 0.40 | 0.35 | 0.45 | 0.35 | 0.40 | 0.40 | 0.55 | 0.40 | | 0.55 | 0.65 |
| 0.15 | L<br>M | 0.35 | 0.40 | 0.35 | 0.45 | <br>0.35 | <br>0.40 | 0.40 | 0.55 | 0.40 | | 0.55 | 0.65 |
| 0.20 | M<br>H | 0.35 | 0.40 | 0.35 | 0.45 | 0.40 | | 0.40 | 0.55 | 0.40 | | 0.55 | 0.65 |
| 0.25 | M<br>H | 0.35 | 0.40 | 0.40 | 0.45 | 0.45 | 0.45 | 0.50 | 0.55 | 0.45 | | 0.60 | 0.65 |
| 0.30 | M<br>H | 0.40 | | 0.40 | 0.45 | 0.45 | | 0.60 | | 0.45 | | 0.70 | |
| 0.35 | H | 0.40 | | 0.45 | | 0.5 | | 0.65 | | 0.50 | | 0.75 | |

Table 3.1 refers to frame systems. The first row lists the minimum member sizes for gravity-only design to meet EC2's simplified slenderness limits. Two options are considered for seismic design: one, appearing under the heading EC2/EC8, observes this size limit; the other, under the heading EC8, ignores it. The depths of interior beams, $h_b$, and columns, $h_c$, are then chosen as the minimum values to

meet all other pertinent requirements:

- the maximum top steel ratio in EC2 at beam supports at the ULS for the “persistent and transient” design situation, including the effects of postulated deviations of vertical members from verticality;
- the upper limit on the column axial load ratio, $n$, per EC2 at the ULS for the “persistent and transient” design situation and per EC8 in the “seismic design situation”;
- (most critical) the 0.5% storey drift limit in EC8 under the damage limitation seismic action.

Witness in Table 3.1 under the heading EC2/EC8 that the minimum member sizes satisfying EC2’s slenderness limit are larger than those needed to meet simply EC8’s 0.5% storey drift limit for the damage limitation seismic action, except in the 5- or 8-storey buildings designed for PGA of 0.30g or 0.35g. With the larger sizes adopted in the latter cases the margin against that storey drift limit is small at one or more storeys and the design per EC8 is optimal. The same applies to practically all designs under the heading EC8, which have smaller member sizes by ignoring EC2’s slenderness limit.

**Table 3.2.** Depths of interior beams and columns in wall-frame buildings and wall length for various fractions of the base shear taken by the walls

| design PGA (g) | DC | 5 storeys | | | | 8 storeys | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | $h_b$ (m) | $h_c$ (m) | $l_w$ (m) | $V_{wall,base}/V_{tot,base}$ (%) | $h_b$ (m) | $h_c$ (m) | $l_w$ (m) | $V_{wall,base}/V_{tot,base}$ (%) |
| 0 (EC2) | - | 0.40 | 0.40 | 2.5 | 65 | 0.40 | 0.45 | 4.0 | 78 |
| | | 0.40 | 0.40 | 3.2 | 75 | 0.40 | 0.45 | 5.0 | 85 |
| 0.10 | L | 0.40 | 0.40 | 1.5 | 37 | 0.40 | 0.45 | 2.0 | 45 |
| | | | | 2.0 | 53 | | | 2.5 | 57 |
| | | | | 2.5 | 65 | | | 3.5 | 73 |
| | | | | | | | | 4.0 | 78 |
| 0.15 | L, M | 0.40 | 0.40 | 1.5 | 37 | 0.40 | 0.45 | 2.0 | 45 |
| | | | | 2.0 | 53 | | | 2.5 | 57 |
| | | | | 2.5 | 65 | | | 3.5 | 73 |
| | | | | | | | | 4.0 | 78 |
| 0.20 | M, H | 0.40 | 0.40 | 1.5 | 37 | 0.45 | 0.45 | 2.0 | 42 |
| | | | | 2.0 | 53 | | | 3.0 | 63 |
| | | | | 2.5 | 65 | | | 4.0 | 76 |
| 0.25 | M, H | 0.45 | 0.45 | 2.0 | 44 | 0.50 | 0.45 | 2.0 | 40 |
| | | | | 2.5 | 57 | | | 3.0 | 61 |
| | | | | 3.5 | 73 | | | 4.0 | 74 |
| 0.30 | M, H | 0.50 | 0.50 | 2.0 | 36 | 0.50 | 0.50 | 2.5 | 47 |
| | | | | 3.0 | 59 | | | 3.5 | 64 |
| | | | | 4.0 | 73 | | | 4.0 | 70 |
| 0.35 | H | 0.55 | 0.50 | 2.5 | 46 | 0.55 | 0.55 | 2.5 | 40 |
| | | | | 3.0 | 56 | | | 3.5 | 58 |
| | | | | 4.0 | 71 | | | 4.5 | 70 |

