A CASE STUDY IN PERFORMANCE-BASED DESIGN USING YIELD FREQUENCY SPECTRA

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Abstract: An analytical formulation is offered to allow performance-based seismic design to be achieved following a direct code-compatible procedure. The approach builds upon the use of the yield displacement as a robust system characteristic. A new format for displaying seismic demands known as Yield Frequency Spectra is introduced to quantitatively link performance objectives with the base shear seismic coefficient for a fixed value of yield displacement. Analytical expressions allow estimating the design base shear strength to satisfy any number of performance requirements, foregoing the need for a behaviour factor. The effect of uncertainties is naturally introduced to inject the proper conservatism for, e.g., the natural randomness in the ground motion or lack of knowledge in modelling and analysis. Finally, an 8-story reinforced concrete frame is designed, showing that EN1998 may not achieve the stated performance targets, while the proposed approach can match them with a single iteration.

Introduction

As a result of economic damage in the 1994 Northridge and 1995 Hyogo-Ken Nambu (Kobe) earthquakes, significant attention has been directed at augmenting the life safety performance objective, characteristic of traditional codes, with additional criteria to limit economic losses in more frequent earthquakes. Basic notions of performance-based design, elaborated in the Vision 2000 report (SEAOC 1995), are widely accepted and are now incorporated in mainstream documents such as ASCE/SEI 41/06 (ASCE 2007) and EN1998 (CEN 2004). Several approaches have been suggested, mainly conforming to the displacement-based design paradigm, as presented by Moehle (1992), Priestley (2000) and Aschheim (2002). They invariably incorporate some form of an equivalent single-degree-of-freedom (or first-mode) representation for use in preliminary design. More importantly, they use as starting point an estimate of the yield displacement, rather the fundamental period of the structure, the former being a more stable parameter for a given structural configuration (Aschheim 2002). On the other hand, though, they are deterministic in focusing design on a specific intensity of shaking represented by a design response spectrum associated with a specified hazard level. The hazard level is typically set at a 10% probability of exceedance in 50 years, equivalent to a 475-year mean return period, or a P_o = –ln(1–0.10)/50 = 0.0021 mean annual frequency (MAF) of exceedance.

When facing the significant uncertainty associated with ground motions, modeling and structural response, deterministic methods are inherently limited. Cornell et al (2002) showed that in the presence of variability due to either aleatory or epistemic sources, the determination of performance at a single level of "design" intensity is unconservative: The more frequent appearance of significant damage at lower levels of intensity will always bias the results. Thus, the use of the "design" intensity results in buildings that may be subject to damage with a higher mean annual frequency of occurrence (greater than the typically desired P_o = 0.0021). To achieve uniform levels of safety in the presence of uncertainties, additional hazard and structural response data are needed, in order to consider site and structure characteristics. At present, modern seismic codes use blanket safety factors, typically embodied into the definition of the strength reduction factor R (or behavior factor q)

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and other design requirements that provide inconsistent levels of safety, even for different buildings within the same class and site. Still, despite the apparent advantages, a fully probabilistic performance-based seismic design approach is difficult to achieve in practice. Design is an inverse problem that, in the case of earthquakes, is based on the non-invertible nonlinear relationships between seismic intensity and structural demands. Thus, iterations are needed, in which each cycle involves the re-design of the structure and its full performance-based assessment via nonlinear static or dynamic procedures (e.g., Krawinkler et al 2006).

