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FRAGILITY FUNCTIONS FOR REINFORCED CONCRETE FRAMED BUILDINGS SUBJECTED TO EARTHQUAKE-INDUCED LANDSLIDE HAZARD

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Abstract: *Post-earthquake reconnaissance missions have demonstrated that secondary events of earthquakes frequently cause even more losses than ground shaking. In the case of earthquake-induced landslides, fragility models have been recently proposed for two-dimensional reinforced concrete (RC) framed systems subjected to slow-moving earth slides, considering buildings standing close to the slope's crest. In this paper, novel fragility models for three-dimensional RC framed buildings subjected to seismically triggered landslides are presented. Two-storey buildings were assumed to be located downstream of the slope and threatened by flow-type landslide hazard. Fragility analysis was carried out by assuming probability distributions for a number of random variables (RVs) and generating one thousand realizations of both the landslide load and building structure, according to Monte Carlo simulation method. Uncertainties in landslide load included the width and impact angle with respect to the building façade. Uncertainties in geometry, material properties and capacity models of the structure were taken into account. For each simulation realization, the structural system was analysed in OpenSees under seismic loading and landslide impact. The peak ground acceleration (PGA) and impact velocity were considered as intensity measures of ground shaking and landslide impact, whereas the maximum inter-storey drift ratio was assumed as engineering demand parameter to identify the attainment of a prescribed damage level. Fragility curves highlight that the conditional probability of collapse given PGA significantly increases if cumulative damage to the RC structure due to ground shaking and landslide loading is considered. This can have strong impact on disaster risk assessment and mitigation.*

Introduction

Strong earthquakes often trigger natural and/or technological events such as landslides, fires and explosions, resulting in losses that are significantly higher than those associated only with ground shaking (see e.g. Wang et al., 2010). In the case of earthquake-induced landslides, slow earth movements can produce significant damage to building structures located close to the slope's crest. By contrast, rock falls and flow-type slides (e.g. debris flows, debris avalanches, earthflows, mudflows) can induce heavy damage to buildings located downstream of the slope, particularly when failure of key structural components result in a progressive collapse of the structure (Brunesi et al., 2015; Brunesi and Parisi, 2017; Adam et al., 2018). The intensity of slow-moving landslides is usually measured in terms of vertical/horizontal, absolute/differential, peak/permanent ground displacement (PGD) of the foundation soil that can be correlated with peak ground acceleration (PGA) and other intensity measures (IMs) of ground shaking (see e.g. Saygili and Rathje, 2008; Rathje and Saygili, 2009). Dealing with rock falls, either the kinetic energy or impact force associated with rock blocks can be used as IM well correlated with physical damage. Finally, scalar or vector-valued IMs, including debris depth/volume, impact velocity/pressure or a combination of them, can be used when considering flow-type landslides. In this respect, it should be noted that catastrophic effects of flow-type landslides can be induced by not only the high density, depth and velocity of the soil mass, but also the impact of objects such as vehicles dragged by the flow.

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Most of research on landslide risk assessment and reduction was carried out within two European projects named SAFELAND (2009–2012; https://www.ngi.no/eng/Projects/SafeLand) and INFRARISK (2013–2016; https://www.infrarisk-fp7.eu). Nonetheless, there is still a significant knowledge gap on the physical vulnerability to earthquake-induced landslides. Fotopoulou and Pitilakis (2013a) evaluated the vulnerability of low-rise reinforced concrete (RC) framed structures to slow-moving earth slides, considering: (i) PGA at the bedrock as IM; (ii) a finite 2D slope model with two different soil types, i.e. clay and sand; (iii) 2D frames with single bay and single storey; (iv) two foundation systems, i.e. isolated footings and continuous slab; and (v) a building located close to the crest of the slope. Fotopoulou and Pitilakis (2013b) extended their previous investigation and carried out a sensitivity analysis considering different idealized slope configurations, soil and geological settings, distances of the structure to the slope crest, and foundation types. Fragility curves were developed as a function of PGA or PGD for different slope angles, water table levels and soil types, foundation typologies, and levels of seismic design code. More recently, Fotopoulou and Pitilakis (2017a,b) evaluated the overall damage caused by earthquake ground shaking and landslide movement, developing coupled fragility curves for their implementation in quantitative risk analysis.