Table 3.2 refers to 5- or 8-storey systems of frames and walls. The first two rows list the minimum column sizes for design to EC2 for negligible 2$^{nd}$-order effects in braced systems. The first row gives also the length, $l_w$, of each one of the two 0.25m-thick walls per horizontal direction, so that the building is considered braced according to the EC2 criteria for cracked bracing walls. The second row gives the corresponding $l_w$ presuming that the walls stay uncracked. Witness the large wall lengths needed to brace the building in these two cases and the large fractions of the total elastic seismic base shear taken by the two walls. The walls in buildings designed to EC8 are chosen with length not only equal to the minimum $l_w$ in the gravity-only braced systems with uncracked walls but also shorter, so as to cover a wide range of values of the fraction of the total base shear, $V_{tot,base}$, taken by the two walls, $V_{wall,base}$, including:

- wall buildings (defined in EC8 as those with $V_{wall,base} \geq 0.65V_{tot,base}$);
- frame-equivalent dual systems (per EC8 those with $0.35 \leq V_{wall,base}/V_{tot,base} < 0.5$) and
- wall-equivalent dual systems (those in EC8 with $0.5 \leq V_{wall,base}/V_{tot,base} < 0.65$).

The seismic designs in Table 3.2 meet the 0.5% storey drift limit for the damage limitation seismic action primarily thanks to the walls. For a design PGA up to 0.20g for a 5-storey building, or to 0.15g for a 8-storey one, their interior columns and beams have the minimum depth meeting the EC2 slenderness limits for braced systems. At higher design PGAs larger frame members are needed to control the drift of the upper storeys, where the walls are ineffective.

## 4. SEISMIC FRAGILITY OF RC BUILDINGS

### 4.1. Methodology for fragility analysis

Fragility curves are constructed using as Intensity Measure (IM) the PGA of the horizontal motion at the top of the soil. Spectral values of individual motions at the fundamental period of the building are taken to vary about the value of the standard spectrum anchored to the IM-value with the coefficient of variation (CoV) listed in Table 4.1.

**Table 4.1**. Coefficients of variation for the fragility curves

| Demand | CoV | Capacity | CoV |
|---|---|---|---|
| Beam chord rotation demand, for given spectral value at fundamental period | 0.25 | Beam or column chord rotation at yielding | 0.33 |
| Column chord rotation demand, for given spectral value at fundamental period | 0.20 | Beam or column ultimate chord rotation | 0.38 |
| Wall chord rotation demand, for given spectral value at fundamental period | 0.25 | Shear resistance in diagonal tension (inside or outside plastic hinge) | 0.15 |
| Beam shear force demand, for given spectral value at fundamental period | 0.10 | Wall chord rotation at yielding of the base | 0.40 |
| Column shear force demand, for given spectral value at fundamental period | 0.15 | Wall ultimate chord rotation at the base | 0.32 |
| Wall shear force demand, for given spectral value at fundamental period | 0.20 | Wall shear resistance in diagonal compression | 0.175 |
| Spectral value, for given PGA and fundamental period | 0.25 | | |

For the two damage states of yielding and ultimate condition in flexure, the Damage Measure (DM) is the chord rotation at the member end. For the ultimate condition in shear, it is the shear force outside the plastic hinge or inside it (considered then alongside the rotation ductility factor). Deterministic estimates of these DMs are obtained for each value of the excitation PGA via a static analysis following Part 3 of EC8 (CEN, 2005). Shear forces in beams and columns are calculated from the plastic mechanism and the yield moments of the sections that have already yielded. Once a plastic hinge forms at a wall base, the shears up the wall are amplified for inelastic higher mode effects per (Keintzel, 1990), as required by EC8 in the design of walls in DC H buildings (a simpler and less demanding amplification rule is adopted in EC8 for DC M walls).

The seismic analysis gives the mean values of DM demands. The mean values of the corresponding capacities for the two damage states are determined again according to Part 3 of EC8 (CEN, 2005). Contrary to the usual approach that does not use Monte Carlo simulation, non-parametric fragility curves are established here point-by-point, from the conditional-on-IM probability that the random variable of DM-demand (for given IM) exceeds the random variable of DM-capacity. The mean values of these two random variables are established as explained above. Their variances are estimated from the values of their CoV itemized in Table 4.1. The CoV-values in Table 4.1 for the chord rotation demands for given spectral value at the fundamental period are based on comparisons of inelastic chord rotation demands in height-wise regular buildings to their elastic estimates (Panagiotakos & Fardis, 1999; Kosmopoulos & Fardis, 2007). The values for the shear force demands are based on parametric studies. Those of the capacities reflect the uncertainty in the models used for the estimation of their mean values and the scatter of material and geometric properties (Biskinis & Fardis, 2010a; 2010b; Biskinis et al, 2004).

Fragility results are obtained and presented separately for each member and storey. They account for mechanical interaction of the damage states between different elements in a mean sense. As the analysis is deterministic and based on mean properties, the demand on a member or failure mode is computed assuming that a damage state in another member or mode of force transfer has been reached, only if that state has taken place with a conditional-on-IM probability of at least 50%. The fragility curve of a member at the ultimate damage state is taken as the maximum among its possible ultimate conditions: of the plastic hinge in flexure or shear, and of the part outside the hinge in shear, i.e., as if there is perfect correlation between these failure modes.

