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Design Code Progress and Conventional Seismic Safety Improvement in Italy

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ABSTRACT

In Italy it is acknowledged that the modern era of seismic design started after the 1908 Messina earthquake. Ever since, design codes and design hazards have had a relatively continuous evolution, following the international development of earthquake engineering, a process that is continuing nowadays. On the other hand, most of the existing building stock, mainly composed of reinforced concrete or unreinforced masonry buildings, is much more slowly renovated, so that these structures are designed with now obsolete or without any seismic provision. Considering that codes, even those at the state-of-the art, do not explicitly control the reliability implied by design, the safety achieved by the progress in seismic design has not been quantified systematically. This is the scope of the study presented in this technical note that, resourcing from the results of a large research project, quantified the fatality rates, computed according to three different methods, following the changes in building codes in Italy. Although highly conventional and mainly useful for relative comparisons, the results show the systematical improvement of safety achieved by evolving seismic design, with the largest increment attributable to the current code, which is based on the same principles as Eurocode 8.

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1. Introduction

The seismic history of Italy is filled with damaging earthquakes that led, since the beginning of the XX century, to the continuous evolution of regulatory seismic design codes. The landmarking event was the 1908 Messina earthquake, with magnitude, $M, 7.1 \pm 0.18$ (Rovida et al. 2020), which killed, in conjunction with a tsunami, some tens thousands of people, according to estimates. The Italian building code released in 1909 contained provisions about building height to be enforced in the areas hit by the earthquake. (In fact, the seismic classification of the territory, a prodromic design hazard assessment, has been based – for a large part of the last century – on requiring seismic design only in the areas where recent earthquakes occurred.) The introduction of a design base shear was after the 1915 Avezzano earthquake ($M\ 7.08 \pm 0.08$). Another major step was the 1975 introduction of the response spectrum concept, and the related dynamic analysis as an alternative to the static one. In 1996, the limit state design and some indications to improve local and global ductility were introduced. A paradigm change occurred in 2003, after the San Giuliano di Puglia earthquake ($M\ 5.7 \pm 0.07$), which caused the partial collapse of a school killing twenty-seven children and their teacher, causing major concern. The seismic code then introduced followed the Eurocode 8 design approach (CEN 2004). For the first time, the design hazard for the whole country, except the Sardinia Island, was based on the *peak ground acceleration* (PGA), on stiff soil, with an exceedance return period depending on the design performance, evaluated by means of probabilistic seismic hazard analysis or PSHA (Stucchi

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et al. 2011). The code was explicitly referring to capacity design principles and the concept of *behavior factor*. Nevertheless, the 1996 code was still applicable and it required the 2009 L'Aquila earthquake (M 6.29 ± 0.07) for the enforcement of sole performance-based design criteria. This code, hereafter referred to as NTC, had a revision in 2018, yet without changing its main principles, which is the one currently enforced. Many other steps, some also not necessarily representing a progress, characterized the seismic design evolution in Italy. The interested reader can find details, and references to the changing codes, in Petruzzelli and Iervolino (2021).

The Italian building stock is mostly made of unreinforced masonry and reinforced concrete buildings, with the latter mostly built in the post-World-War-II and the former being mostly older than that. Therefore, only a minority of the building heritage reflects current seismic design technology. This, in conjunction with a relatively high seismic hazard throughout the country, let Italy be considered one of the most exposed nations to seismic risk in Europe. On the other hand, none of the enforced codes, including the current one, generally enabled the designer to explicitly control the seismic reliability implied by design. This is why the Italian civil protection department of the government funded the research program *Rischio Implicito – Norme Tecniche per le Costruzioni Esistenti* or RINTC-e (Iervolino 2024) that is, precisely to evaluate the seismic reliability of *low-code* or *pre-code* structures.

This project followed one, named hereafter RINTC-n (Iervolino and Dolce 2018), in which the design was based on the current code, and the results of which enabled to compute, although in a highly conventional manner, the seismic fatality rates implied by state-state-of-art seismic design, as a measure of achieved seismic safety (Iervolino and Pacifico 2021). Among other results, it was found that, for current-code-conforming structures, even if the design ground motion intensity has the same return period of exceedance at all sites in Italy, the resulting fatality rates tend to increase when the hazard of the construction site also increases. This is possibly due to: (i) the oversizing effects of code-prescribed minima (including gravity-load design in low-to-moderate hazard regions), and (ii) the ground motion intensity with return periods larger than that of design, which tends to disproportionately increase at high-hazard sites with respect to mid- and low-hazard sites (Cito and Iervolino 2020).