In this study, the fragility of RC framed building structures to flow-type landslides is investigated, assuming the building to be located downstream of the slope. The analytical methodology proposed by Parisi and Sabella (2017) was extended to consider 3D (rather than 2D) structural systems and cumulative damage due to earthquake ground shaking and landslide impact, the latter with any direction and width with respect to the building plan.

Modelling assumptions

Fragility analysis was carried out according to the Monte Carlo simulation method, assuming a RC framed structure representative of low-rise buildings designed for gravity loads only according to past Italian building codes in force before 1971 (Manfredi et al., 2007). The structure was a two-storey framed system with rigid basement, squared plan, one-way floor systems, and three columns and two beams per storey along each direction (Figs. 1 and 2). The columns were assumed to have squared cross section, whereas beams had the same span length and rectangular cross section.

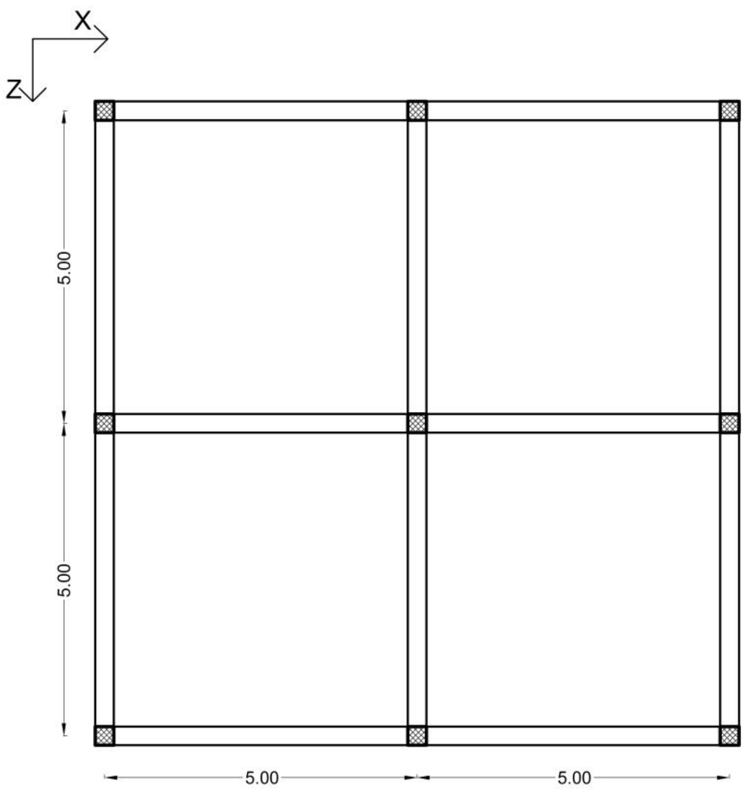


Figure 1. Building plan (dimensions in metres).

