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OPINION

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Some issues in the practical application of risk-targeted ground motions

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Abstract

Risk-targeted spectra for seismic design have been proposed in earthquake engineering literature to harmonize seismic reliability for different structures designed at different sites. Such a proposal has been motivated by the fact that designing for uniform seismic hazard across building sites has been shown to generally lead to non-uniform seismic risk. This note uses a case study implementation of the risk-targeted design actions philosophy for seven Italian sites, to showcase two noteworthy practical issues with this otherwise appealing approach. First, at lower-to-moderate-hazard sites, which may be the majority in a country, design against gravity loads and detailing according to minimum code requirements, can result in higher-than-anticipated overstrength, not commensurate with the adopted level of seismic design loads, thus deviating from the target reliability. This leaves only higher-hazard sites with real margins for homogenization of reliability, which raises the bar. Second, risk-targeted spectrum approaches typically require some a-priori assumptions for structural fragility at low-performance objectives, corresponding to alleged high damage. These assumptions, among others, carry an implicit adoption of shaking intensity measures with which to express fragility, whose lack of sufficiency and efficiency may in turn reduce the level of homogenization of risk that can be achieved.

KEYWORDS
design spectrum, performance-based design, seismic reliability

1 | INTRODUCTION

Many modern building codes[1,2] adopt a uniform-hazard approach to define seismic design actions. However, research has shown that structures designed under this concept are at different risk, expressed in terms of annual failure rate, λ_f, in locations with different earthquake hazard, despite the commensurate strength allocation.[3–5] Two major factors emerge as possible culprits for this site-to-site discrepancy in terms of analytically computed reliability. First, are the differences in the shape of site-specific hazard curves[6] at return periods beyond those of the design intensity.[7] Second, structures may possess seismic overstrength reserves that are not explicitly controlled, as they stem from code detailing requirements, design against other non-seismic actions, for example gravity loads, drift limitations, and other factors inherent in the simplifications of the design procedure; for example, when it is based on elastic design.[8,9]

One potential way of alleviating the seismic reliability discrepancy among sites, is to move away from the uniform hazard concept and tailor the design actions for meeting a specific probabilistic performance objective, which a way to achieve so-called *risk-targeted design*, to which this journal recently dedicated a special issue.[10] This entails choosing an appropriate target rate of failure (often taken as interchangeable with the annual failure probability) to meet said objective, λ_f^*, and making some a-priori assumptions about the corresponding structural fragility, that is, the conditional probability of failure, given the level of shaking intensity.[11,12]

Despite the apparent simplicity of linking the seismic design actions to a risk objective, in practice this approach of deriving, so-called, risk-targeted ground motion (RTGM), presents some issues[13] that this study explores by implementing the original proposal for their derivation by Luco and co-authors,[14] for seven Italian sites. Risk-targeted and uniform-hazard elastic design spectra are calculated for these sites and the available results of a recent research project are used to simulate the design of low- and mid-rise reinforced concrete (RC) moment-resisting frames at each site. Incremental dynamic analysis[15] (IDA) of surrogate inelastic single degree of freedom (SDoF) systems is then used to obtain estimates of each design's reliability in terms of annual failure rate. Comparing these reliability metrics with each other highlights the tradeoff between achieving predefined risk targets and the simplifications inherent in (elastic) design procedures.

2 | CASE-STUDY SITES AND STRUCTURES

For each of the seven Italian sites shown in Figure 1A, a risk-targeted design spectrum was developed setting a target annual failure rate $\lambda_f^* = 2 \cdot 10^{-4}$ and a 10% fragility at the design shaking intensity, or $P\left[f|Sa\left(T\right)=sa_{RT}\right]=0.10$, where f indicates structural failure (i.e., failure to meet design performance requirements), $Sa(T)$ is the spectral pseudo-acceleration at vibration period T, and sa_{RT} is the RTGM intensity. Furthermore, assuming that the shaking intensity causing failure (i.e., the seismic fragility), quantified in terms of $Sa(T)$, follows a lognormal model with logarithmic standard deviation $\beta = 0.6$, the value of sa_{RT} per vibration period can be calculated by iterating Equation (1) until $\lambda_f \approx \lambda_f^*$:

$$\lambda_f = \int_0^{+\infty} \Phi\left[\frac{ln\left(Sa\right)-ln\left(sa_{RT}\right)}{\beta}+1.28\right]\cdot\left|d\lambda_{Sa}\right|, \tag{1}$$

where $\Phi(\cdot)$ denotes the standard Gaussian cumulative distribution function, λ_{Sa} is the hazard curve in terms of $Sa(T)$ at the construction site, and λ_f is the annual rate of earthquakes causing failure. This equation implies that the logarithm of median spectral acceleration causing failure must be $ln(sa_{RT})+1.28\cdot\beta$, for sa_{RT} to correspond to the 10% quantile value of the fragility.