## 4.2. Indicative results

Figures 4.1 to 4.4 and 4.5(top) refer to frames. They come in sets of four figures each: the first row is for yielding, the second for ultimate; the first column of each set concerns beams, the second columns. Different curves in each figure are for different storeys. The conclusions on the fragility of frames are:

- Beams are much more likely to reach the ultimate damage state than columns.
- As shown in Figure 4.1, frame columns that do not meet the slenderness limits in EC2 have much higher fragility than those doing so, at both damage states.
- Frames designed per EC8 for DC L (i.e., with a *q*-factor of 1.5 and without any detailing for ductility or capacity design) do not perform well if their design PGA is above the ceiling of 0.10g recommended by EC8 for applying DC L (Figure 4.2(left)): their beams may fail in shear even before they yield – but are unlikely to do so below the design PGA.
- Except for the point above, frames designed to EC8 give very satisfactory fragility results, even well beyond the design PGAs. Their performance is rather insensitive to their geometric and design parameters, such as design PGA and DC, as well as other parameters considered in parametric studies not included in this paper (notably, the bay length, the concrete and steel grades, etc.).
- The first beams to yield in frames designed to EC8 for DC M or H are very likely to do so between the damage-limitation and the design PGA. Their columns are quite likely to stay elastic well beyond that range. By contrast, beams and columns of DC L frames, which are designed to stay elastic until two-thirds of the design PGA but have an overstrength thanks to the difference between mean and design values of material strengths, are very likely to stay elastic well above the design PGA – but may fail abruptly at yielding.
- Design for a higher PGA (see Figures 4.2(right) and 4.3(left)) reduces the fragility, especially against the ultimate damage state, but the benefit is disproportionately low.
- Design to DC M in lieu of H (Figure 4.3) may reduce slightly the fragility of beams against yielding, but may increase that of columns against ultimate; however, such effects are neither systematic nor marked.
- There is no systematic effect of the number of storeys on the fragility of beams, but that of columns seem to decrease in taller buildings (see Figures 4.1(right) and 4.4).
- Frames designed to EC2 only (Figure 4.5(top)) have higher beam fragility than those designed to EC8 for DC L and PGA = 0.15g (Figure 4.2(left)), but their columns have lower fragility.

Figures 4.5(bottom) to 4.7 for wall-frame buildings come in sets of six. The first row is for beams, the second for columns, the third for walls. Different curves in the first two rows are for different storeys. The first column is for yielding; the second for ultimate. The results lead to the following conclusions:

- Walls are the most critical elements at both damage states. They are very likely to reach the ultimate condition – mostly in shear – even below the design PGA!
- The walls of nonductile wall buildings designed to EC2 only for gravity loads are not markedly more fragile at either damage state than in EC8 wall buildings or wall-equivalent dual systems; however, their columns and beams of all storeys are (cf. Figure 4.5(top) to 4.6(top) and 4.7).
- EC8 wall-equivalent dual and wall buildings have similar fragilities, higher than frame-equivalent dual for their walls but lower for their beams and columns (see Figure 4.6 for a frame-equivalent dual and a wall building; wall-equivalent duals are in-between, but closer to wall systems). Beams and columns in dual systems have higher fragility than in pure frames (cf. Figure 4.6 to 4.5(top)).
- Design to higher PGA (as in Figure 4.7) or higher DC (cf. Figure 4.6(top) to 4.7(top)) reduces very

little the fragility (except in DC L designs for PGA = 0.15g, which have much higher fragility than their DC M counterparts).

- Taller buildings exhibit only slightly higher fragilities.

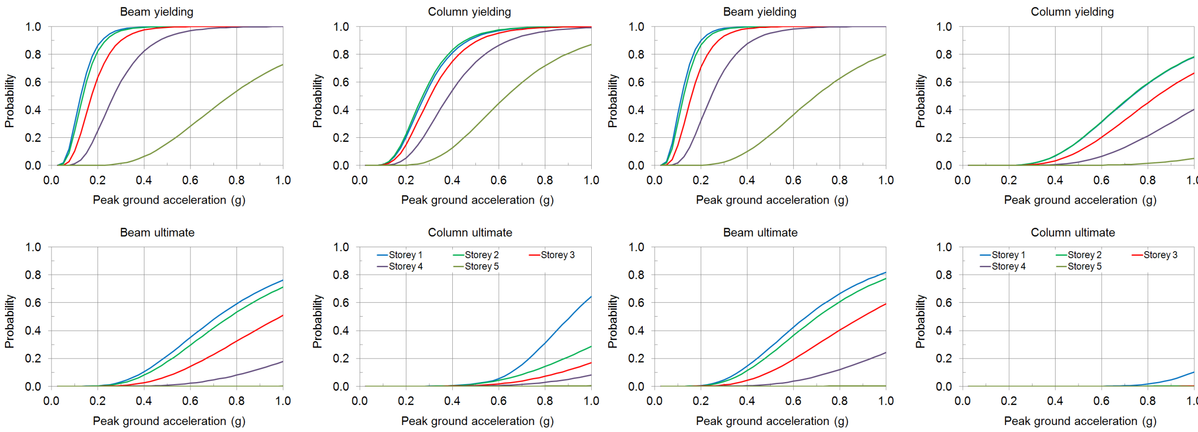


**Figure 4.1.** Fragility curves of 5-storey frame buildings satisfying (left) or violating (right) EC2's slenderness limits and designed to EC8 for PGA = 0.2g with DC H