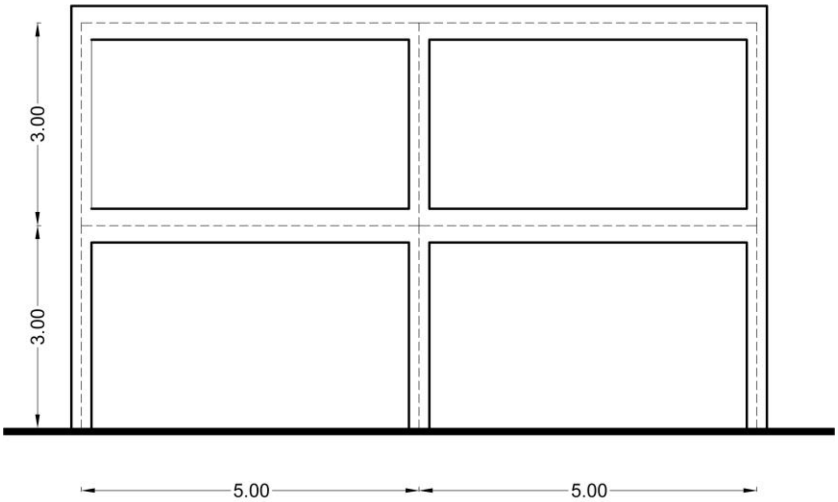


Figure 2. Building elevation (dimensions in metres).

Beams and columns were modelled as inelastic beam-column elements in accordance with a lumped plasticity approach. Plastic hinges were assigned to end sections of frame members. Moment–rotation diagrams were derived under the following assumptions: (i) concrete with zero tensile strength and compressive behaviour modelled through an elastic parabolic branch followed by a perfectly plastic branch; (ii) reinforcing steel modelled as elastic-perfectly plastic material; and (iii) moment (M) and rotation (θ) capacities influenced by potential flexural-shear interaction (Biskinis et al., 2004) and biaxial bending (Di Ludovico et al., 2013). In order to account for the lack of seismic detailing, the ultimate chord rotation was reduced by a factor equal to 0.825 as per Eurocode 8 (EC8) - Part 3 (EN 1998-3:2005). In case of shear or mixed flexural-shear failure, the ultimate rotation was defined as follows (Zhu et al., 2007):

$$\theta_a = 0.184 \exp(-1.45\mu) \tag{1}$$

where μ is the effective friction factor on the shear crack surface.

Floor systems were not explicitly modelled, so they were only assumed to behave as rigid floor diaphragms, distributing gravity loads on beams according to the orientation of floor joists. Infill walls were not included in the capacity model of the structure.

PGA was assumed as IM for earthquake ground shaking. This choice is supported by previous studies where PGA and Arias intensity were identified as IMs establishing the best correlation between seismic ground shaking and landslide occurrence (e.g.: Wang et al., 2010; Jafarian et al., 2018). Furthermore, PGA is usually assumed to define the seismic hazard level in design and assessment of structures according to national and international building codes, such as EC8 - Part 1 (EN 1998-1:2004). Regarding the intensity of the landslide event, the impact velocity of the soil mass was assumed as IM. This is motivated on one hand by the relationship between the impact velocity and kinetic energy of the flow, and on the other, by the high correlation of both non-structural and structural damage with such an IM (Mavrouli et al., 2014). The landslide load on the structure consisted of a lateral pressure defined through the following equation:

$$p_{dx}(z_s) = \rho g(D - z_s) + \rho v^2 \cos^2 \delta \tag{2}$$

$$p_{dz}(z_s) = \rho g(D - z_s) + \rho v^2 \sin^2 \delta \tag{3}$$

where: g = acceleration of gravity; D = flow depth; ρ = flow density; z_s = height of generic soil layer from the sliding surface of the flow; v = impact flow velocity; δ = angle between flow direction and perpendicular axis of impacted building façade (δ = 0 in case of perpendicular impact, which is the worst loading condition). On the right-hand side of Eq. (2), the first contribution defines the hydrostatic pressure that is linearly distributed over the height, whereas the second contribution provides the kinetic pressure that was assumed to be uniformly distributed. According to Parisi and Sabella (2017), an equivalent uniform pressure p_{eq} was derived to be applied on columns. That pressure was used to define the components of a lateral load per unit length defined as follows:

$$q_{cx} = p_{eq} b_c \cos\delta \tag{4}$$

$$q_{cz} = p_{eq} h_c \sin\delta \tag{5}$$

where b_c and h_c indicate the dimensions of column cross section along the X- and Z-axis of the building plan. It is important to note that the height of the soil flow was assumed to range between 0.5 and 1.5 times the inter-storey height. Figure 3 shows the loading pattern in plan with respect to the structure.