An underlying assumption of the same equation is that a first mode-dominated structure, such as a moment-resisting mid-rise frame, will be designed under elastic base shear demand F_e calculated as:

$$F_e = m^* \cdot \Gamma \cdot \left[Sa_e\left(T_1\right)/q\right], \tag{2}$$

with T_1 being the first mode vibration period, $Sa_e\left(T_1\right)=(2/3)\cdot sa_{RT}$ is the elastic spectral acceleration demand, m^* the equivalent mass of the corresponding SDoF oscillator, Γ the first-mode participation factor and q a structure-dependent, so-called, *behavior factor*, that relates the elastic demand to the plastic deformation of the structure under the design shaking. The reduction of the RTGM demand by a factor of 2/3 follows the procedure established in FEMA P695,[16] even if it must be acknowledged that there are alternatives in the literature for linking the risk-targeted to the actual design-level elastic demand.[11,17] Although there are proposals to calibrate site- and structure-specific q-factors using risk-targeted seismic design criteria,[18,19] herein q is treated as a maximum allowable code-mandated value that accounts for both inelastic deformation capacity and overstrength associated with a particular structural typology (furthermore, it is treated as period-independent; this is because structures with relatively large periods are considered and, in any case, it does not represent a limitation).

To apply Equation (1), λ_{Sa} was calculated — for each site — for a set of periods $0s \le T \le 2s$ using the REASSESS software[20] to perform the probabilistic seismic hazard analysis (PSHA), based on a seismic source model for Italy[21] consistent with the one adopted by the building code currently in effect, for soil site class consistent with Eurocode 8 type C.[1] Additionally, PSHA was also performed to obtain hazard curves for a more advanced intensity measure, that is average

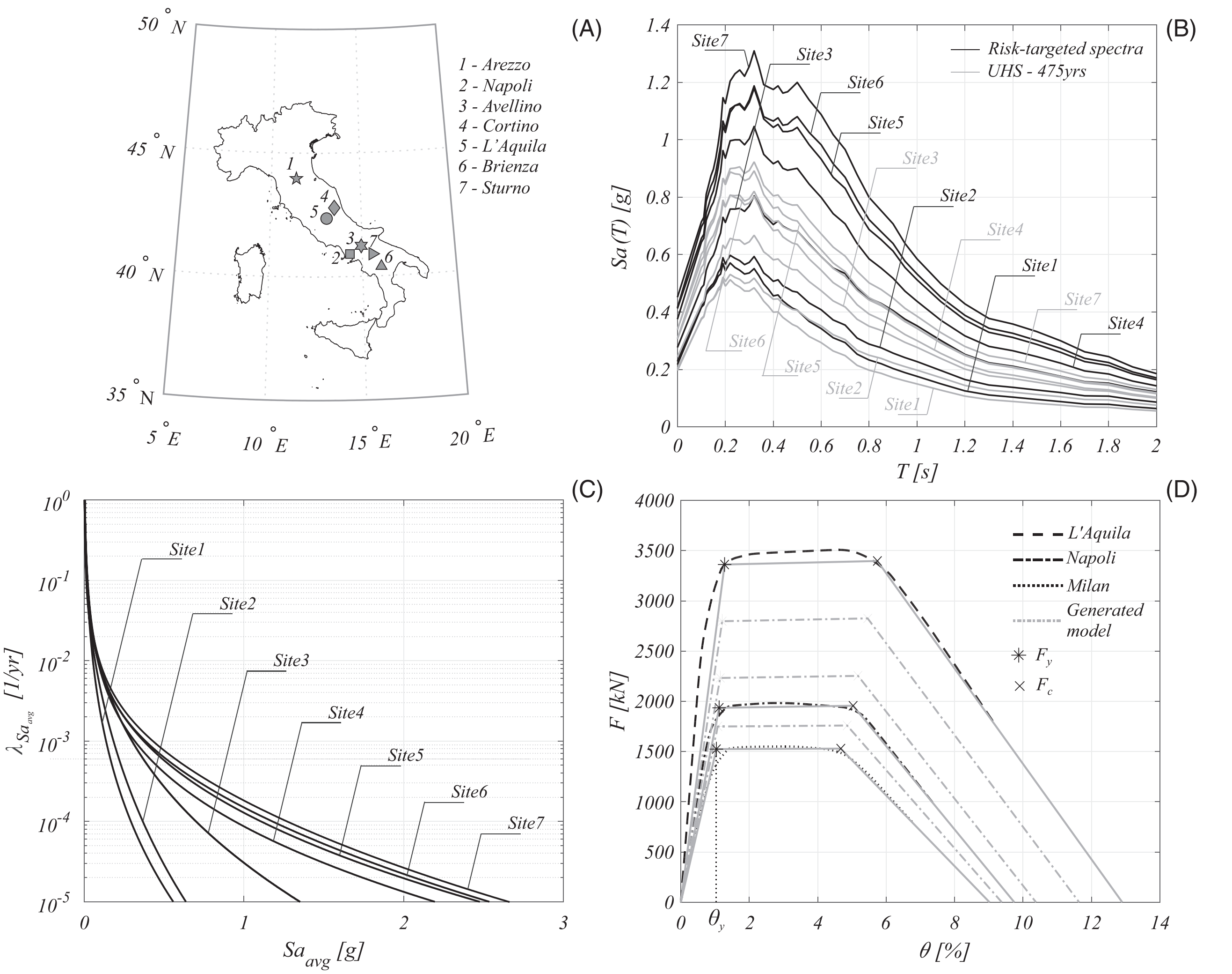

FIGURE 1 Map of Italy showing the seven case-study sites (A); risk-targeted and uniform-hazard design spectra calculated (B); average spectral acceleration hazard curves (C); reference static pushovers of the three-storey reinforced concrete buildings and generated backbones at intermediate base shears (D).