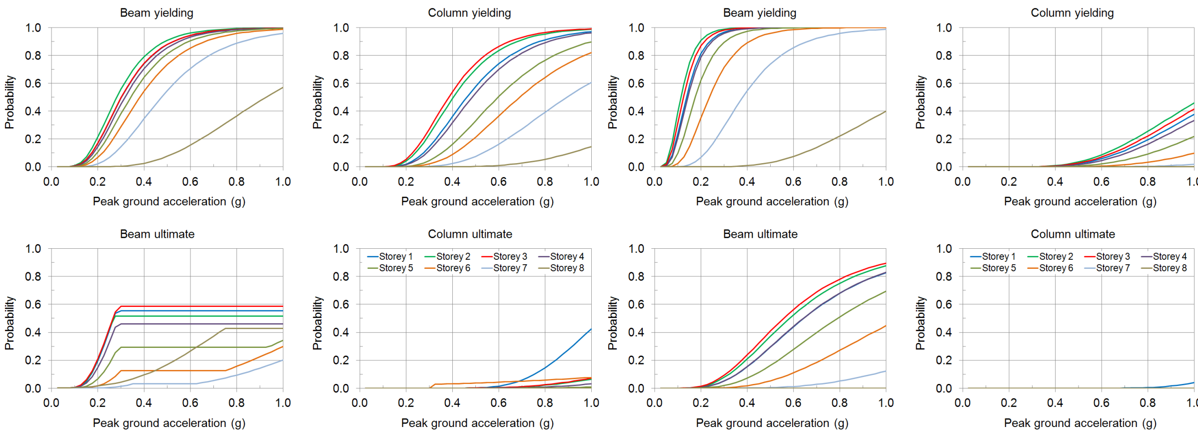


**Figure 4.2.** Fragility curves of 8-storey frame buildings satisfying EC2's slenderness limits and designed to EC8 for PGA = 0.15g with DC L (left), or M (right)

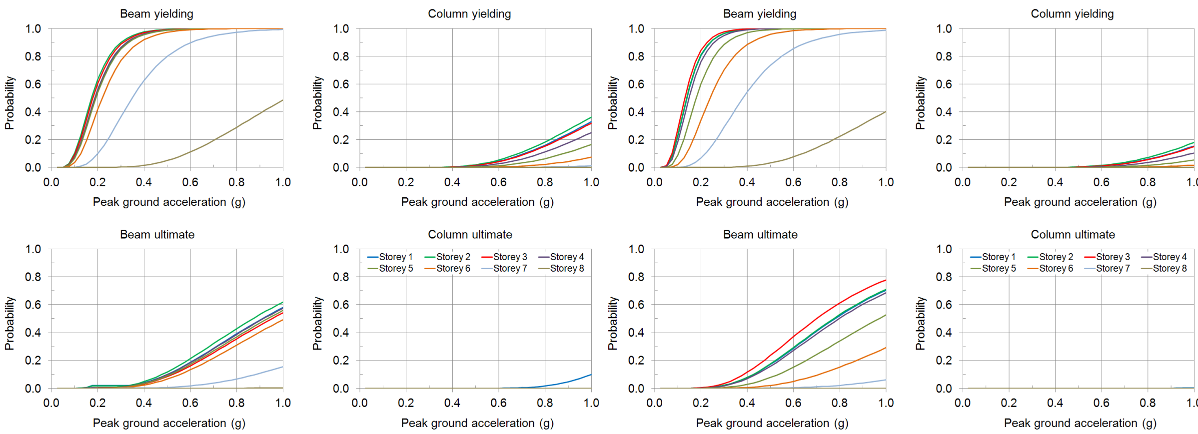


**Figure 4.3.** Fragility curves of 8-storey frame buildings satisfying EC2's slenderness limits and designed to EC8 for PGA = 0.3g with DC M (left), or H (right)