The objective of this note is to measure the eventual improvement of seismic structural safety due to changes in design codes in Italy, via the results of the RINTC-e project. To this aim, the remainder of the manuscript is structured such that the project and its results are briefly described first. Then, the conventional methodologies to compute the fatality rates are illustrated. Finally, the rates are mapped with respect to the design technology progress and compared with those referring to current design. Some final remarks close the paper.

2. Assessing the Seismic Reliability of Current-Code- and Old-Code-Conforming Italian Buildings

RINTC projects considered five structural typologies: unreinforced masonry buildings, URM; reinforced concrete buildings, RC; precast reinforced concrete buildings, PRC; steel buildings, S; and base-isolated reinforced concrete buildings, BI (not considered herein), as described in the following. Because the RINTC-n structures were already described elsewhere (Iervolino and Pacifico 2021), the following refers to the RINTC-e project, being the core of this study. The analyzed structures are supposedly located in five Italian sites: L'Aquila (AQ), Catania (CT), Naples (NA), Rome (RM), and Milan (MI), generally in decreasing order of hazard as evaluated currently as well as in previous assessments, since the last century. The evolution of design codes proceeded asynchronously for the different typologies in Italy, as briefly recalled hereafter, while the rationale on the definition of design epochs can be found in the references given in this section.

Four design epochs were identified for URM residential buildings: Pre-'20, '20-'45, '45-'87, Post-'87. In fact, URM buildings considered in the project were real buildings that can be further categorized as historical (e.g., stone or brick masonry for walls and vaults or timber floors) and modern (e.g., with artificial block masonry for walls and reinforced concrete floors) with three, four, five or seven stories. For some of these structures upgrading to and old code (put in the '45-'87 bin) or NTC was also simulated (Lagomarsino et al. 2023; Penna et al. 2024).

For RC residential buildings, three epochs were considered: Pre-'70, '70s, '80-'90. RC structures are three or six story frames analyzed in three configurations: infilled frame, *pilotis* frame and bare frame. The design according to the old codes was simulated for all the sites considered. The RC structures can be further categorized as gravity-load design and seismic design, given that seismic design was not mandatory at all sites through the design epochs (De Risi et al. 2023; Di Domenico et al. 2023).

PRC structures are single story industrial buildings four of which designed at the Milan site without accounting for seismic loads and referring to two relevant epochs: '70s and '80-'90s. These structures are characterized by different roof and cladding (precast elements with or without additional RC topping and infilled frames or precast cladding panels, respectively) and connection details (friction or dowel). Two of these buildings were also relocated in Naples, while the others were re-designed ad-hoc in Naples. Finally, the four buildings were re-designed in L'Aquila. Moreover, one of those was also re-designed referring to Pre-'70 epoch (Bosio et al. 2023).

S single story industrial buildings were designed according to the '80-'90 epoch in L'Aquila, Naples and Milan (the results for this design epoch can be extended to buildings designed since the '60s, to follow). The structural scheme is a frame (hinged or fixed) in the transverse direction frames, while some portals have a bracing system (L-shaped or square hollow section) in the longitudinal direction. Three different configurations were considered: bare frame, trapezoidal sheeting cladding, and sandwich panels sheeting (Cantisani and Della Corte 2023). Table 1 summarizes the configurations at the sites (low-, mid-, and high-hazard) considered herein, that is, MI, NA AQ.

The seismic reliability assessment considered structural failure with respect to two performance levels, that is *usability preventing damage* (UPD) and *global collapse* (GC). The structural and non-structural metrics corresponding to these performance levels were defined ad-hoc to capture the global building response and are not those conventional of the building codes used for design. In the RINCT-n, UPD former is mainly related to the seismic damage experienced by non-structural elements, such as infillings reinforced concrete frames, while the GC is generally related to the horizontal displacement capacity corresponding to the fifty-percent drop of the base shear, determined by means of non-linear static analysis. In the RINTC-e project the same definitions apply for the performances, yet more typology- and design-epoch-specific adjustments were needed to capture the failure modes of low- and pre-code buildings, such as brittle failures in RC frames. Details are in the referenced papers.