spectral acceleration Sa_{avg}, which is currently trendy in being preferred to $Sa(T)$ to estimate failure probabilities,[22–24] by virtue of being a special case of an intensity measure that accounts for spectral shape at periods beyond that of the first mode.[25] This intensity measure is given by:

$$Sa_{avg} = \left[\prod_{i=1}^{N} Sa\left(T_i\right)\right]^{1/N} \tag{3}$$

and in this case $N = 17$ spectral ordinates out of those contemplated in the ground motion prediction model of Ambraseys and co-authors[26] are used for its definition, more specifically $\{T_i\} = \{0.70, 0.75, 0.80, 0.85, 0.90, 0.95, 1.00, 1.10, 1.20, 1.30, 1.40, 1.50, 1.60, 1.70, 1.80, 1.90, 2.00\}$ s. For the calculation of the $\lambda_{Sa_{avg}}$ curves, a correlation model among spectral ordinates was also needed.[27] Figure 1B shows the calculated risk-targeted spectra, plotted together with uniform hazard spectra at the same sites corresponding to $\lambda_{Sa} = 0.0021$ (i.e., 475 year exceedance return period uniform hazard spectra), and the $\lambda_{Sa_{avg}}$ curves (Figure 1C).

Using the static pushover curves and metadata[28] of RC frames that were designed according to the Italian building code at sites with different hazard levels, in a previous work,[19] the authors were able to generate, via interpolation, families of surrogate inelastic oscillators with varying lateral resistance. This was done for each direction of frame alignment of a three- and a six-storey RC bare frame building, for a total of four structural configurations. These oscillators have

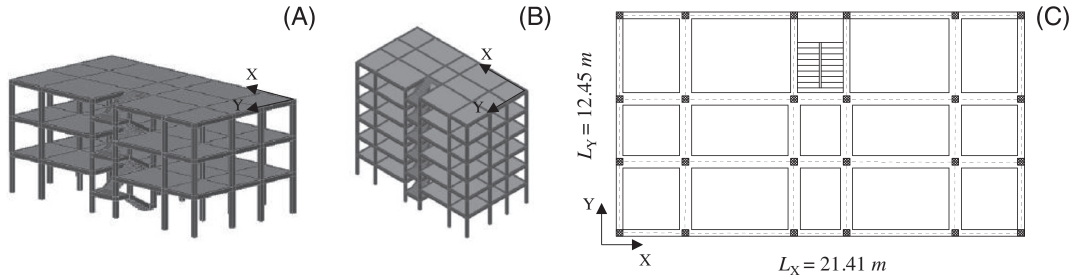

FIGURE 2 Configuration of the two case-study RC moment-resisting frame buildings: elevation of the three-storey (A) and six-storey (B) building and plan common to both (C). Main directions indicators X, Y also shown.

idealized tri-linear monotonic backbones that can be considered representative approximations of the pushover curves that would correspond to alternative Eurocode 8 compatible designs at different elastic base shear demands. For example, Figure 1D shows the pushover curves of the three-storey building designed at a high-, medium- and low-hazard site in Italy (L'Aquila, Naples and Milan, respectively) in terms of base shear F and roof drift θ, along with their piece-wise linear approximation that is amenable to interpolation at intermediate values. From the design of those structures, whose configuration in elevation and plan is shown in Figure 2, it was established that the third pushover from the low-hazard Milan site, constitutes a lower bound for lateral strength of the code-conforming building, since gravity-load design and minimum code requirements in terms of reinforcement detailing and cross-section dimensions govern the design of all load-bearing RC members.

3 | IMPLICATIONS OF LINEAR-ELASTIC DESIGN

At each of the seven sites and for each building configuration, that is both principal directions of the three- and six-storey RC frame (denoted for brevity as 3st-X, 3st-Y, 6st-X, 6st-Y) the elastic base shear demand F_e was established using the spectral ordinates of both the risk-targeted and uniform-hazard spectra shown in the figure above, assuming negligible contribution from higher modes, that is using Equation (2). The actual maximum (capping-point) strength $F_c = \alpha_u \cdot F_e$ was calculated via the overstrength coefficient α_u. For this calculation, it should be noted that that m^*, Γ, T_1 are known for the original reference designs, so they can also be assigned to the generated intermediate systems with some interpolation where necessary, and that $q = 3.9$ was assumed in all cases, consistently with the aforementioned (original) cases. The overstrength coefficient α_u was obtained from the static nonlinear analyses of the reference buildings, with more details available in the earlier work.[19]

Although it seems straightforward, it is worth dwelling on the fact that the fundamental period T_1 during design is obtained from a linear-elastic finite element model, where members' gross cross-section flexural stiffness is reduced according to simplified code-mandated rules, which generally leads to stiffer elements with respect to experimentally measured average secant stiffness of RC members at yield.[29] In other words, the condition $T_1 < T^*$ was consistently true for all cases of simulated design considered, where T^* is the period of the surrogate inelastic SDoF oscillator given by:

$$T^* = 2\pi \cdot \sqrt{m^* \cdot (\theta_y \cdot H)/F_y} \tag{4}$$

where H is building height, F_y is the base shear at nominal yield, corresponding to the formation of a plastic mechanism and θ_y the corresponding roof drift which ranges from 0.8% to 1% for the structures considered here.