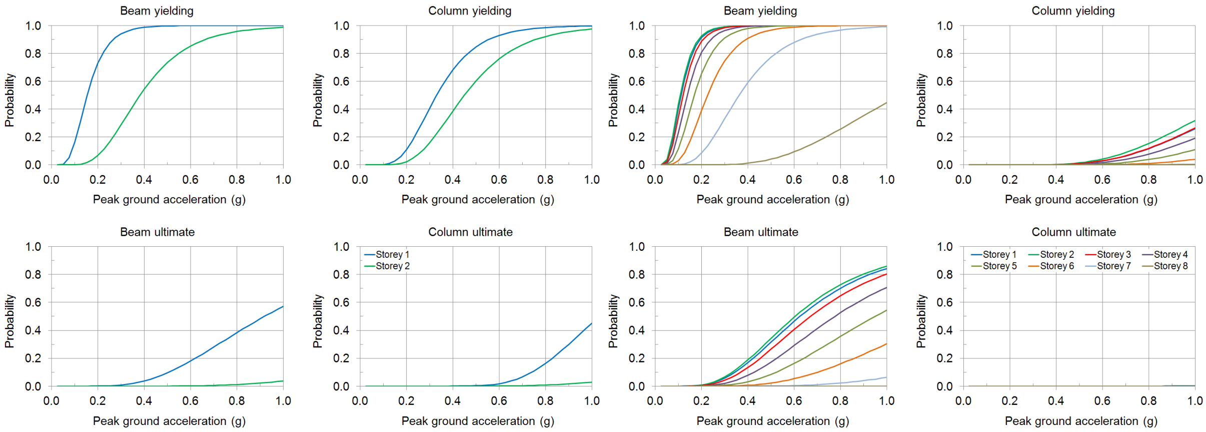


**Figure 4.4.** Fragility curves of frame buildings satisfying EC2's slenderness limits and designed to EC8 for DC H and PGA = 0.2g: (left) 2 storeys; (right) 8 storeys

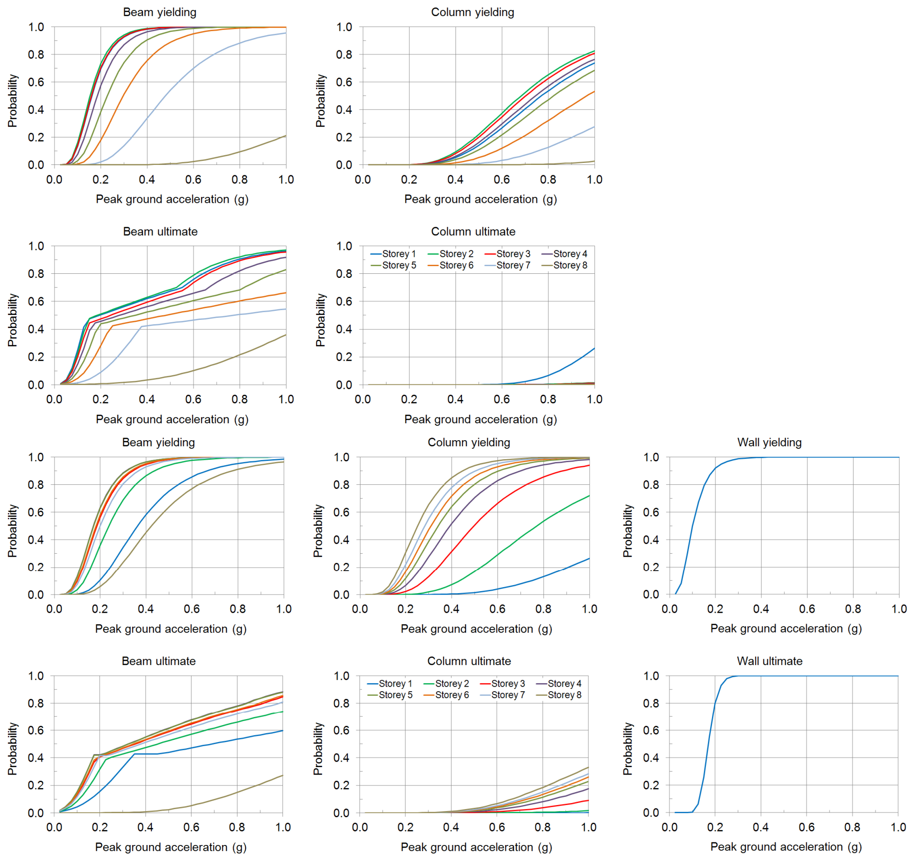


**Figure 4.5.** Fragility curves of 8-storey buildings designed to EC2 only for gravity: (top) frame building designed as unbraced; (bottom) wall building designed as braced

The above conclusions are confirmed in Tables 4.1 to 4.5 that give the median values of PGA at which the yielding and ultimate damage states are attained. The values for frame buildings meeting the simplified EC2's slenderness limits are listed under the heading EC2/EC8; for those that violate them under the heading EC8. The minimum values for beams and columns among all storeys are given; normally the elements of the two lower storeys are critical. A dash indicates that this type of element attains this damage state in the mean for PGA > 1.0g. Note in Table 4.1 that the design PGA does not affect the fragility of 2-storey ductile frames that satisfy EC2's slenderness limits, because in those buildings the dimensions of beams and columns are dictated by the requirements for negligible 2$^{nd}$-order effects and the minimum reinforcement is sufficient to meet the ULS verifications.