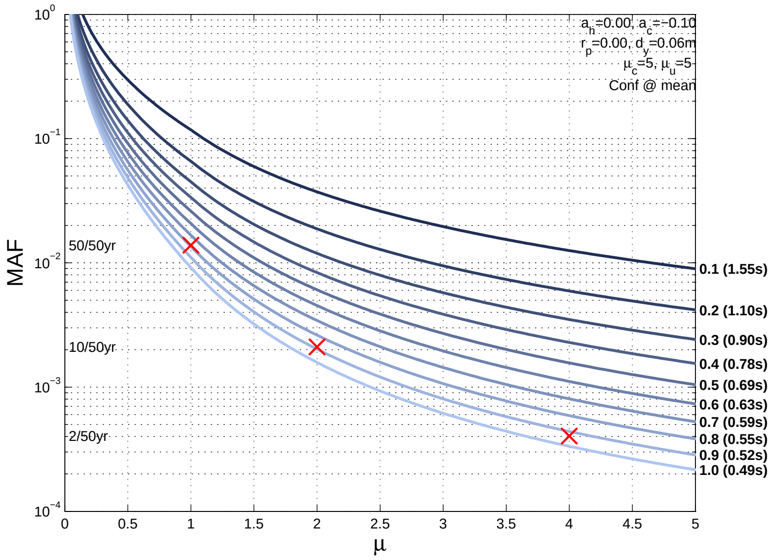

Figure 1. YFS contours at C_y = 0.1,0.2,…,1.0 for an elastoplastic system (δ_y = 0.06m) for a site in Los Angeles, CA, overlaid by the design points of three performance objectives for μ = 1, 2, 4 at 50%, 10% and 2% in 50yrs rates, respectively. The oscillator must have sufficient strength to satisfy each performance objective; the third objective governs with base shear coefficient of $C_y \approx 0.93$ and a period of $T \approx 0.51$s

As a partial solution, Vamvatsikos et al (2013) proposed "Yield Frequency Spectra" (YFS) as a rapid means to establish the strength required for a preliminary design to provide a desired level of confidence in satisfying one or more performance objectives related to system drift and ductility demands. YFS provide a visual representation of a system's performance that quantitatively links the MAF of exceeding any displacement value (or ductility μ) with the system yield strength (or seismic coefficient at yield, C_y). As with other methods, an "equivalent" single-degree-of-freedom model is utilized to establish the preliminary design, which may be based on current code criteria. Fig.1 presents an example of YFS developed for an elastic-perfectly-plastic oscillator. In this case, three performance objectives are specified (the red "x" symbols) while curves representing the site hazard convolved with the system fragility are plotted for fixed values of C_y. Thus, the minimum acceptable C_y that fulfils the set of performance objectives for the site hazard can be readily determined.

Code-compatible YFS design formulation

YFS application necessitates the use of a full set of hazard curves: One for each period of interest for the specific site. For cases where such complete information is not available, it would be desirable to have at least an approximate solution that can be based on the basic tools of the seismic code: Smoothed, uniform hazard design spectra with the addition of an estimate of the slope of the hazard curve in the region of interest. Following the work of Vamvatsikos and Aschheim (2014), we shall outline the derivation of a set of closed-form expressions that can achieve YFS-like results with a minimum of computations.

First of all, let us consider the equivalent single-degree-of-freedom (SDOF) representation of the system. The response of such an oscillator having yield strength, F_y, and reactive weight, W, can be described in normalized terms, where the base shear coefficient at yield, C_y, is defined by

$$C_y = \frac{F_y}{W}. \tag{1}$$

While (pseudo) spectral acceleration is defined in relation to spectral displacement, in common usage the spectral acceleration associated with a yielding system, $S_{ay}(T)$, is equivalent to $C_y g$. For a system with a given yield displacement, δ_y, changes in C_y represent changes in both strength and stiffness, and hence result in a change in period. For yielding SDOF systems,

$$\begin{aligned} T &= 2\pi\sqrt{\frac{\delta_y}{C_y g}} \\ C_y &= \frac{\delta_y}{g}\left(\frac{2\pi}{\mathrm{T}}\right)^2 \end{aligned} \tag{2}$$

The normalized response of an oscillator having peak displacement δ is given by the ductility, μ,

$$\mu = \frac{\delta}{\delta_y} \tag{3}$$

with the ductility level μ_{lim} corresponding to a limit-state ductility value equal to $\mu_{lim} = \delta_{lim}/\delta_y$.