The seismic reliability metric adopted by the RITNC project is the failure rate (λ_f) or the mean number of earthquakes (i.e. mainshocks of seismic sequences) that, in one year, cause failure:

Table 1. Summary of the analyzed structural configurations (configs.) per site, structural typology and design epochs.

Typology	Epoch	AQ	NA	MI
URM	Pre-'20	2 configs.	1 configs.	–
	'20-'45	3 configs.	–	1 config.
	'45-'87	2 configs.	2 configs.	1 config.
	Post-'87	–	–	–
	Upgraded to NTC	2 configs.	2 configs.	–
RC	Pre-'70	6 configs.	6 configs.	6 configs.
	'70s	6 configs.	6 configs.	6 configs.
	'80-'90	6 configs.	6 configs.	6 configs.
PRC	Pre-'70	1 config.	–	–
	'70s	2 configs.	2 configs.	2 configs.
	'80-'90	2 configs.	2 configs.	2 configs.
S	Pre-'70	–	–	–
	'70s	–	–	–
	'80-'90	12 configs.	4 configs.	4 configs.

$$\lambda_f = \nu \cdot P[F|E] = \int_{IM} P[F|IM = x] \cdot \left| \frac{d\lambda_{im}(x)}{dx} \right| \cdot dx. \tag{1}$$

In the equation, ν is the rate of earthquakes (a *mainshock*, in fact) above a minimum magnitude of interest, while $P[F|E]$ is the probability of failure given the occurrence of an earthquake above the minimum magnitude. According to the *performance-based earthquake engineering* framework (Cornell and Krawinkler 2000), $P[F|E]$ can be evaluated as the integral of the failure probability conditional to the value of a ground motion intensity measure (IM), $P[F|IM = x]$, that is the fragility curve of the structure, and the absolute value of derivative of the hazard curve, $|d\lambda_{im}(x)/dx|$ for the site times dx (McGuire 2004).

To assess the seismic structural fragility *multi-stripe* non-linear dynamic *analysis* (MSA) (Jalayer and Cornell 2009) of three-dimensional numerical models of all configurations, was carried out. Ten IM stripes, corresponding to ten exceedance return periods at the site (between 10^1 and 10^5 years), were considered. For each model, the chosen IM is the spectral pseudo-acceleration (with five percent viscous damping) at a period close to the first vibration mode. Real ground motion records, as the analysis input, were selected according to a hazard consistent procedure (Spillatura et al. 2021). Finally, for each structure a lognormal fragility function was calibrated on the MSA results. The hazard curves for the considered IM at the sites were computed based on PSHA considering an authoritative seismic source model for Italy. Details on both hazard and fragility are given in Iervolino et al. (2023).

3. Fatality Rates Evaluation

The fatality rate, λ_d, can be defined as the expected number of earthquakes in one year causing death due to building failure, either structural or non-structural. It can be evaluated as the fatality probability given the occurrence of a seismic event, $P[D|E]$, times the expected number of earthquakes in one year. In turn this probability can be computed as the sum, over all possible damage states of the structure, of the products between probability that the event determines damage $DS_i, i = 1, 2, \ldots, n$, and the fatality probability given DS_i:

$$\lambda_d = \nu \cdot P[D|E] = \nu \cdot \sum_{i=1}^{n} P[D|DS_i] \cdot P[DS_i|E] = \mathrm{P}[D|UPD] \cdot \left(\lambda_{f,UPD} - \lambda_{f,GC}\right) + P[D|GC] \cdot \lambda_{f,GC}. \tag{2}$$

Fatality probabilities are, in principle, associated with any structural performance or damage state. This is because even the lowest damage levels, excluding structural damage, contemplate non-structural components performance, the damage of which can cause injuries and fatalities. The fatality rates computed herein, similar to Iervolino and Pacifico (2021), follow a conventional approach considering that only two failure rates are available from the RINTC project (Iervolino, Baraschino, and Spillatura 2023; Iervolino, Spillatura, and Bazzurro 2018), for the GC and UPD performances only, indicated as $\lambda_{f,GC}$ and $\lambda_{f,UPD}$, respectively. This yields to the right-hand side of Eq. (2), where $P[D|GC]$ and $P[D|UPD]$ are the fatality probabilities according to three different methods, two of which were also applied in Iervolino and Pacifico (2021).