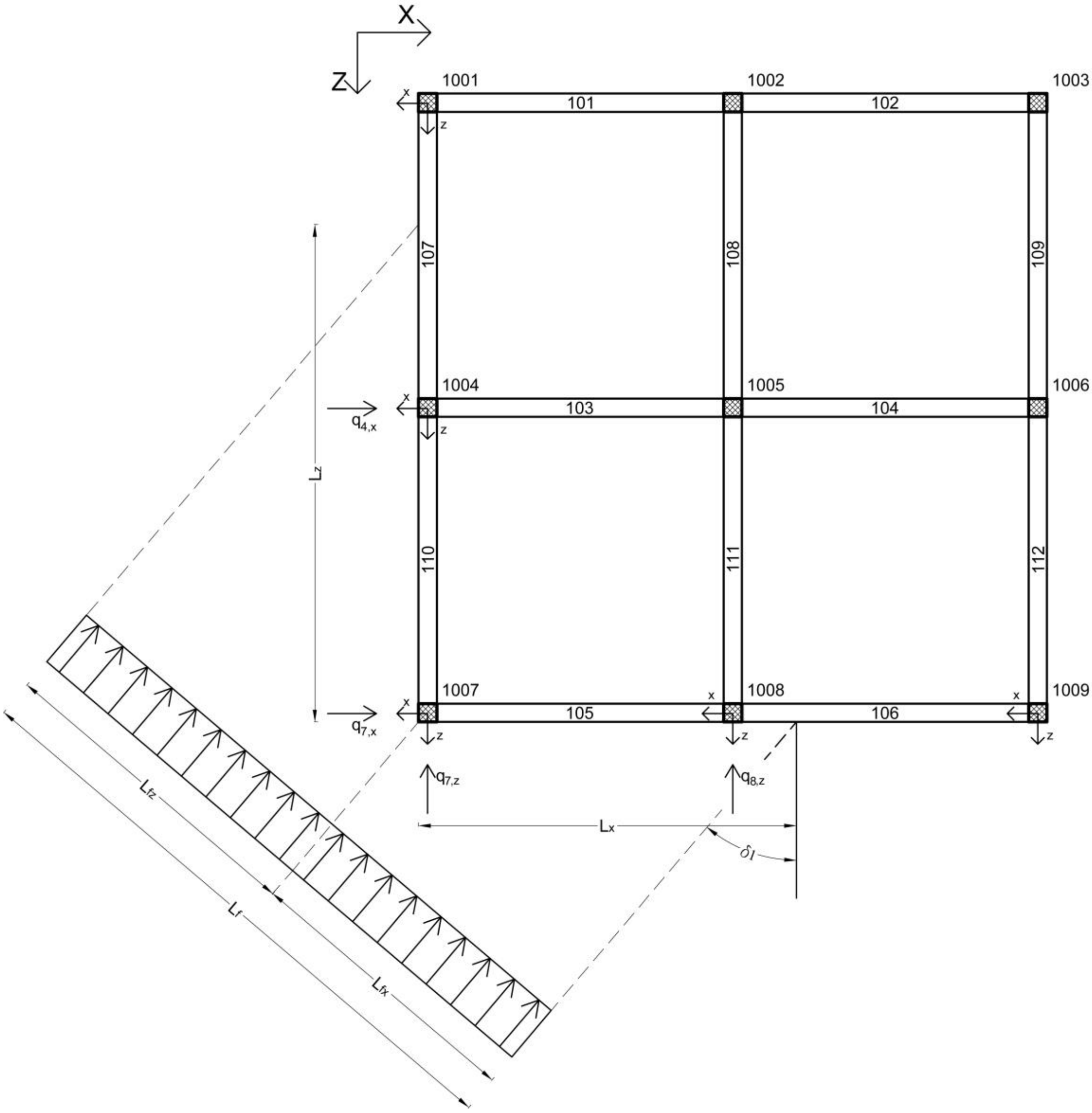


Figure 3. General scheme for diagonal landslide impact.