The underlying assumption of the risk-targeted design spectrum is that $Sa(T)$ at the fundamental mode is used both as a measure of failure capacity and a means to determine the yield strength of an equivalent SDoF oscillator. Thus, the difference between T_1 and T^*, where the former is taken from modal analysis of the typical elastic model used for seismic design by practitioners and the latter from static pushover analysis of a nonlinear model, hints at two potential issues. First, the use of a linear-elastic analysis for force-based design, where material non-linearities at higher force demands are not modelled for the sake of simplicity, can be seen as tantamount to an additional source of overstrength, simply because $Sa_e(T_1) > Sa_e(T^*)$ over the examined period range. Second, the fragility of an oscillator with vibration period T^* is being expressed in terms of spectral acceleration at another, shorter period, which could exacerbate the problems already

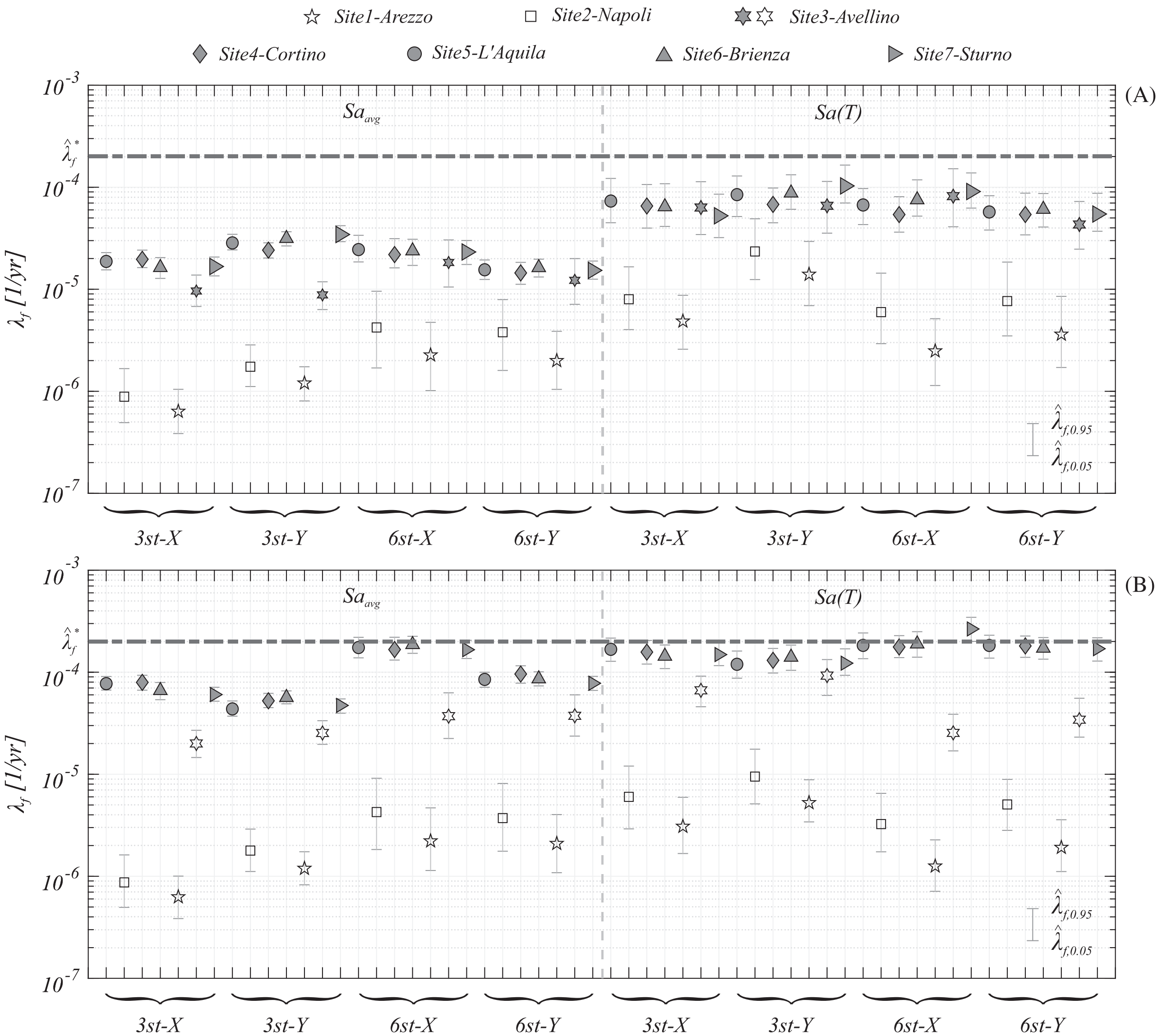

FIGURE 3 Annual failure rates for the four structural configurations at seven sites using two alternative intensity measures as hazard-fragility interfacing variables: structures with elastic base shear demand based on spectral ordinate at T_1 from modal analysis (A) and at T^* from the secant stiffness at nominal yield (B).