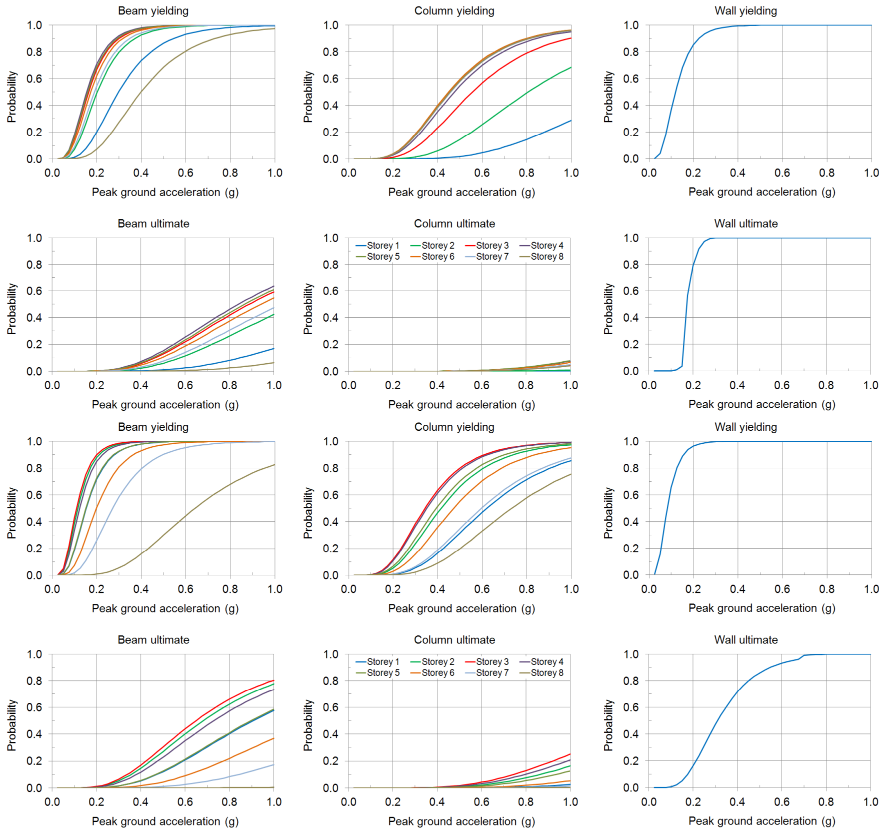


**Figure 4.6.** Fragility curves of 8-storey dual buildings designed to EC8 for DC H and PGA = 0.2g: (top) wall system; (bottom) frame-equivalent dual system

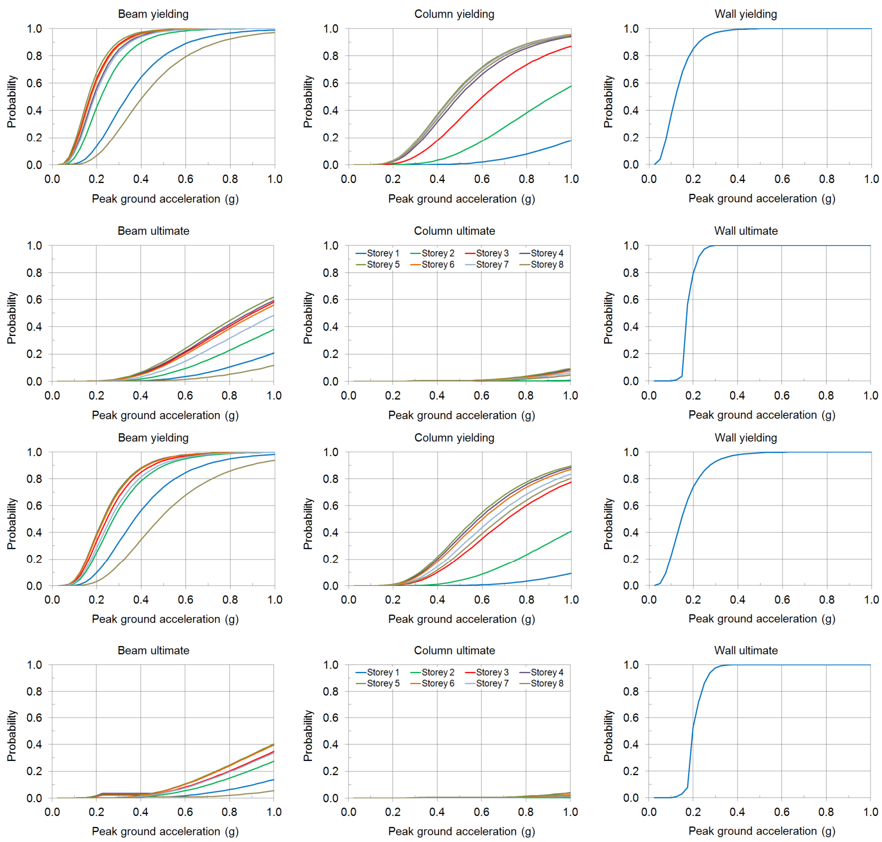


**Figure 4.7.** Fragility curves of 8-storey wall buildings designed to EC8 for DC M and PGA = 0.20g (top) or PGA = 0.30g (bottom)

