Seismic risk-based design of simply-supported bridges in Italy

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ABSTRACT: Current design codes are mostly based on a force-based seismic design approach, which accounts for the inelastic capacity of structures by means of a reduction coefficient, i.e. behavior factor. In recent years, there has been a growing interest in assessing the safety margin of code-compliant structures. By overcoming the constraints linked to computational efforts, structural reliability analysis has started to be diffused beyond academia and into practical engineering applications. Consequently, recent studies assessed the level of structural safety implicitly achievable by following design codes and pointed out how current code provisions lead to a non-uniform safety level. For the abovementioned reasons, the present contribution assesses the seismic structural safety of newly-designed bridges in Italy. This is done by quantifying the failure probability via the solution of the direct reliability problem, which corresponds to the evaluation of the probability of exceeding one (or more) limit state(s) of interest during the bridge's design life. The work proposes a methodology for the risk-targeted design of bridges, considering as a reference different configurations of simply-supported bridges with circular single-column piers.

1. INTRODUCTION

The majority of seismic design codes used worldwide rely on force-based methods, where the earthquake action used for sizing the structural components of a system is expressed in the form of a Uniform Hazard Spectrum (UHS) (Gkimprixis et al. 2020). This provides the seismic demand, expressed in terms of spectral acceleration at different structural periods. The choice to design a structure in accordance with a "uniform" level of seismic demand relies on the assumption that such a procedure would lead to the same annual probability of failure (i.e. collapse) wherever the building is located (Gkimprixis et al. 2020). Following the development of modern performance-based earthquake engineering, the research community has focused on understanding whether such a design approach is able to ensure a sufficient and uniform level of structural safety against earthquake actions for different structural archetypes located at various sites. Many studies have shown that this objective was not achievable following a uniform hazard design framework (e.g. Cornell and Krawinkler 2000, Tubaldi et al. 2012). Over the past decade, risk-targeted seismic design emerged as one of the most promising approaches for designing structures with controlled seismic risk and/or loss levels. While

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most of the studies and codes focus on the design of buildings, risk-targeted bridge design is a less explored topic. In fact, only a few studies have proposed risk-targeting design methods for these structures. Wang et al. (2014) proposed a method to design reinforced concrete (RC) bridge columns to achieve a uniform risk of failure. Deb et al. (2022) proposed a method for risk-targeted performance-based seismic design of bridge piers for Californian Ordinary Standard Bridges to facilitate risk-informed design and decision making.

The present study proposes a simplified risk-targeted method for the seismic design of RC piers in multi-span bridges. The only design variables considered are the pier diameter and the longitudinal reinforcement ratio. A meta-model is built to describe the changes in the bridge seismic fragility with these two design parameters. The optimal values of the design parameters are found as the solution of a simplified reliability-based optimization problem, for which it is not necessary to resort to time-consuming optimization strategies. The methodology is applied in various locations across Italy to illustrate the variations in the optimal risk-based design properties of bridges across regions with varying seismic hazard.

2. RISK-TARGETING DESIGN PROCEDURE

The assessment of the bridge risk and the design of the bridge properties that satisfy a prefixed performance level are evaluated via direct and inverse reliability problems. The risk-targeted design problem for a single bridge pier is considered.

2.1. Direct problem

The basis of the proposed design procedure is the solution of the direct reliability problem, which corresponds to evaluating the probability of exceeding one (or more) limit state(s) of interest during the time interval of interest. For this purpose, an intensity-measure (IM)-based approach is employed (Scozzese et al. 2020). A capacity/demand format is used to evaluate the limit-state exceedance probability given the seismic intensity. The capacity is measured by a positive real-valued random variable C, whose possible realizations are denoted by c, with probability density function (PDF) $f_C(c)$ and cumulative distribution function (CDF) $F_C(c)$. The demand D is also expressed as a positive real valued random variable, whose possible realizations are denoted by d. The conditional distribution of the demand following events with a seismic intensity im is described by $f_{D|IM}(d|im)$. The probability of failure associated to the condition $C<D$ conditional to $IM=im$, is:

$$P_{f|IM}\left(im\right)=\int F_C\left(z\right)f_{D|IM}\left(z\middle|im\right)dz \qquad (1)$$