Fragility analysis

In this study, the fragility of the selected type of gravity-load designed building was estimated for the ultimate limit state, according to the flow-chart shown in Figure 4. Uncertainties associated with loads, materials, geometry and capacity model were modelled through random variables (RVs) with the probability distributions and statistics (i.e. mean and coefficient of variation, CoV, or range), which are listed in Table 1 and were used in previous studies (e.g. Parisi and Sabella, 2017). Concrete cover in columns and beams was deterministically set to 30 mm. The impact angle δ and width L_f of the earth flow were assumed to be uniformly distributed over the ranges [0, 90°] and [d_c, d_b], respectively, being d_c the diagonal of the column cross section and d_b the diagonal of the building plan. In Table 1, the following symbols have been used: f_c = compressive strength of concrete; f_y = yield strength of reinforcing steel; l = span length of beams; ρ_{tot} = total longitudinal reinforcement ratio; ρ_w = transverse reinforcement ratio; θ_y = yield rotation; θ_f = ultimate rotation corresponding to flexural failure; V_u = ultimate shear force corresponding to shear failure. Subscripts 'exp' and theor' indicate experimental and theoretical values of variables, so their ratios define the statistical error of the capacity model.

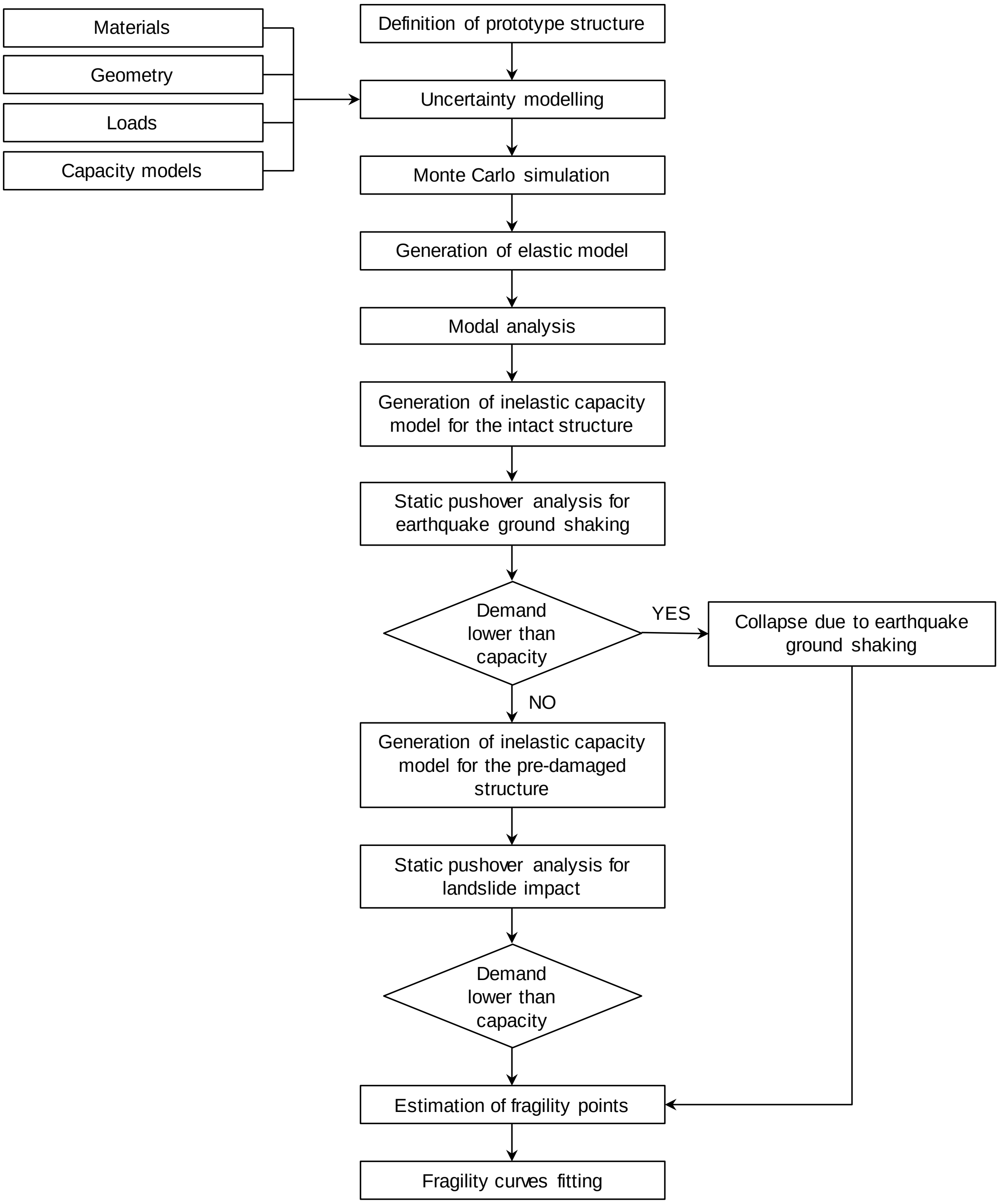


Figure 4. Flow-chart of earthquake-induced landslide fragility analysis.