3.1. Method 1

In this method, stemming from Tsang and Wenzel (2016), the fatality probabilities are taken equal to the fatality rates given structural damage obtained from HAZUS (FEMA 2004). $P[D|GC]$ and $P[D|UPD]$ are computed considering GC and UPD corresponding to *complete structural damage* and *moderate structural damage* in HAZUS, respectively. Because the fatality probabilities depend on the structural typology, each RINTC-e structure was associated with a corresponding HAZUS building class. Table 2 specifies, for each building, the assigned HAZUS class, description, and the corresponding conditional fatality probabilities $P[D|UPD]$ and $P[D|GC]$.

3.2. Method 2

This method uses the work of Zuccaro and Cacace (2011), where the *European Macroseismic Scale* (ESM) (Grünthal 1998) is used to associate the vulnerability classes (A to D, from the most vulnerable to least one) to the casualty probability. Herein, UPD and GC are considered to correspond to DS_3 and DS_5 of EMS, respectively. This is because DS_5 is global collapse, while DS_3 includes partial failures of non-structural components and the building becomes temporarily unusable. Because ESM deals with residential buildings only, this approach was not applied to S and PRC RINTC-e buildings. Table 2 also summarizes the adopted associations and reports the corresponding conditional fatality probabilities $P[D|UPD]$ and $P[D|GC]$ for this method.

3.3. Method 3

The third method, not considered in Iervolino and Pacifico (2021), relies on fib80 bulletin (FIB 2016), which provides causality probabilities given structural failure for different consequence classes (CC) as defined in the so-called Eurocode 0 (CEN 2005). RINTC-e buildings were associated with CC2, that is, consequence class *medium*. The approach does not distinguish between different damage states, yet assigns a unique fatality probability per CC. Therefore, for the sake of compatibility with the existing framework, this is assumed to be associated only with GC, so that the fatality rate is computed as:

$$\lambda_d = P[D|GC] \cdot \lambda_{f,GC}. \tag{3}$$

The associations between the RINTC-e structural configurations and fib80 probabilities are also given in Table 2. Comparing the probabilities, it appears that HAZUS and fib80 provide somewhat comparable $P[D|GC]$ across all cases, whereas those from the ESM-based method tend to be larger. The $P[D|UPD]$ values from HAZUS correspond to zero in method 2, while as mentioned, fib80 only gives fatality probabilities for collapse. The differences between the probabilities among the three methods may be due to the specific they have in their specific contexts, the way they are derived there, and how they are used to compute the fatality risk herein.

4. Results

The resulting fatality rates given in this section are interpreted as annual fatality probabilities for an individual continuously exposed to seismic risk in a building of a given typology at a given site; i.e., the *local personal risk*. They are also presented and compared to those of current-code-conforming structures, where applicable. It is also to preliminarily note that, in the RINTC projects 10^{-5} has been set as lowest possible value of the failure rate, to avoid large extrapolation of hazard and fragility models.

Figure 1 shows the arithmetic average (markers) of the rates computed for all the configurations at all sites (AQ, NA, MI), as a function of the design epoch, while the vertical bars represent the range of rates contributing to the average. The red markers and related bars represent results for the current code (Iervolino and Pacifico 2021). The general trending improvement of structural safety is evident, with a few exceptions. For PRC industrial buildings, different plan configurations and design choices in the '80-'90 structures make the trend slightly less clear (Bosio et al. 2023); however, in all cases the average fatality rates are around 10^{-5} or lower. As it pertains to the conventional methods adopted: it appears that, for RC and URM residential buildings, method 3 and method 1 provide lower and larger rates (up to about one order of magnitude), respectively.

Table 2. Categorization of RINTC-e buildings and performance according to the three methods considered.