It is assumed that an event such that $IM>im$ can be described by a Poisson process fully defined by the Mean Annual Frequency (MAF) $\nu_{IM}(im)$. Under the assumptions that the probability distribution of the earthquake characteristics remains the same at each earthquake occurrence, and so does the probability of exceedance of the limit state, the probability of failure per year (failure rate) coincides with the MAF of failure and can be evaluated as follows:

$$\nu_f=\int_{im} P_{f|IM}\left(im\right)\cdot\left|\mathrm{d}\nu_{IM}\left(im\right)\right| \qquad (2)$$

The probability of failure in a time interval, e.g. the expected design lifetime t_L, can be obtained as:

$$P_{f,t_L}=1-e^{-\nu_f\cdot t_L} \qquad (3)$$

The capacity is assumed as a log-normal random variable and the two parameters associated to this distribution, the median $\hat{c}$ and the standard deviation of the logarithms β_C, are known and independent of the IM. It is also assumed that the demand conditional on the seismic intensity is a log-normal random variable, with parameters $\hat{d}$ and β_D. The relationship between IM and D can be expressed as:

$$\log\left[D\middle|IM=im\right]=\log\left[\hat{d}\left(im\right)\right]+\varepsilon=a+b\cdot\log\left(im\right)+\varepsilon \qquad (4)$$

where ε is a normally distributed random variable with zero mean and standard deviation β_D. The three parameters a, b and β_D can be determined through ordinary least squares regression. In this study, cloud analysis is carried out to develop the probabilistic seismic demand model (Jalayer 2003). The conditional probability of failure can be expressed in a closed form as:

$$P_{f|IM}(im) = \Phi\left[\frac{\log\left[\hat{d}(IM)/\hat{c}\right]}{\sqrt{\beta_D^2+\beta_C^2}}\right] \tag{5}$$

and the MAF of failure can be evaluated by Eqn. (2) once the MAF of im is assigned.

2.2. Inverse problem

Let $\mathbf{x} \in \mathrm{R}^n$ denote the vector of design parameters (e.g. pier longitudinal reinforcement ratio and pier diameter). The risk-targeted design of bridges is an inverse reliability problem that can be cast in the form of an optimization problem: find the set of optimal design parameters $\mathbf{x}^*$ such that an objective function (cost function) is minimised. The solution must satisfy a stochastic constraint requiring that the failure probability (or the MAF of failure) is less or equal to a pre-fixed value, as well as other constraints on the values that can be assumed by $\mathbf{x}$. In mathematical terms, the problem can be formalised as follows:

$$\begin{aligned} &\min_{\mathbf{x}} && g(\mathbf{x}) \\ &\text{subject to} && \mathbf{h}(\mathbf{x}) \le 0 \\ & && v_f(\mathbf{x}) - \bar{v}_f \le 0 \end{aligned} \tag{6}$$

where $g(\mathbf{x})$ is a cost function, depending on the design parameters, and $\mathbf{h}(\mathbf{x})$ is the set of constraints on the range of variation of $\mathbf{x}$. In Eqn. (6), the dependency of the MAF of failure on the design parameters $\mathbf{x}$ has been made explicit. The choice of a suitable cost function is essential for ensuring that a single design point is obtained. In fact, various combinations of the design parameters ensure that $v_f(\mathbf{x}) - \bar{v}_f \le 0$.