The fragility procedure was implemented in MATLAB® (2018) and allowed an automatic generation and analysis of 10^3 demand-capacity models within OpenSees® (McKenna et al., 2004), according to a Monte Carlo sampling procedure that took into account the uncertainty modelling discussed above. For each realization, an elastic capacity model of the structure was first developed and a multi-mode analysis was carried out to determine modal shapes and vibration periods. Then, a first nonlinear model of the intact structure was generated and analysed under gravity loads and earthquake actions. Static pushover analysis with displacement control was performed to predict the seismic response of the structure.

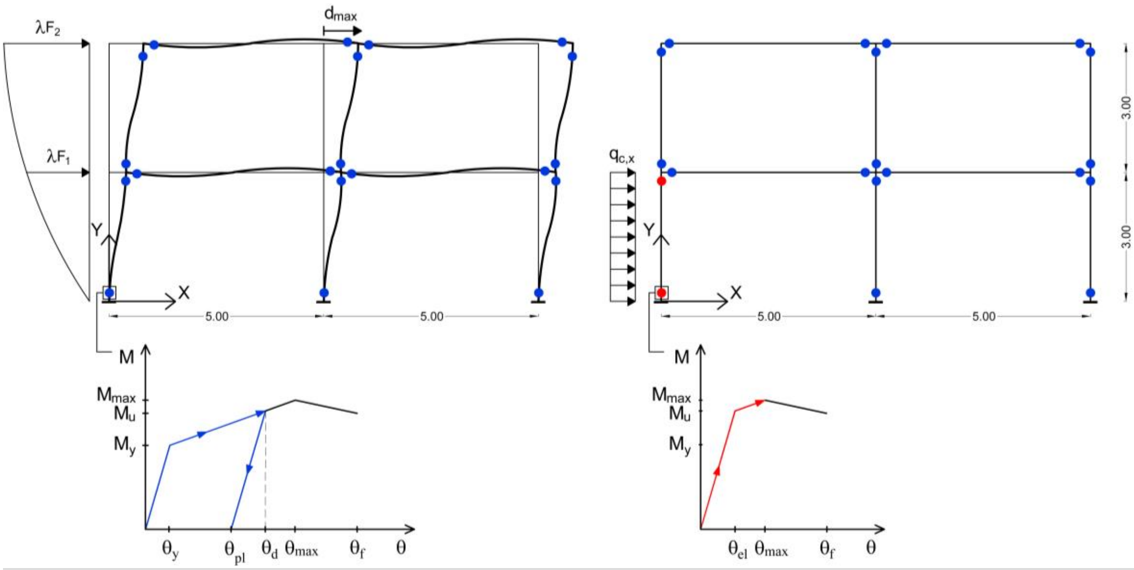
Category	Item	RV	Mean/Range	CoV	Distribution
Loads	Flowing soil	ρ	1.8 ton/m^3	3%	Uniform
		D	h_i	$0.08h_i$	Uniform
		δ	0–90°	–	Uniform
		L_f	d_c –20 m	–	Uniform
Materials	Concrete	f_c	16 MPa	31%	Lognormal
	Reinforcing steel	f_y	250 MPa	8%	Lognormal
Geometry	Column	b	300 mm	4%	Normal
		h	300 mm	4%	Normal
		h_i	3 m	4%	Normal
		ρ_{tot}	0.59%	0	Normal
		ρ_w	0.20%	0.02%	Normal
	Beam	b	300 mm	4%	Normal
		h	500 mm	4%	Normal
		l	5 m	4%	Normal
		ρ_{tot}	0.70%	0	Normal
		ρ_w	0.235%	0.06%	Normal
Capacity model	Column/beam	$\theta_{y,exp}/\theta_{y,theor}$	1.015	33.1%	Lognormal
		$\theta_{f,exp}/\theta_{f,theor}$	0.995	40.9%	Lognormal
		$V_{u,exp}/V_{u,theor}$	1	25%	Lognormal
		$\theta_{a,exp}/\theta_{a,theor}$	1	35%	Lognormal