At this point, one can employ the SAC/FEMA (2000) expressions proposed by Cornell et al (2002) to join together the base shear coefficient, the spectral acceleration demand, S_{ao}, and the limiting ductility μ_{lim} in a single equation (Vamvatsikos and Aschheim 2014):

$$C_y = \frac{S_{ao}}{g\mu_{lim}^{1/b}} \cdot \exp\left[\frac{k_1}{2b^2}\beta_{T\theta}^2\right] \tag{4}$$

where b is the local slope of the demand-intensity relationship, k_1 is the local slope of the hazard curve (both slopes in log-log space), g is the gravity acceleration, and $\beta_{T\theta}$ is the total dispersion in seismic demand given the intensity level. In the case of EN1998 (CEN 2004), a value of $k_1 = 3$ is generally suggested for Europe at the 10% in 50 years level.

Still, Eq. (4) necessitates iterations due to the dependence of b and $\beta_{\theta d}$ on the system period. To remove this complication, the influence of period should be introduced explicitly into Eq. (4). For a smoothed design spectrum, the respective design spectral acceleration, S_{ao}, for a given performance objective depends on the portion of the spectrum where the solution resides. Several distinct regions, typically referred to as constant acceleration, constant velocity, and constant displacement, are present in the range of periods (approximately 0.2 to 2.5 sec) that is useful for many practical engineering applications. Each region can be represented using the general form

$$S_{ao}(T) = S_{amax}\left(\frac{T_c}{T}\right)^r, \quad T \in [T_a, T_b] \tag{5}$$

where T_c is a constant (often known as a corner period) with units of sec and S_{amax} is the corresponding value of spectral acceleration, which also happens to be the maximum in the range of validity of Eq. (5). r = 0, 1, 2 for the constant acceleration, velocity and displacement parts of the spectrum, respectively. T_a and T_b (also having units of sec) define the extent of each segment of the spectrum along the period axis. Note that for conservativeness in the long period range, EN1998 may mandate a minimum S_a regardless of period, thus introducing a final constant acceleration region at periods above a certain long-period threshold, typically higher than 4 sec. This also implies $r = 0$ above, albeit with a much decreased value of S_{amax}.

Starting from Eq. (4), we assume a relatively constant (period independent) value for b, $\beta_{\theta d}$ and k_1 in each spectral segment. By introducing Eq. (5), replacing T by its equivalent from Eq. (2), and solving for C_y, one obtains:

$$C_y = \left\{ \frac{S_{amax}}{g\mu_{lim}^{1/b}} \cdot \left(\frac{T_c}{2\pi} \sqrt{\frac{g}{\delta_y}} \right)^r \exp\left[\frac{k_1}{2b^2} \beta_{T\theta}^2 \right] \right\}^{\frac{1}{1-0.5r}} \tag{6}$$

For convenience with units, Eq. (2) contains two instances of g. In the first instance, S_{amax} is normalized by g. In the second instance, g is required in the square root to reconcile the units of δ_y and T. If units of meter and second are employed throughout Eq. (6), then g = 9.81 m/s^2 should be employed in both instances. The general form of Eq. (6) can now be specialized for the three spectral regions. Setting r = 0 for the constant acceleration region we get

$$C_y = \frac{S_{amax}}{g\mu_{lim}^{1/b}} \cdot \exp\left[\frac{k_1}{2b^2} \beta_{T\theta}^2 \right] \tag{7}$$

A rough estimate of b and $\beta_{T\theta}$ in the constant acceleration region is given by $b \approx 1.2$ and $\beta_{T\theta} \approx 0.5$, assuming that $0.2 < T < 0.5$ sec and $\mu_{lim} > 2$. For $\mu_{lim} \leq 1$ the oscillator behaves elastically, therefore b equals 1.0 and a lower $\beta_{T\theta}$ should be adopted instead. For the constant velocity region, r = 1, thus

$$C_y = \left(S_{amax} \cdot \frac{T_c}{2\pi} \right)^2 \frac{1}{g\delta_y \mu_{lim}^2} \exp\left[k_1 \beta_{T\theta}^2 \right] \tag{8}$$

where b ≈ 1 for moderate to long periods and $\beta_{T\theta} \approx 0.45$ for ductilities higher than about 2.