2.3. Design procedure

The reliability-based design procedure for this problem consists of the following steps:

1. Select various combinations of the design parameters DPs. These could be arranged to form a design of experiments matrix $\boldsymbol{X}_E = [\mathbf{x}_1 .. \ \mathbf{x}_j .. \ \mathbf{x}_{N_E}] \in R^{n \times N_E}$, where $\mathbf{x}_j = [x_{1j} \ \ x_{2j} \ \ x_{nj}]^T \in R^n$ denotes the vector corresponding to the j-th combination of design parameters, and N_E denotes the total number of design points;
2. For each combination of the DPs, the design flexural resistance M_{Rd} of the plastic hinge section at the base of the pier is derived in accordance with Eurocode 8 provisions (CEN EN 1998-1:2004). Subsequently, the transverse reinforcement is designed by applying capacity design principles (CEN EN 1998-1:2004); the confined concrete properties in the plastic hinge are evaluated using the Mander et al. (1988) model and a nonlinear FE model of the bridge is developed;
3. Cloud analysis is performed to develop a probabilistic demand model for the EDPs of interest. In this study, a single limit state, corresponding to the exceedance of the ductility capacity of the bridge, is considered. Thus, the monitored EDP is the displacement demand at the pier top, which must be compared to the displacement capacity;
4. The probability $P_{f|IM}(im, \mathbf{x}_E)$ of exceedance of the limit state of interest conditional to the chosen IM and the combination of DPs in $\mathbf{x}_E$ is evaluated;
5. Based on the values of the conditional failure probability evaluated in correspondence of the support points, a surrogate model is fitted that provides the conditional failure probability for any possible value of $\mathbf{x}$ without needing to perform other seismic response analyses. The simplest approach for developing the surrogate model is to use linear interpolation;
6. Given a site of interest, characterized by a hazard curve $v_{IM}(im)$, the MAF of failure

given $\mathbf{x}$, $v_f(\boldsymbol{x})$, can be evaluated and used to solve the problem formalised in Eqn. (6).

2.4. Cost function

The form adopted for the optimization problem is such that the consequences of pier failure in terms of direct and indirect losses are controlled by setting a maximum value of the MAF of failure. Since the total bridge life cycle cost is the sum of the cost of bridge construction and the cost due to failure, in order to minimise this cost one could consider the pier cost as the cost function. The cost function is assumed to coincide with the design resisting moment at the base of the pier, M_{Rd}. This quantity is expected to be correlated to the bridge construction cost, as it increases with the pier diameter, the amount of longitudinal reinforcement, the concrete class and other factors. Moreover, by minimising M_{Rd} the design shear (and thus the amount of transverse reinforcement) is also minimised.

2.5. Target failure probability

According to Eurocode 0 (CEN EN 1990:2002), the minimum recommended values of the reliability index for a reference period of 1 year should be 4.2 for consequence class CC1 structures, 4.7 for CC2 class structures and 5.2 for CC3 class structures. These correspond respectively to a MAF of failure of 1.33×10^{-5} years^{-1}, 1.33×10^{-6} years^{-1}, and 9.96×10^{-8} years^{-1}. However, it is not clear whether the values recommended by Eurocode 0 should be considered for the seismic design, as the draft version of the revised Eurocode 0 explicitly exclude these (Fajfar 2018). Douglas and Gkimprixis (2018) provide a summary of assessed and target MAFs of failure from the literature. Using a database of collapsed RC buildings in Italy and Greece over the previous few decades, Douglas and Gkimprixis (2018) conclude that the observed risk of collapse for such structures is between 1×10^{-6} and 1×10^{-5}. Because of the importance of road bridges both for life safety and their economic impact during and following earthquakes, a target MAF of failure of 1×10^{-6} is adopted for the following case study. The effect of this choice is examined by also considering 1×10^{-5} in a subsequent step.