**Table 4.1.** Median PGA (g) at attainment of the damage states in 2-storey frame buildings

| | | EC2/EC8 | | | | EC8 | | | |
|---|---|---|---|---|---|---|---|---|---|
| design PGA | DC | Beam yielding | Beam ultimate | Column yielding | Column ultimate | Beam yielding | Beam ultimate | Column yielding | Column ultimate |
| 0.10 | L | 0.25 | 0.73 | 0.23 | 0.90 | 0.30 | 0.78 | 0.17 | 0.55 |
| 0.15 | L | 0.30 | 0.93 | 0.22 | 0.85 | 0.34 | 0.98 | 0.20 | 0.46 |
| 0.15 | M | 0.16 | 0.85 | 0.35 | 0.84 | 0.14 | 0.72 | 0.20 | 0.86 |
| 0.20 | M | 0.16 | 0.85 | 0.35 | 0.84 | 0.14 | 0.72 | 0.20 | 0.86 |
| 0.25 | M | 0.16 | 0.85 | 0.35 | 0.84 | 0.15 | 0.77 | 0.34 | 0.81 |
| 0.30 | M | 0.16 | 0.85 | 0.35 | 0.84 | 0.16 | 0.86 | 0.31 | 0.73 |
| 0.20 | H | 0.15 | 0.92 | 0.33 | - | 0.13 | 0.82 | 0.19 | 0.89 |
| 0.25 | H | 0.15 | 0.92 | 0.33 | - | 0.12 | 0.80 | 0.28 | - |
| 0.30 | H | 0.15 | 0.92 | 0.33 | - | 0.16 | 0.99 | 0.25 | 0.93 |
| 0.35 | H | 0.15 | 0.92 | 0.33 | - | 0.15 | 0.92 | 0.33 | - |

**Table 4.2.** Median PGA (g) at attainment of the damage states in 5-storey frame buildings

| | | EC2/EC8 | | | | EC8 | | | |
|---|---|---|---|---|---|---|---|---|---|
| design PGA | DC | Beam yielding | Beam ultimate | Column yielding | Column ultimate | Beam yielding | Beam ultimate | Column yielding | Column ultimate |
| 0.10 | L | 0.24 | 0.82 | 0.34 | - | 0.24 | 0.29 | 0.24 | 0.38 |
| 0.15 | L | 0.30 | - | 0.30 | 0.46 | 0.30 | 0.97 | 0.23 | 0.69 |
| 0.15 | M | 0.12 | 0.56 | 0.86 | - | 0.11 | 0.59 | 0.35 | 0.86 |
| 0.20 | M | 0.13 | 0.65 | 0.84 | - | 0.13 | 0.63 | 0.33 | 0.77 |
| 0.25 | M | 0.16 | 0.77 | 0.74 | 0.95 | 0.16 | 0.78 | 0.50 | 0.91 |
| 0.30 | M | 0.18 | 0.91 | 0.78 | - | 0.18 | 0.91 | 0.78 | - |
| 0.20 | H | 0.12 | 0.66 | 0.73 | - | 0.12 | 0.72 | 0.27 | 0.91 |
| 0.25 | H | 0.13 | 0.70 | 0.68 | - | 0.13 | 0.72 | 0.51 | - |
| 0.30 | H | 0.13 | 0.73 | - | - | 0.13 | 0.73 | - | - |
| 0.35 | H | 0.16 | 0.88 | 0.99 | - | 0.16 | 0.88 | 0.99 | - |

**Table 4.3.** Median PGA (g) at attainment of the damage states in 8-storey frame buildings

| | | EC2/EC8 | | | | EC8 | | | |
|---|---|---|---|---|---|---|---|---|---|
| design PGA | DC | Beam yielding | Beam ultimate | Column yielding | Column ultimate | Beam yielding | Beam ultimate | Column yielding | Column ultimate |
| 0.10 | L | 0.23 | 0.50 | 0.57 | - | 0.24 | 0.83 | 0.39 | - |
| 0.15 | L | 0.28 | 0.26 | 0.38 | - | 0.29 | 0.28 | 0.33 | 0.94 |
| 0.15 | M | 0.11 | 0.56 | - | - | 0.11 | 0.58 | 0.83 | - |
| 0.20 | M | 0.14 | 0.66 | 1.00 | - | 0.14 | 0.66 | 0.80 | - |
| 0.25 | M | 0.16 | 0.78 | 0.92 | - | 0.15 | 0.77 | 0.83 | - |
| 0.30 | M | 0.17 | 0.87 | - | - | 0.17 | 0.87 | - | - |
| 0.20 | H | 0.11 | 0.60 | - | - | 0.11 | 0.60 | - | - |
| 0.25 | H | 0.12 | 0.65 | - | - | 0.12 | 0.65 | - | - |
| 0.30 | H | 0.13 | 0.70 | - | - | 0.13 | 0.76 | - | - |
| 0.35 | H | 0.15 | 0.84 | - | - | 0.16 | 0.84 | - | - |