sometimes associated with $Sa(T)$ as an intensity measure for numerical prediction of dynamic instability, as a proxy for sidesway collapse mechanism.[22,23] To address these potential concerns, the following steps were taken. First, a set of alternative simulated designs was performed where the elastic base shear demand was calculated as $F_e = m^* \cdot \Gamma \cdot [Sa_e(T^*)/q]$, thus virtually eliminating the discrepancy between T_1 from modal analysis and T^*, which is a concept that has strong proponents in the displacement-based design camp.[30] Second, fragility was assessed for all surrogate SDoF systems via IDA, using the forty-four accelerograms of the FEMA-P695 far-field set[16] in conjunction with a dedicated interface[31] for OpenSees.[32] Failure was identified at the flat-line height of the IDA curves[33] and lognormal fragility models were fitted to the analysis results following the same procedure in previous related work of the authors.[19] For each oscillator, fragility was assessed in terms of three intensity measures: $Sa(T_1)$, $Sa(T^*)$ and Sa_{avg}, with the spectral ordinates always referring to T_1 and T^* of each specific structure, while Sa_{avg} was the same for all. Integrating each of these fragility functions with the already-available hazard curve at each site, provides the annual failure rate estimates shown in Figure 3, where the indication $Sa(T)$ implies that the hazard-fragility interfacing variable (intensity measure) coincides with the spectral ordinate used to calculate the design base shear.

For some of the cases considered, the maximum resisting base shear is not governed by the risk-targeted design spectra, as that is superseded by the intrinsic lateral strength implicit in gravity-load design and adhering to minimum reinforcement ratios, minimum cross-section dimensions and other detailing rules. In the figure, the corresponding failure rates are plotted with non-filled markers to enhance the visual distinction, which serves as confirmation of previous research results that uniform seismic reliability is a venture that may only concern higher hazard sites, at least when low-performance levels are considered, nearing dynamic instability as a prelude to sidesway collapse. In other words, any evaluation of how effective RTGM is towards homogenization of seismic reliability (to follow) ought to exclude these cases whose overstrength is insensitive to the design actions. This observation is perhaps less trivial than it may seem, because such low-hazard sites are often included when showcasing the regional disparity in seismic risk[3,12] that motivates the introduction of RTGM or other risk-targeted design approaches.[34,35]

A more conspicuous observation is that, among the structures for which minimum requirements do not override seismic design loads, only the right-hand group in panel B achieves reliability levels consistently near the target λ_f^*. In fact, this is the only case where $Sa(T^*)$ is both the spectral ordinate driving design and the intensity measure for fragility and hazard. In other words, where two of the premises behind the risk-targeted design spectrum's derivation are both met. On the other hand, in the case of panel A, where the design spectral ordinate is determined by linear-elastic modal analysis, rather than the secant stiffness from static nonlinear analysis, risk assessment using both intensity measures leads to failure rates between 10^{-4} and 10^{-5}. These are lower than the target $2 \cdot 10^{-4}$, at times by an order of magnitude, suggesting that the simplification of using elastic analysis with conventional member stiffness reduction in conjunction with a risk-targeted design approach can become a source of additional overstrength, at least in the range of periods 0.45 to 2.0 s examined here. This is corroborated by the difference in reliability estimates between the two panels using Sa_{avg}, which is not affected by the presumably lower explanatory power of $Sa(T)$ with respect to seismic response.

Finally, when looking exclusively at cases not affected by design requirements other than the RTGM (filled markers), one can observe an alignment of failure rates when fragility is expressed in terms of the $Sa(T^*)$ spectral ordinate, hinting at the uniformity of seismic reliability that is the premise behind RTGM. However, this becomes less clear-cut when the fragility is expressed in terms of the other ground motion intensity measures, especially Sa_{avg}. Naturally, such an observation can only be made after recalling that these comparisons are between point estimates of the various failure rates, which are derived from finite samples of structural responses and are therefore affected by estimation uncertainty.[36] For this reason, the graph is also equipped with vertical bars indicating the fifth and ninety-fifth quantile intervals of the estimators, indicated as $\hat{\lambda}_{f,0.05}$ and $\hat{\lambda}_{f,0.95}$, respectively, calculated via a *parametric bootstrap* procedure.[37] This additional information shows that reliability across the sites can appear homogeneous also under $Sa(T_1)$, but that is not the case for the more efficient Sa_{avg}. This observation implies that the uniformity of seismic risk, which is the goal of RTGMs, may be partially hindered by the fact that it is being enforced via a fragility expressed in an arbitrary intensity measure, at least at low-performance levels, although the importance of this issue remains to be quantified.