3. CASE STUDY DESCRIPTION AND RESULTS OF PARAMETRIC ANALYSES

A two-span bridge with a continuous multi-span deck is used to illustrate the application of the proposed design method. The RC pier is 5.4m high and has a circular cross-section with diameter D_p. The three-dimensional FE model of the bridge is developed in OpenSees (2011) using the beam element with inelastic hinge developed by Scott et al. (2006) to describe the bottom of the pier, and linear elastic elements to describe the remaining part of the pier. The elastic damping properties of the system are characterized by a Rayleigh damping model. The same bridge is assumed to be located at various sites in Italy, characterized by different seismic hazards. The only DPs herein considered are the pier diameter D_p and the longitudinal reinforcement ratio ρ_L; thus $\mathbf{x}$=[D_p, ρ_L]. These DPs are assumed to vary in a realistic range that reflects construction practice and satisfies code requirements. In particular, the values of D_p of 1.4m, 1.8m, and 2.2m and the values of ρ_L of 1%, 2%, 3% and 4% are considered. For simplicity, two-dimensional linear interpolation is used to find the values of dependent variables corresponding to intermediate values of D_p and ρ_L.

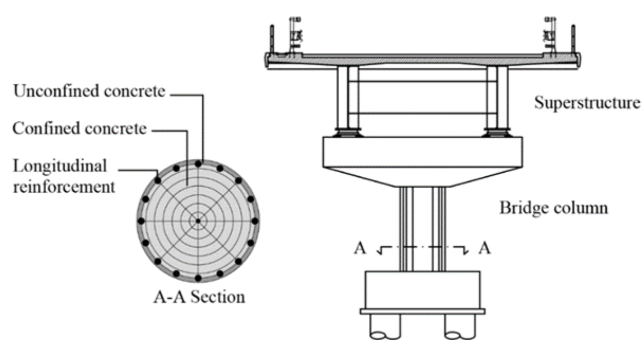
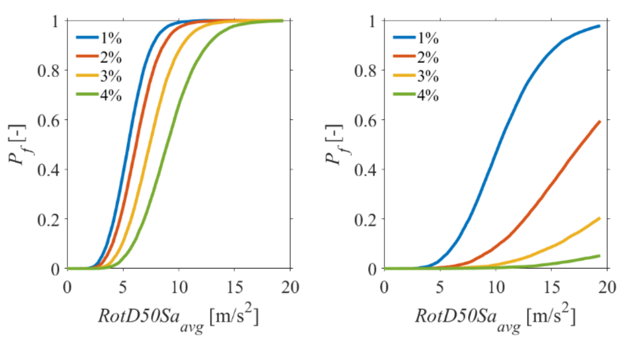


Figure 1: Bridge model considered.

Cloud analysis is performed to develop the probabilistic seismic demand models (PSDMs) for the various design cases. For this purpose, the same ground motion records employed in Tubaldi et al. (2022) is used. The maximum top displacements $u_{max,L}$ and $u_{max,T}$ along the longitudinal and transverse direction are considered to develop the PSDM. The intensity measure considered is $RotD50Sa_{avg}$, which is obtained as follows: first, the *RotD50* (Boore 2010) of the pseudo-acceleration response spectrum for the 221 records (two horizontal components) is computed, for a series of periods in the range between 0.1s and 2.5s, and for a 5% damping ratio. Then, the geometric mean across these periods is evaluated to obtain the $RotD50Sa_{avg}$. It is noteworthy that the proposed *IM* is not structure-specific.

Figure 2 reports the fragility curves for the various combinations of DPs. It can be observed that overall, increasing D_p is more effective than increasing ρ_L in reducing the bridge fragility.



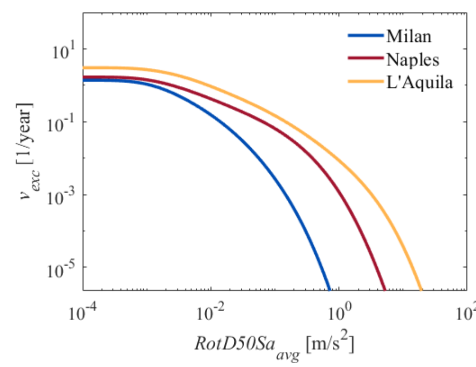
(a) (b)

Figure 2: Fragility curves for different combinations of DPs (a) D_p =1.4m (b) D_p =2.2m.