**Table 4.4.** Median PGA (g) at attainment of the damage states in 5-storey dual buildings

| | | Frame-equivalent dual system | | | | | | Wall system | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| design PGA | DC | Beam yielding | Beam ultimate | Column yielding | Column ultimate | Wall yielding | Wall ultimate | Beam yielding | Beam ultimate | Column yielding | Column ultimate | Wall yielding | Wall ultimate |
| 0.10 | L | 0.25 | 0.70 | 0.23 | 0.87 | 0.11 | 0.22 | 0.30 | 0.40 | 0.26 | - | 0.11 | 0.16 |
| 0.15 | L | 0.30 | 0.94 | 0.22 | 0.86 | 0.13 | 0.21 | 0.35 | - | 0.25 | - | 0.13 | 0.17 |
| 0.15 | M | 0.12 | 0.62 | 0.35 | - | 0.09 | 0.25 | 0.15 | 0.78 | 0.41 | - | 0.09 | 0.17 |
| 0.20 | M | 0.12 | 0.62 | 0.35 | - | 0.09 | 0.25 | 0.16 | 0.84 | 0.41 | - | 0.11 | 0.17 |
| 0.25 | M | 0.16 | 0.83 | 0.39 | 0.94 | 0.11 | 0.19 | 0.22 | - | 0.47 | - | 0.13 | 0.18 |
| 0.30 | M | 0.18 | 0.96 | 0.53 | 0.92 | 0.13 | 0.20 | 0.31 | - | 0.77 | - | 0.17 | 0.28 |
| 0.20 | H | 0.11 | 0.71 | 0.28 | - | 0.09 | 0.34 | 0.15 | 0.80 | 0.34 | - | 0.10 | 0.31 |
| 0.25 | H | 0.14 | 0.83 | 0.38 | - | 0.10 | 0.38 | 0.19 | - | 0.45 | - | 0.12 | 0.35 |
| 0.30 | H | 0.16 | 0.92 | 0.51 | - | 0.12 | 0.47 | 0.26 | - | 0.75 | - | 0.17 | 0.40 |
| 0.35 | H | 0.19 | - | 0.52 | - | 0.12 | 0.39 | 0.28 | - | 0.73 | - | 0.16 | 0.42 |

**Table 4.5.** Median PGA (g) at attainment of the damage states in 8-storey dual buildings

| design PGA | DC | Frame-equivalent dual system | | | | | | Wall system | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Beam yielding | Beam ultimate | Column yielding | Column ultimate | Wall yielding | Wall ultimate | Beam yielding | Beam ultimate | Column yielding | Column ultimate | Wall yielding | Wall ultimate |
| 0.10 | L | 0.22 | 0.69 | 0.26 | - | 0.12 | 0.33 | 0.27 | 0.81 | 0.30 | - | 0.12 | 0.17 |
| 0.15 | L | 0.28 | 0.97 | 0.25 | - | 0.15 | 0.28 | 0.33 | - | 0.28 | - | 0.14 | 0.19 |
| 0.15 | M | 0.10 | 0.50 | 0.37 | - | 0.07 | 0.23 | 0.14 | 0.71 | 0.46 | - | 0.11 | 0.16 |
| 0.20 | M | 0.13 | 0.64 | 0.34 | - | 0.08 | 0.31 | 0.16 | 0.86 | 0.46 | - | 0.12 | 0.17 |
| 0.25 | M | 0.14 | 0.75 | 0.33 | - | 0.09 | 0.33 | 0.20 | - | 0.42 | - | 0.13 | 0.18 |
| 0.30 | M | 0.17 | 0.85 | 0.50 | - | 0.13 | 0.22 | 0.23 | - | 0.57 | - | 0.15 | 0.20 |
| 0.20 | H | 0.11 | 0.65 | 0.34 | - | 0.09 | 0.38 | 0.15 | 0.84 | 0.45 | - | 0.11 | 0.29 |
| 0.25 | H | 0.14 | 0.79 | 0.32 | - | 0.10 | 0.40 | 0.19 | 0.92 | 0.42 | - | 0.12 | 0.28 |
| 0.30 | H | 0.14 | 0.79 | 0.49 | - | 0.12 | 0.35 | 0.19 | 1.00 | 0.60 | - | 0.14 | 0.26 |
| 0.35 | H | 0.15 | 0.80 | 0.64 | - | 0.13 | 0.38 | 0.22 | - | 0.75 | - | 0.15 | 0.29 |

## 5. CONCLUSIONS

The performance goals of EC8 are met in a very consistent and uniform way across all types of buildings considered and their geometric or design parameters, except in RC walls, which fail early in shear despite their design against it. In fact, these walls do not perform much better than those of braced systems designed to EC2 alone.

The slenderness limits and the lateral bracing requirements of EC2 place severe restrictions on the size of columns and walls, which, although ignored in ordinary seismic design practice, materially impact the outcome of the design and, to a smaller extent, the seismic fragilities of the building's members.

The reduction in fragility from higher design PGA is disproportionately low. Even buildings designed for gravity loads only, but in full accordance to EC2, possess substantial seismic resistance.

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