4 | RISK-TARGETED VS UNIFORM-HAZARD DESIGN

From the presented results it seems as though, despite some caveats, risk-targeted design spectra provide relatively uniform seismic reliability for a specific structure designed across sites with differing hazard, as long as said hazard does not drop below a threshold that renders the lateral resistance insensitive to the level of seismic actions. This can be appreciated by looking at the map of Figure 4A, which divides Italy in three zones. The dark shading, featuring more than twenty percent (22%) of the total area, represents the part where the lateral strength attributed to any of these four structures designed on soil type C, using the RTGM described above, would be determined by the seismic actions. The lighter shading signifies that, in those areas, the risk-targeted design scenario would lead to lateral strength governed by code detailing requirements for all four structures and accounts for more than sixty percent (62%) of the Italian territory. The remaining area (16%), with intermediate shading, corresponds to sites where some of the four structures fall in the first and some in the second category. This goes to show that, even for a notoriously earthquake-prone country such as Italy, only less than half of the territory (22%–38%) would see a shift in seismic reliability by adopting a RTGM approach, at least for the specific structures used in this example and with the set reliability goals.

Momentarily excluding these low-hazard sites from the discussion, the structure-specific risk homogeneity cannot be said to persist within the confines of a structural typology, in the sense that some variability of λ_f can be observed between the case-study structures, which are all low- to mid-rise moment-resisting RC frames with the same planar configuration, their main differences being in actual number of storeys and bays. However, lacking an objective yardstick for how close

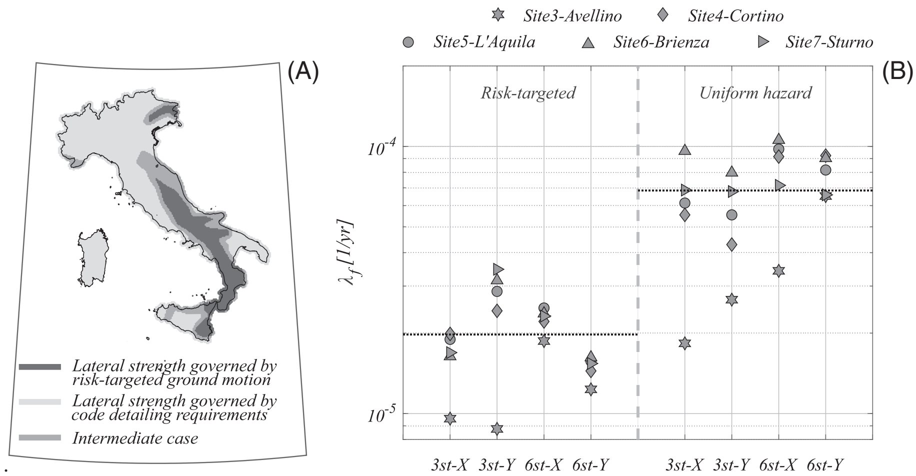

FIGURE 4 Map dividing Italy into tree zones, according to whether the risk-targeted design ground motion would govern lateral strength of the case-study structures (A). Mean annual frequencies of failure for simulated designs across five sites, using risk-targeted and uniform hazard spectra (B).

together those reliability estimates must congregate to dub risk-targeted design a success, a possibility is to compare with the main alternative, that is design against uniform-hazard seismic actions. This comparison is made in Figure 4B, where the juxtaposition is limited to the use of Sa_{avg} as intensity measure for five sites, for which it has been ascertained that strength allocation is not insensitive to the seismic actions, be that risk-targeted or uniform-hazard. In this comparison, only the case where T_1 from modal analysis is used to calculate design base shear is used, as it is considered a benchmark with greater vicinity to actual practice.

It is worth pointing out that, for this comparison, both the risk-targeted and uniform hazard spectra have shapes directly derived from the calculations, courtesy of the ground motion model(s) therein, without any smoothing adjustments for cosmetic or practical purposes. This was a conscious choice, since other works have shown that the way such spectral shape modifications are implemented has far from negligible effects[13] which are not examined here. For all three sites, the assumptions made about the target reliability against failure, have produced higher risk-targeted spectral ordinates than the corresponding 475 years return period uniform hazard spectra, across all vibration periods (T). This is reflected in the fact that the estimated annual failure rates are generally lower for the risk-targeted design cases, but it does not constitute per se a useful point for comparison, since the reliability objective that drives their relationship is, to a certain extent, arbitrary.

At first glance, it appears that for each structure, the site-to-site variability of the reliability metric λ_f is less in the risk-targeted design case with respect to the uniform hazard one. However, the structure-to-structure variability is less obvious to the naked eye. For this reason, the arithmetic means of all twenty failure rates were obtained per design strategy, indicated in the figure by the horizontal dotted dark line, together with the empirical coefficient of variation (CoV) about said means. The CoV for the risk-targeted design cases was found equal to 0.35, while the one for the uniform hazard ones resulted only somewhat larger at 0.36. Recalling that, in the previous section, the failure rates for the same cases appeared more tightly knit together under the lens of fragility in first-mode spectral ordinates, this result may allow to quantify the intensity measure effect indirectly: it is enough to render the improvement in the disparity of structure-specific reliability across sites with different seismic hazard, when switching from UHS- to RTGM-based design, non-obvious or at least quite limited.

5 | DISCUSSION

This brief note presented a limited number of case-study examples, where the extensive results of previous research projects were exploited to simulate the design of RC frames under risk-targeted seismic actions. In its simplicity, force-based seismic design using linear-elastic analysis allocates strength demand to load-bearing members in proportion to the design actions, while detailing rules guarantee an adequate degree of inelastic deformation capacity. This gives rise to the concept that the design actions can be modulated in accordance with both site-specific hazard and structural fragility, in order to avoid the site-to-site discrepancies in seismic reliability that are observed when designing against elastic seismic demand with a predetermined exceedance frequency. Despite the intuitive appeal of a proposal to achieve more homogeneous seismic reliability, the examples presented here showcase some of its well- and less-known issues.