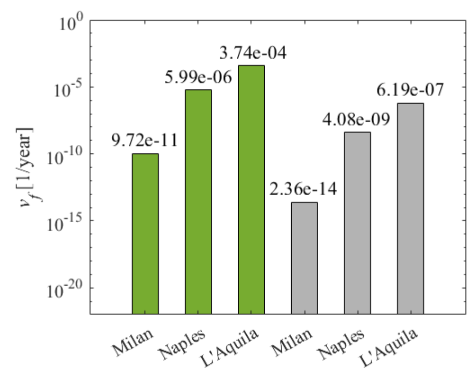
4. RESULTS OF THE RISK-TARGETING DESIGN APPROACH

The hazard curves for each site have been built using the software REASSESS V2.0 (Chioccarelli et al. 2019), which performs probabilistic seismic hazard assessment (PSHA). The ground motion prediction equation adopted is that proposed by Lanzano et al (2019) for *RotD50Sa*. The seismogenic source model is the one proposed by Meletti et al. (2008) with parameters taken from Barani et al. (2009). The interval of interest of the selected *IM* values ranges between 10^{-5}g and +2g. The condition of "Soil Type A" has been considered.

Figure 3a shows hazard curves in terms of MAF of exceedance of different values of $RotD50Sa_{avg}$ for three Italian cities: Milan, Naples and L'Aquila. The three sites are exposed to roughly low-, mid-, and high-seismic hazard. Figure 3b compares the MAFs of bridge pier failure corresponding to the minimum values of DPs (in green), and to the maximum values of DPs (in grey) for the three considered sites.



(a)



(b)

Figure 3: (a) Comparison of hazard curves in terms of RotD50Saavg for three different sites in Italy; (b) comparisons of risks for ρ_L =1%, D_p =1.4m in green and ρ_L =4%, D_p =2.2m in grey.

Figure 4 shows the values of the design resisting moment M_{Rd} at the pier base. It can be noted that increasing D_p and ρ_L results in an increase of M_{Rd}. In general, the design resisting moment is more sensitive to D_p than to ρ_L for low D_p values. However, for high D_p values increasing ρ_L results in large increase of M_{Rd}. In the same plot, the optimal design point (denoted by a star) and other combinations of DPs (marked with circles) satisfying the stochastic constraint of a MAF of failure equal or less than 10^{-6} are also shown. In particular, Figure 4a reports the results for a bridge located in L'Aquila. It can be observed that only one DP combination satisfies the required stochastic constraint in L'Aquila. Figure 4b shows the results for a bridge located in Naples. In this case, there are various combinations of DPs that satisfy the constraint on the acceptable risk of failure. Among these, the one that minimizes the resisting moment corresponds to D_p =1.4m and ρ_L =3.5%.

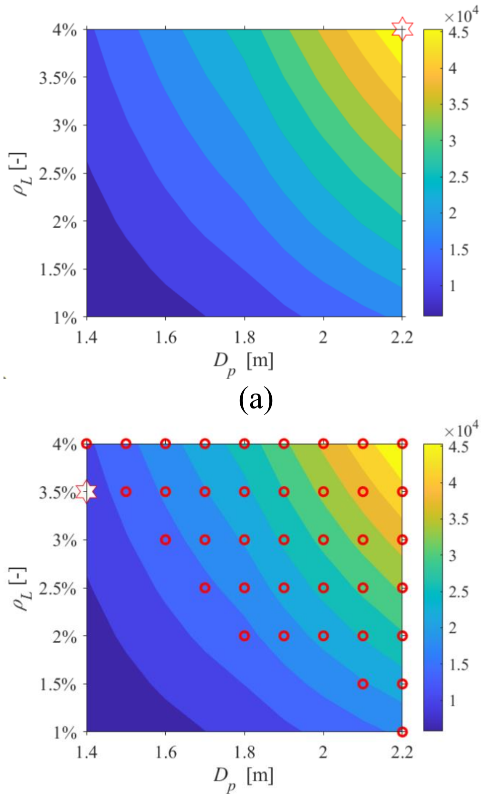


Figure 4: Values of the resisting moment M_{Rd} (unit kNm) for different combinations of DPs for a bridge site (a) in L'Aquila and (b) in Naples. The design parameters satisfying the stochastic constraint are marked with a circle, the optimal design point is marked by a star.