Numerical results obtained with the above equations need to be validated by checking that the corresponding period from Eq. (2) is within the period limits of the corresponding spectral region. Due to the constant (and discontinuous) b and $\beta_{\theta d}$ assumptions involved, it is possible that for some values of δ_y and μ_{lim}, valid solutions for C_y may be obtained in both the constant acceleration and constant velocity regions. In such cases, it is advisable to select the constant velocity solution for corner periods $T_c \geq 0.5$ sec and the constant acceleration solution otherwise, simply by virtue of where the corresponding b and $\beta_{\theta d}$ assumptions are most accurate. Alternatively, one may simply use more accurate estimates of b and $\beta_{T\theta}$ that may be derived, for example, from the work of Vamvatsikos and Cornell (2006) or Ruiz-Garcia and Miranda (2007).

For the constant displacement region, Eq. (6) cannot be solved for a unique C_y. Rather, it becomes an identity, for which all feasible C_y's (or systems) in this region are acceptable. To wit: all elastic oscillators in this region have the same spectral displacement; due to the equal displacement "rule" (enforced by b = 1), oscillators of varied strengths having a given yield displacement have the same peak displacement, and thus the same ductility demand. Mathematically acceptable solutions can be obtained for each period, although realistic solutions may be constrained to certain ranges of period or yield displacement. But even the mathematical solutions represent ideal conditions—in reality the spectrum never truly conforms to the r = 2 shape, nor is the value of b equal to exactly 1 for any structure, regardless of period, in this spectral region. Thus, using the actual uniform hazard spectrum together with accurate estimates for b and $\beta_{T\theta}$ via Eq. (4) will tend to constrain the appearance of this issue of C_y indeterminacy to a small part of the long-period spectrum. Practically speaking, one can always choose to be conservative by selecting a C_y that would appear by extending the constant-velocity region into the constant displacement range, essentially assuming r = 1 rather than the value of 2.0. Alternatively, one may consider using Eq. (6) with r = 1.5 or 1.8 as a better approximation. Either way, one would rarely expect to encounter such flexible systems in most realistic applications.

Case study Eurocode-based design

To illustrate the application of the proposed YFS framework, an 8-story reinforced concrete (RC) space frame building is studied. It is a common structural configuration in earthquake-prone areas (e.g., USA or Europe) for either office or residential buildings. Figure 2a shows the plan view of the building, consisting of four frames, each one of three bays along the two main horizontal directions. The overall plan dimensions are 18.30m x 18.30m while the total height is H_{tot} = 32.60m, with a 4.60m high first story and 4.0m stories thereafter. The taller first story, dictated mainly by architectural or functional reasons, creates a slightly discontinuous distribution of stiffness with elevation (Figure 2b). The floor system is waffle slab, allowing for lower dead loads. Live load of 2.0 kN/m^2 and permanent load of 1.50kN/m^2 were assumed, while an additional permanent load of 1.20 kN/m^2 was considered for the perimeter beams due to cladding.