At lower-hazard sites, member strength demand from seismic loads can be superseded by demand from gravity-load design, drift limitations[38] or simply detailing requirements such as minimum reinforcement ratios. This renders the seismic reliability against collapse insensitive to the elastic seismic demand provided by the design spectrum for such regions. For the Italian RC buildings and reliability target used here as examples, these regions can be more than sixty percent of the country's total area, so this limitation can be far from negligible.

The secant stiffness of RC structures at nominal yield is only considered approximately in linear-elastic analysis that forms the basis for design. Given that the approximation is generally towards stiffer calculation models, that in itself can become an additional source of overstrength. Additionally, the corresponding spectral ordinate is less of a robust measure for structural fragility, as shown by the different estimates when better-performing intensity measures are employed. The examples presented here showed that the combination of these two effects can lead to overshooting the reliability objective failure rate by up to an order of magnitude.

Finally, when the offending risk variability is examined only in the narrower domain of higher-hazard sites, the margins for improvement with respect to uniform-hazard design are smaller than they appear when low-hazard site reliability is also included. For these comparisons, the uncertainty which affects the estimation of the failure rate because of record-to-record variability of structural response, should not be neglected. For the cases examined here, involving four similar structural configurations spread across five sites, the coefficient of variation in failure rate remains practically unchanged when employing RTGMs.

Given that this study willingly left out certain aspects, such as the tendency of codes to adopt conventional spectral shapes, higher-mode-influenced structures, or even drift-governed lateral-load systems, it seems that, although RTGMs can represent a useful tool in harmonizing seismic risk, some issues remain, and addressing them may be relevant for the advancement of seismic design of structures.

DATA AVAILABILITY STATEMENT

All data used or produced in this study are available upon request.

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REFERENCES

1. CEN. EN 1998-1 - Eurocode 8: Design Provisions for Earthquake Resistance of Structures, Part 1: General Rules, Seismic Actions and Rules for Buildings; 2004.
2. Consiglio Superiore dei Lavori Pubblici. Norme tecniche per le costruzioni. *Gazzetta Ufficiale della Repubblica Italiana*. 2018;42.
3. Iervolino I, Spillatura A, Bazzurro P. Seismic reliability of code-conforming Italian buildings. *J Earthq Eng*. 2018;22:5-27. doi:10.1080/13632469.2018.1540372
4. Costa A, Romão X, Oliveira CS. A methodology for the probabilistic assessment of behaviour factors. *Bull Earthq Eng*. 2010;8(1):47-64. doi:10.1007/s10518-009-9126-5
5. Žižmond J, Dolšek M. Formulation of risk-targeted seismic action for the force-based seismic design of structures. *Earthq Eng Struct Dyn*. 2019;48(12):1406-1428. doi:10.1002/eqe.3206
6. McGuire RK. Probabilistic seismic hazard analysis and design earthquakes: closing the loop. *BSSA*. 1995;33(5):1275-1284. doi:10.1016/0148-9062(96)83355-9
7. Iervolino I, Giorgio M, Cito P. The peak over the design threshold in strong earthquakes. *Bull Earthq Eng*. 2019;17(3):1145-1161. doi:10.1007/s10518-018-0503-9
8. Ricci P, Domenico MD, Verderame GM. Parameters affecting the behaviour factor and the seismic safety of EC8-designed reinforced concrete buildings. *COMPDYN Proceedings*. 2019;1(June):514-528.
9. Žižmond J, Dolšek M. Evaluation of factors influencing the earthquake-resistant design of reinforced concrete frames according to Eurocode 8. *Struct Infrastruct Eng*. 2016;12(10):1323-1341. doi:10.1080/15732479.2015.1117112
10. Iervolino I. Risk-targeted seismic design: prospects, applications, and open issues, for the next generation of building codes. *Earthq Eng Struct Dyn*. 2023;52(13):3919-3921. doi:10.1002/eqe.3999
11. Silva V, Crowley H, Bazzurro P. Exploring risk-targeted hazard maps for Europe. *Earthq Spectra*. 2016;32(2):1165-1186. doi:10.1193/112514eqs198m