Figure 5 shows the variation of the MAF of failure with the design resisting moment M_{Rd} for the site of L'Aquila. It can be observed that there is a strong and inverse correlation between these two quantities. A similar trend is observed for other sites.

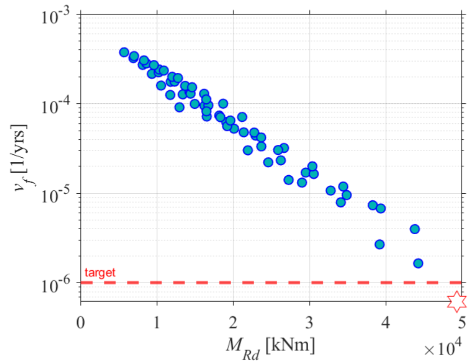


Figure 5: Variation of the MAF of collapse vs. design resisting moment M_{Rd} obtained for various DP combinations for a bridge site in L'Aquila. The dashed red line indicates the target MAF of failure of 10^{-6} and the optimal design point is marked by a star.

5. RISK-BASED DESIGN MAPS FOR ITALY

The proposed risk-based design procedure is applied to design the bridge pier across Italy, considering a target MAF of failure of 10^{-6}. Figure 6 shows the variation of minimum resisting moment M_{Rd} at the base of the pier across Italy, corresponding to the optimal design point. In large parts of Italy the minimum value of M_{Rd}, corresponding to ρ_L =1%, D_p =1.4m, is sufficient to satisfy the constraint and achieve risk levels less than 10^{-6}.

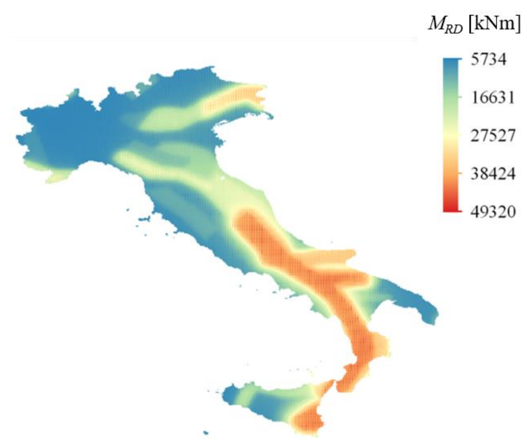
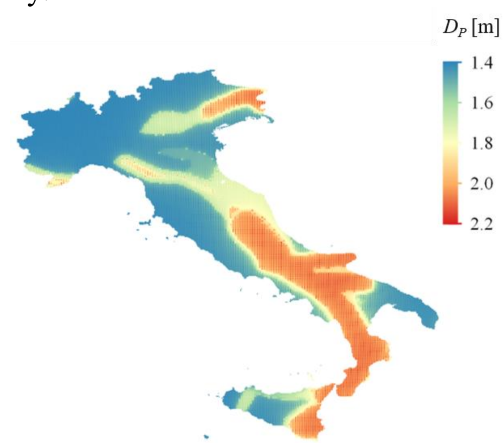
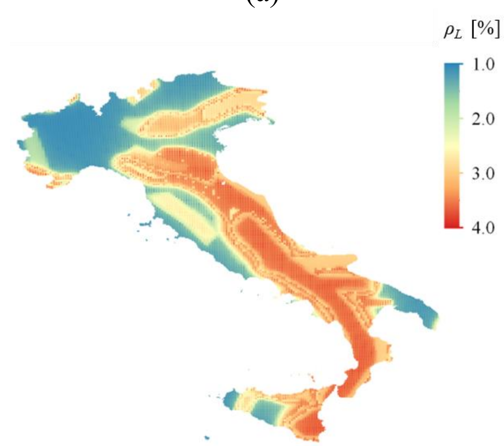


Figure 6: Variation across Italy of the minimum resisting moment M_{Rd} at the base of the pier.