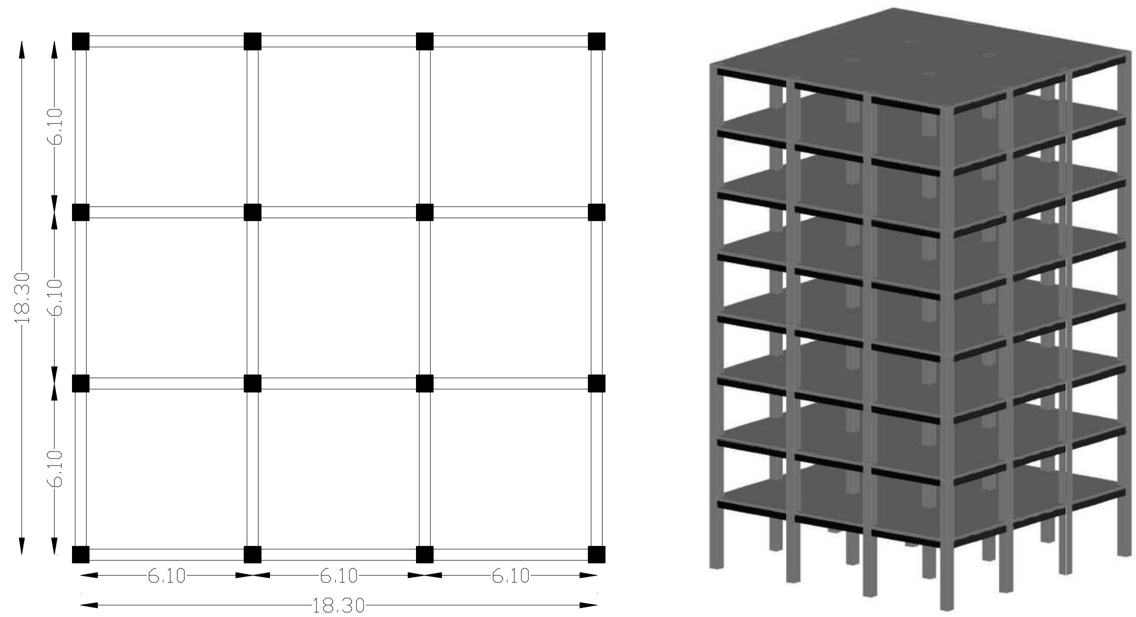

Figure 2. (a) Plan view of a typical story and (b) 3D depiction of RC space frame building

The benchmark building was initially designed and detailed according to EN1990, EN1992 and EN1998 provisions (CEN 2002a, 2002b, 2004). A three dimensional model was created for the structural design realization using a commercial structural design package. Seismic loads, being consistent with the highest importance factor (γ_I = 1.4) imposed by EN1998 and a reference peak ground acceleration, a_{gR} = 0.36g, were accounted for the modal response spectrum analysis. The building is assumed to be founded on firm soil conditions (soil type B). The smoothed design response spectra is defined by periods of T_b = 0.15s, T_c = 0.5s and T_d = 2s, as recommended in EN1998 for a high-seismicity Type 1 spectra: The constant acceleration plateau, spanning between T_b and T_c, is followed by a constant velocity region up to T_d, at which the constant displacement region begins. For soil type B, the maximum plateau acceleration for the Strength Limitation (SL) limit-state is $S_{amax,SL}$ = 1.51g, which is reduced by 60% to check for the Damage Limitation (DL) limit-state (as required for importance class 3), $S_{amax,DL}$ = 0.60g.

Moreover, the High Ductility Class (DCH) considered herein ensures the existence of a stable and trustworthy system of absorbing high levels of inelastic deformation in predefined critical areas of the primary structural members. The behavior factor, q, was calculated equal to 5.85, indicative of the multi-story and multi-bay, highly ductile frame system. The selection of such a highly demanding structural realization (i.e., increased seismic loads along with the consideration of DCH) was motivated by the need to validate the currently proposed YFS framework for an extreme design case identifying, at the same time, potential limitations that may emerge. Concrete class C35/45 and steel grade S500 were adopted throughout the structure. As expected for a flexible system, the design was primarily controlled by EN1998-imposed drift limitations, i.e., the DL performance objective) and hence, the associated increased stiffness requirements dictated the use of large cross-sections for the structural members: 60cm x 60cm columns and 60cm x 40cm beams.

A two-dimensional, 8-story and three-bay model was created with the use of Opensees finite element code (Mazzoni et al 2000) using fiber elements that can simulate concrete cracking. The eigenvalue analysis of the model indicated an elastic (uncracked) first mode of T_1 = 1.233s, while a first-mode pushover analysis suggested an effective (cracked) period of T_{eff} = 1.656s. Incremental Dynamic Analysis (IDA, Vamvatsikos and Cornell 2002) was performed to assess the actual performance of the system. The convolution of the seismic hazard and the fragility (IDA curves) for the case study building allowed for calculating the MAF of exceeding the target P_o. Given that virtually no scaling is required for matching the DL state accelerations, even the use of $S_a(T_1)$ as the intensity measure is sufficient for a rigorous performance assessment of the code-based design. MAF calculations can be carried out either numerically or with an analytical approximation (Vamvatsikos 2014), the former adopted herein. By estimating a record-to-record dispersion of nearly 20% for the DL limit state, the actual MAF of exceeding 0.75% limiting interstory drift was calculated equal to 0.0174, significantly higher (65%) than the maximum allowable MAF of 0.0105. In other words, the initial design of the 8-story RC space frame failed to satisfy the performance criterion imposed by EN1998. This may be the expected result wherever large ductilities and high seismicity zones are involved, as the large response variability tends to exacerbate the inaccuracies of single-intensity-level approaches. Still, we should be careful not to generalize this conclusion; in many other cases, EN1998 may actually be found to overcompensate in the opposite direction, being quite conservative and often uneconomical.