	HAZUS				ESM				Fib 80		
RINTC-e buildings	Class	Description	$P[D\|UPD]$	$P[D\|GC]$	Vuln. class	Description	$P[D\|UPD]$	$P[D\|GC]$	Cons. class	Cons. for loss of human life	$P[D\|GC]$
URM two-story buildings (Massive stone)	URML	Low-Rise Unreinforced Masonry Bearing Walls	1.86E-05	2.16E-02	B	Masonry Massive stone	0.00E+00	9.75E-02	CC2	Medium	5.00E-02
URM two-story buildings (Manufactured stone units)	URML	Low-Rise Unreinforced Masonry Bearing Walls	1.86E-05	2.16E-02	C	Masonry Unreinforced with manufactured stone units	0.00E+00	9.75E-02	CC2	Medium	5.00E-02
URM with three- (or more) story buildings. (Massive stone)	URMM	Mid-Rise Unreinforced Masonry Bearing Walls	1.86E-05	2.16E-02	B	Masonry Massive stone	0.00E+00	9.75E-02	CC2	Medium	5.00E-02
URM with three- (or more) story buildings. (Manufactured stone units)	URMM	Mid-Rise Unreinforced Masonry Bearing Walls	1.86E-05	2.16E-02	C	Masonry Unreinforced with manufactured stone units	0.00E+00	9.75E-02	CC2	Medium	5.00E-02
RC-G three-story BF buildings	C1L	Low-Rise Concrete Moment Frame	0.00E+00	1.79E-02	C	RC frame without earthquake-resistant design (ERD)	0.00E+00	1.95E-01	CC2	Medium	5.00E-02
RC-S three-story BF buildings	C1L	Low-Rise Concrete Moment Frame	0.00E+00	1.79E-02	D	RC frame with moderate level of ERD	0.00E+00	1.95E-01	CC2	Medium	5.00E-02
RC-G six-story BF buildings	C1M	Mid-Rise Concrete Moment Frame	0.00E+00	1.41E-02	C	RC frame without ERD	0.00E+00	1.95E-01	CC2	Medium	5.00E-02
RC-S six-story BF buildings	C1M	Mid-Rise Concrete Moment Frame	0.00E+00	1.41E-02	D	RC frame with moderate level of ERD	0.00E+00	1.95E-01	CC2	Medium	5.00E-02
RC-G three-story IF and PF buildings	C3L	Low-Rise Concrete Frame with URM Infill Walls	0.00E+00	2.09E-02	C	RC frame without ERD	0.00E+00	1.95E-01	CC2	Medium	5.00E-02
RC-S three-story IF and PF buildings	C3L	Low-Rise Concrete Frame with URM Infill Walls	0.00E+00	2.09E-02	D	RC frame with moderate level of ERD	0.00E+00	1.95E-01	CC2	Medium	5.00E-02
RC-G six-story IF and PF buildings	C3M	Mid-Rise Concrete Frame with URM Infill Walls	0.00E+00	1.84E-02	C	RC frame without ERD	0.00E+00	1.95E-01	CC2	Medium	5.00E-02
RC-S six-story IF and PF buildings	C3M	Mid-Rise Concrete Frame with URM Infill Walls	0.00E+00	1.84E-02	D	RC frame with moderate level of ERD	0.00E+00	1.95E-01	CC2	Medium	5.00E-02
PRC buildings	PC2L	Low-Rise Precast Concrete Frames with Concrete Shear Walls	0.00E+00	2.09E-02	–	–	–	–	CC2	Medium	5.00E-02
S buildings	S2L	Low-Rise Steel Braced Frame Steel	0.00E+00	1.12E-02	–	–	–	–	CC2	Medium	5.00E-02

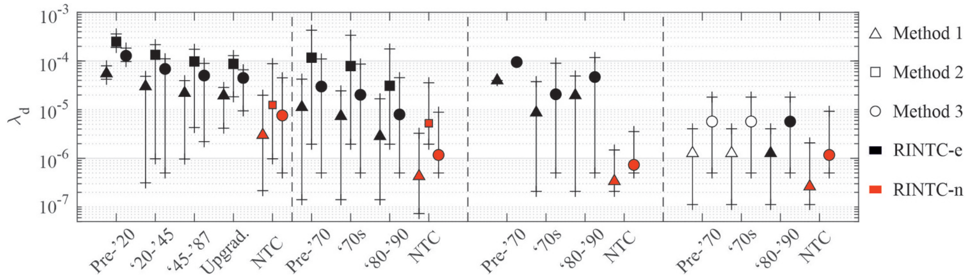

Figure 1. Averages and range of fatality rates as a function of the design epoch. (White markers indicate that design of S for the '80-'90 epoch also applies to previous epochs).