Figure 7a and Figure 7b show a map of the optimal values of the pier diameter D_p and of the longitudinal reinforcement ratio ρ_L. In regions with lowest seismicity, the optimal DPs coincide with the minimum values of D_p and ρ_L, whereas in the regions with highest seismicity, they coincide with the maximum ones, as expected. Non-smooth changes of optimal DP values can be observed across adjacent regions that are characterized by quite similar levels of hazard. This is because high values of D_p and low values of ρ_L yield similar risk levels to low values of D_p and higher values of ρ_L. Obviously, a smoother variation of the optimal pier properties can be obtained if a single design parameter is considered, by keeping the other one fixed. Figure 8a shows the optimal values of ρ_L obtained considering a fixed diameter D_p of 2.2m. In this case, ρ_L exhibits a smooth variation across the country.



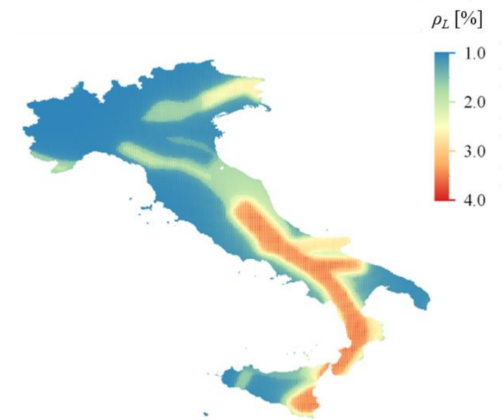
(a)



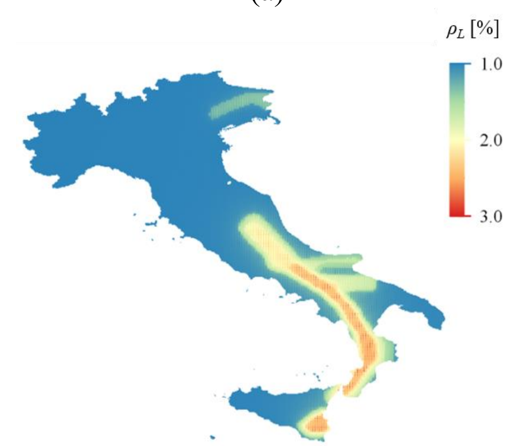
(b)

Figure 7: Variation across Italy of the optimal pier diameter D_p (a) and of the optimal ρ_L (b).

The effect of the choice of the target risk level on the design parameters is evaluated by applying the proposed design procedure for a target MAF of failure of 10^{-5}. The results obtained for a fixed value of the pier diameter D_p = 2.2m are shown in Figure 8b. As expected, increasing the target risk level results in a significant reduction of the longitudinal reinforcement ratio across Italy.



(a)



(b)

Figure 8: Variation of the optimal ρ_L across Italy for D_p =2.2m obtained considering a target MAF of failure of 10^{-6} (a) and 10^{-5} (b).

6. CONCLUSIONS

This article illustrates a risk-targeting design procedure for bridge piers. Based on the obtained results, the following main conclusions can be drawn:

- The design resisting moment at the base of the pier exhibits a significant inverse correlation with the target MAF of failure and can be used to define the objective (cost) function to be minimised;

- Targeting values of the mean annual frequency of failure lower than 10^{-6} $years^{-1}$ in regions of high seismicity requires design parameters that are out of the investigated range;
- A large variation of the optimal design parameters is observed across Italy, as a result of significant variations in the seismic hazard.
- In large parts of Italy, the minimum longitudinal reinforcement according to Eurocode 8 is sufficient to guarantee a target mean annual frequency of failure below 10^{-5} $years^{-1}$.

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