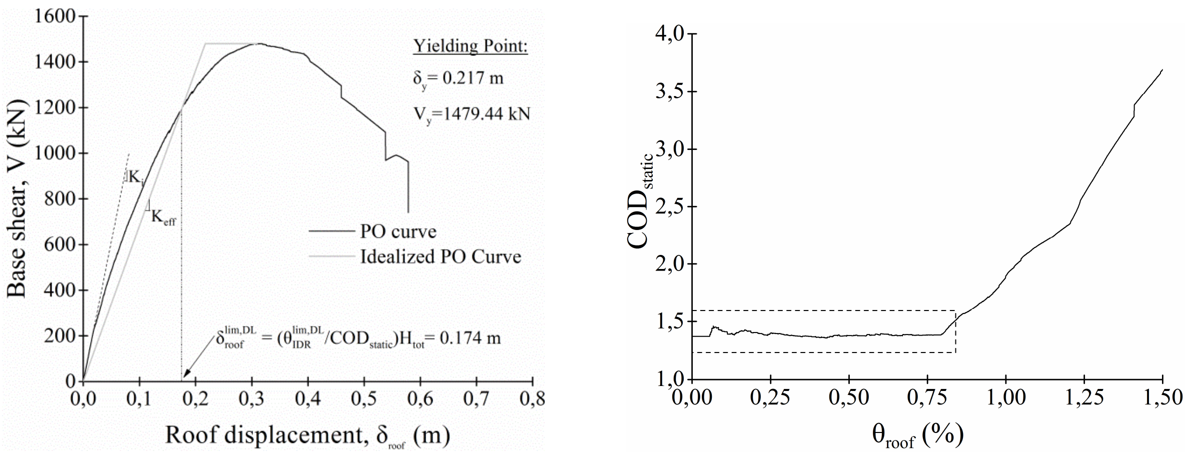

Figure 3. Inelastic Static Analysis of the code-compatible designed 8-story, R/C space frame system: (a) Pushover curve and its bilinear idealization, (b) COD_{static} calculation.

Case study YFS-based design

The proposed Yield Frequency Spectra framework will be employed to redesign the structure. First we need to determine the design requirements in terms of the system's yield strength (or seismic coefficient), C_y, and the structural period, T, both of them tied together via the yield displacement, δ_y in Eq.(1). A simple bilinear elastic-plastic law was considered to model the system's response. Additionally, the yield displacement, δ_y, which is a cornerstone parameter for the YFS calculation, was determined via a first-mode pushover analysis of the space-frame model (Fig. 3a) as $\delta_y = 0.217$ m. Thus, the initial estimate for the ESDOF system's yield displacement is reasonably defined as:

$$\delta_y^* = \frac{\delta_y}{\Gamma} \tag{9}$$

where Γ is the first-mode participation factor.

Employing the results of an eigenvalue analysis yields a Γ = 1.30 for the case study building. However, this definition of Γ neglects the contribution of periods other than T_1 to the dynamic response of such a high-rise, moment-resisting, frame system. In the case at hand, higher modes as well as the elongation of the first mode due to cracking can cause such an effect. Thus, a "multi-modal" approximation was considered to reflect the contribution of additional periods to the equivalent system's yield displacement. To this end, the parameter G was introduced as a multi-modal substitute of the first-mode participation factor Γ that can offer an accurate estimate of roof drift given the spectra acceleration in the elastic region. By inverting the widely used formula for the target displacement (e.g. from ASCE/SEI 41-06), one can obtain:

$$G = \frac{\delta_{roof}}{S_a(T_{eff})}\frac{4\pi^2}{T_{eff}^2} \tag{10}$$

In order to define a representative median ratio of roof drift, δ_{roof}, over the corresponding spectral acceleration, $S_a(T_{eff})$, modal response spectrum analyses can be employed for several ground motion records to capture at least the higher mode contribution. Alternatively, one can employ the "elastic" part of the 50% fractile IDA curve (Fig. 4) for a more accurate assessment. Thus, a median value of G equal to 1.64 was determined in the (nominally) elastic range of the case study building (i.e., $0 \le S_a(T_{eff}) \le 0.30$ g). The substitution of Γ with G in Eq. (5) led to an estimate for the yield displacement, δ_y^*, equal to 0.129 m. Such a higher value of G versus Γ (up to 26%) is captured efficiently by the dynamic analysis results.