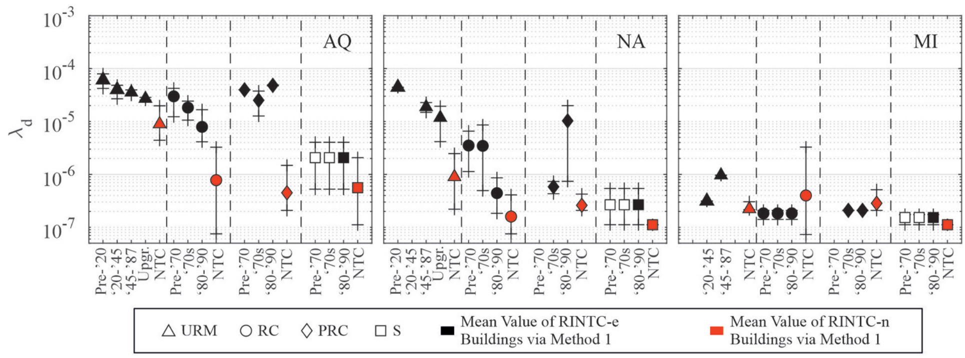

Figure 2. Average and range of fatality rates as a function of the design epoch and site. (White markers indicate that design of S for the '80-'90 epoch also applies to previous epochs).

For a deeper understanding, Figure 2 shows the fatality rates for each considered structural typology, as a function of the design epoch and the construction site. To prevent the figure from being cluttered, considering that a similar trend is observed for all the methods considered, only the results from method 1 are reported. Once again, the markers represent the averages, and the vertical bars are the ranges of rates contributing to the mean. For the high-hazard site, namely L'Aquila, the average fatality rates are below 10^{-6} for the current code and above this value for previous codes, for all structures except masonry, where the improvement of safety is present, yet smaller. Also for the mid hazard site of Naples, the current-code rates are around 10^{-6} or lower, yet the drop with respect to previous codes appears smaller compared to L'Aquila, and it is almost invisible in Milan, a low-hazard site. These results appear to confirm previous conclusions, for example, the relatively limited relevance of seismic design, for structural safety, in moderate-to-low hazard sites (Iervolino et al. 2023). URM rates appear generally larger than other typologies at least at MI and NA, and this was expected by the failure rates, reflecting – for example – that unreinforced masonry in moderate-to-high seismic hazard regions affects safety.

5. Final Remarks

This technical note presented the evolution of building seismic safety, measured in terms conventional of fatality rates, corresponding to the development of design codes. The results are based on the failure rates from a large research project, computed for pre- and low-code Italian buildings, somewhat representing the existing Italian residential and industrial building stock. The fatality probabilities conditional on structural performance rates were conventionally derived by means of three methods based on: (1) HAZUS; (2) EMS; (3) and fib80 bulletin.

It is found that the three methods considered provide average results spanning about one order of magnitude. The fatality rates obviously follow the trend of the failure rates, decreasing with the development of design technology, with the largest improvement due to the enforcement of the current code. This tends to hold independent of the design epoch, which confirms previous findings for current-code-conforming structures.

Looking at method 1, at the L'Aquila (high hazard) site, the current code reduces the order of magnitude of the average fatality rates to less than 10^{-6}, for almost all structural typologies. This also happens at the mid- and low-hazard sites. However, the relative safety improvement gained with the last codes tends to reduce as the design hazard reduces, which is expected, as the impact of seismic design on seismic reliability generally reduces with reducing design hazard. In Milan, seismic hazard is low enough, to compute the failure rates, to require extrapolation of both hazard and fragility functions, explaining why safety improvements are hardly appreciated.

Finally, it is to underline that the fatality rates computed herein have the advantage to represent a quantitative measure of structural safety; nevertheless, the way they were derived herein is highly simplified and conventional, such that these results are mainly for comparison and should be used with caution otherwise.

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Author contributions

CRediT: **Iunio Iervolino:** Conceptualization, Funding acquisition, Methodology, Supervision, Writing – original draft, Writing – review & editing; **Adriana Pacifico:** Data curation, Formal analysis, Investigation, Software, Validation, Writing – review & editing.

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Data Availability Statement

Data used in this study are available upon request to the authors.

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