12. Douglas J, Ulrich T, Negulescu C. Risk-targeted seismic design maps for mainland France. *Natural Hazards*. 2013;65(3):1999-2013. doi:10.1007/s11069-012-0460-6
13. Spillatura A, Vamvatsikos D, Kohrangi M, Bazzurro P. Harmonizing Seismic Performance via Risk Targeted Spectra: state of the art, dependencies, and implementation proposals. *Earthq Eng Struct Dyn*. 2023;52:4277-4299. doi:10.1002/eqe.3941
14. Luco N, Hamburger RO, Klemencic M, Seattle A, Jeffrey W, Kimball K, Risk-targeted versus current seismic design maps for the conterminous United States. In: SEAOC Convention Proceedings. ASCE; 2007.
15. Vamvatsikos D, Cornell CA. Incremental dynamic analysis. *Earthq Eng Struct Dyn*. 2002;31(3):491-514. doi:10.1002/eqe.141
16. Federal Emergency Management Agency (FEMA). P695 Quantification of Building Seismic Performance Factors; 2008.
17. Gkimprixis A, Tubaldi E, Douglas J. Comparison of methods to develop risk-targeted seismic design maps. *Bull Earthq Eng*. 2019;17(7):3727-3752. doi:10.1007/s10518-019-00629-w
18. Macedo L, Silva A, Castro JM, A more rational selection of the behaviour factor for seismic design according to Eurocode 8. *Eng Struct*. 2019;188:69-86. doi:10.1016/j.engstruct.2019.03.007
19. Baltzopoulos G, Grella A, Iervolino I. Seismic reliability implied by behavior-factor-based design. *Earthq Eng Struct Dyn*. 2021;50(15):4076-4096. doi:10.1002/eqe.3546
20. Chioccarelli E, Cito P, Iervolino I, Giorgio M. REASSESS V2.0: software for single- and multi-site probabilistic seismic hazard analysis. *Bull Earthq Eng*. 2019;17(4):1769-1793. doi:10.1007/s10518-018-00531-x
21. Meletti C, Galadini F, Valensise G, et al. A seismic source zone model for the seismic hazard assessment of the Italian territory. *Tectonophysics*. 2008;450(1-4):85-108. doi:10.1016/j.tecto.2008.01.003
22. Eads L, Miranda E, Lignos DG. Average spectral acceleration as an intensity measure for collapse risk assessment. *Earthq Eng Struct Dyn*. 2015;44(12):2057-2073. doi:10.1002/eqe.2575
23. Kazantzi AK, Vamvatsikos D. Intensity measure selection for vulnerability studies of building classes. *Earthq Eng Struct Dyn*. 2015;44(15):2677-2694. doi:10.1002/eqe.2603
24. Baltzopoulos G, Baraschino R, Iervolino I. On the number of records for structural risk estimation in PBEE. *Earthq Eng Struct Dyn*. 2019;48(5):489-506. doi:10.1002/eqe.3145
25. Bojórquez E, Iervolino I. Spectral shape proxies and nonlinear structural response. *Soil Dyn Earthq Eng*. 2011;31(7):996-1008. doi:10.1016/j.soildyn.2011.03.006
26. Ambraseys N, Simpson K, Bommer J. Prediction of horizontal response spectra in Europe. *Earthq Eng Struct Dyn*. 1996;25:371-400.
27. Jayaram N, Baker JW. Statistical tests of the joint distribution of spectral acceleration values. *Bull Seismol Soc Am*. 2008;98(5):2231-2243. doi:10.1785/0120070208
28. Ricci P, Manfredi V, Noto F, et al. Modeling and seismic response analysis of italian code-conforming reinforced concrete buildings. *J Earthq Eng*;22:105-139. doi:10.1080/13632469.2018.1527733
29. Panagiotakos TB, Fardis MN. Deformation of reinforced concrete at yielding and untimate. *ACI Struct J*. 2001;98(2):135-147. doi:10.14359/10181
30. Priestley MJN, Calvi GM, Kowalsky MJ. *Displacement-Based Seismic Design of Structures*. 2nd ed. IUSS Press; 2007.
31. Baltzopoulos G, Baraschino R, Iervolino I, Vamvatsikos D. Dynamic analysis of single-degree-of-freedom systems (DYANAS): a graphical user interface for OpenSees. *Eng Struct*. 2018;177:395-408. doi:10.1016/j.engstruct.2018.09.078
32. McKenna F. OpenSees: a framework for earthquake engineering simulation. *Comput Sci Eng*. 2011;13(4):58-66. doi:10.1109/MCSE.2011.66
33. Vamvatsikos D, Cornell CA. Applied incremental dynamic analysis. *Earthq Spectra*. 2004;20(2):523-553. doi:10.1193/1.1737737
34. Vamvatsikos D, Kazantzi AK, Aschheim MA. Performance-based seismic design: avant-garde and code-compatible approaches. *ASCE ASME J Risk Uncertain Eng Syst A Civ Eng*. 2016;2(2). doi:10.1061/AJRUA6.0000853
35. Krawinkler H, Zareian F, Medina RA, Ibarra LF. Decision support for conceptual performance-based design. *Earthq Eng Struct Dyn*. 2006;35(1):115-133. doi:10.1002/eqe.536
36. Iervolino I. Assessing uncertainty in estimation of seismic response for PBEE. *Earthq Eng Struct Dyn*. 2017;46(10):1711-1723. doi:10.1002/eqe.2883
37. Baraschino R, Baltzopoulos G, Iervolino I. R2R-EU: software for fragility fitting and evaluation of estimation uncertainty in seismic risk analysis. *Soil Dyn Earthq Eng*. 2020;132(November 2019):106093. doi:10.1016/j.soildyn.2020.106093
38. Macedo L, Castro JM. Collapse performance assessment of steel moment frames designed to Eurocode 8. *Eng Fail Anal*. 2021;126:105445. doi:10.1016/j.engfailanal.2021.105445

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