To limit damages under frequent earthquakes of low intensity, the EN1998-imposed serviceability interstory drift limit of 0.75% was considered herein, along with the corresponding MAF of 0.0105, consistent with an exceedance probability of 10% in 10 years. The associated limiting ductility for DL can be expressed as:

$$\mu_{DL} = \frac{\delta_{DL}}{\delta_y} = \frac{\theta_{MIDR,DL} / COD}{\theta_{roof,y}} \tag{11}$$

where $\theta_{roof,y}$ is the roof drift at yield, equal to δ_y/H_{tot}=0.665%. The coefficient of distortion (COD) relates the maximum interstory drift, θ_{MIDR}, with the corresponding (typically lower in value) roof drift. This is defined by Moehle (1992) as COD = $\theta_{MIDR}/(\delta_y/H)$. In the (nominally) elastic region the COD_{static} parameter can be derived on the basis of the pushover analysis results neglecting, though, the contribution of the higher modes (Fig. 3b). For higher accuracy, a "multi-modal" approximation was considered again and the adopted $COD_{dynamic}$ can be derived either by modal response spectrum analyses or nonlinear dynamic analyses for a set of ground motion records. Particularly, for a given $S_a(T_{eff})$, both θ_{MIDR} *and* θ_{roof} were determined from the median IDA capacity curve provided in Figure 5 and a $COD_{dynamic}$ equal to 1.517, slightly higher than COD_{static}, was obtained by the average of the θ_{MIDR} over θ_{roof} ratios of each record. Eventually, using the term of $COD_{dynamic}$ in Eq. (11), the limiting ductility was found to be μ_{DL} = 0.759.

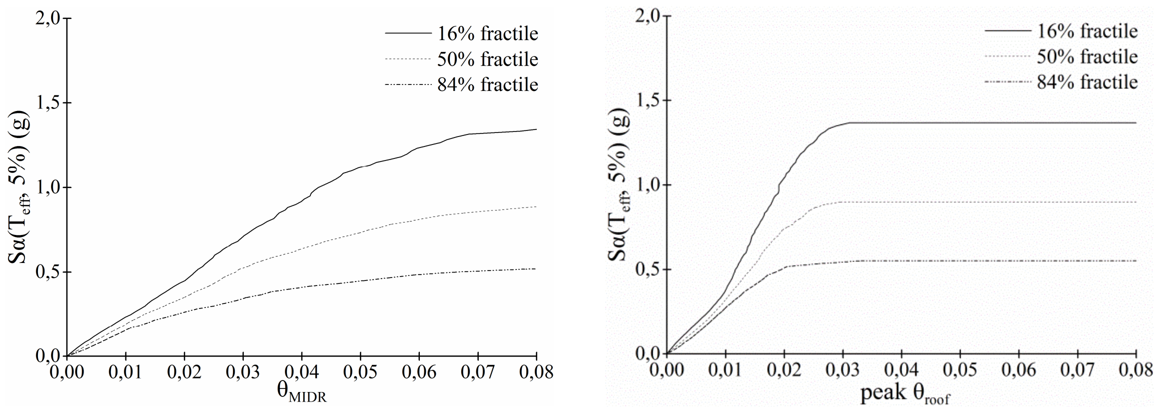

Figure 4. Summary (16%, 50% and 84% fractiles) of the IDA curves for the code-compatible designed 8-story, RC space frame system: (a) $Sa(T_{eff})$ versus max interstory drift, θ_{MIDR}, (b) $Sa(T_{eff})$ vs peak roof drift, θ_{roof}.

Within the YFS framework, the definition of the performance levels also involves choosing the magnitude of the related aleatory and epistemic uncertainty, represented by the total dispersion of $\beta_{T\theta}$. Higher epistemic uncertainty is, in general, associated with deficient knowledge about the actual structural capacity and demand. Normally, lower values of epistemic uncertainty are related to the elastic and nearly elastic response of structural systems, while high levels of inelastic deformations are associated with increasing values for $\beta_{T\theta}$. Similarly, higher ductilities and larger influence of higher modes will increase the aleatory component of $\beta_{T\theta}$. For making sure that a fair comparison is made with respect to EN1998, no epistemic uncertainty was considered. While it may be argued that record-to-record aleatory randomness is not needed for an “elastic” limit-state, this is not the case for the building at hand. Record-to-record variability appears even below the nominal yield point, both due to cracking nonlinearity and thanks to presence of the higher modes. Deriving some help from the IDA results, a total (aleatory only) dispersion value of 20% was selected.

As anticipated, DL governs for a high-rise moment resisting frame. By supplying T_c = 0.5s, $S_{amax,DL}$ = 0.60g, μ_{DL} = 0.759, $\beta_{T\theta}$ = 0.2 and k_1 = 2.5 (reduced from the EN1998 recommended value of 3.0 as DL corresponds to a lower probability of exceedance) the critical yield strength coefficient was calculated equal to, C_y = 0.338. The corresponding period, T_{YFS} (Eq.1), was derived as 1.24s, while the required base shear strength at the yield was determined as

$$V_{y,YFS} = a_1 C_y W \tag{12}$$

α_1 is typically assumed to be the first mode mass participation factor. Still, this is not considered an appropriate representation of the participating mass of all modes, thus a value of 1.0 was assumed instead. *W* is equal to 7469kN (mass of 761.352tn), representing the total weight of the case study building. The required (YFS-derived) base shear at the yield was finally estimated as $V_{y,YFS}$ = 2524kN.

Particularly, both the shorter structural period and the higher yield base shear, required by the proposed methodology for the PBSD (i.e., T_{YFS} = 1.24s versus T_{eff} = 1.67s and $V_{y,YFS}$ = 2524kN versus V_y = 1469kN), signify that this multi-story, moment-resisting RC frame has to become significantly stiffer and (for a constant δ_y) also stronger as a means to provide sufficient resistance to the seismic loads. The necessity for redesigning the frame structure, highlighted by the YFS application, was also in full agreement with the outcome of the performance assessment of the building, where the actual MAF of exceeding the target P_o exceeded by 65% the maximum allowable MAF related to the dominant DL performance level.

According to the YFS application for redesigning the space frame building, (a) the structural (cracked) period needs to be close to 1.24 s and (b) the yielding point should correspond to base shear in the vicinity of 2500 kN. This resulted to cross-sections of 70cm x 70cm for columns and 75cm x 55 cm for beams. Along these lines, the eigenvalue analysis of the redesigned frame model resulted in a fundamental (uncracked) vibration period of 0.89 s, which is 28% shorter than the first-mode period related to the initially designed frame model (i.e., $T_1 = 1.23$ s). The resulting effective period, calculated according to Eq. (3), was $T_{eff} = 1.246$ s, almost identical to the required $T_{YFS} = 1.24$s. The actual MAF of exceeding DL for the redesigned building was eventually calculated equal to 0.0097, which is lower by 8.2% than the maximum allowable MAF of 0.0105. In other words, the YFS framework, reached rapidly a redesign alternative that accurately fulfilled the imposed performance targets in a single step. In contrast, these were not satisfied by the typical, code-compatible design procedure.

Conclusions

Yield Frequency Spectra offer a robust framework for deriving the initial seismic design parameters to satisfy any desired performance objective. Using analytical expressions, such estimates can be achieved without any complex calculations. While nonlinear dynamic analyses, e.g., in the form of IDA, need to be employed to verify the final structure's performance, it is expected that the preliminary design delivered will be much closer to the stated requirements than the product of Eurocode 8, or other similar force-based approaches. At least for the case study at hand, a single iteration was all that was needed to achieve near-perfect compliance.

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