Table 1. Random variables, statistics and probability distributions of case-study buildings.

The displacement demand was predicted through the N2 method as per EC8 - Part 3 (EN 1998-3:2005), at each level of PGA that was assumed to range from 0.05*g* to 0.3*g* with step equal to 0.025*g*. The upper bound to seismic intensity, i.e. PGA = 0.3*g*, was found through a preliminary analysis of the weakest capacity model of the structure. If displacement demand did not attain the capacity of the sampled capacity model, a second nonlinear capacity model was generated in OpenSees®, assigning residual capacity properties to plastic hinges. In this respect, the authors of this study made the following assumptions: (i) the landslide impact was assumed to occur distinctly after the end of earthquake ground shaking, allowing a sequential nonlinear analysis of the structure to predict cumulative damage produced by the landslide impact; and (ii) the unloading of plastic hinges after ground shaking followed the initial stiffness, meaning that a non-degrading stiffness model was adopted. When the inelastic model of the intact structure reached collapse under earthquake actions, a soft-storey mechanism was observed according to the lack of seismic conceptual design related with past Italian building codes (Manfredi et al., 2007). Static pushover analysis of the pre-damaged structure was thus carried out, replacing the horizontal force pattern by lateral pressures acting on one or more columns of the ground floor.

The impact velocity v was assumed to range between 0 and 10 m/s with step equal to 1 m/s. When v was set to zero, only the damage associated with ground shaking was evaluated, whereas landslide impact with the maximum selected velocity always caused collapse of columns. Such a static pushover analysis was performed with force control up to the target pressure associated with v. In order to account for effects of diagonal landslide impact on the structure, the biaxial interaction model proposed by Di Ludovico et al. (2013) was used, properly reducing the rotation capacity of plastic hinges in each direction of column cross sections. Figures 5a and 5b summarise the sequential modelling and analysis of the structure subjected to horizontal seismic actions and landslide-related pressures, respectively.

Based on the huge number of sample realizations ($N_{sim} = 10^3$), the fragility P_f — i.e. the probability of exceeding the ultimate rotation in a plastic hinge of a ground-floor column given PGA and v — was estimated as the ratio between the number of collapse cases and N_{sim}. Therefore, fragility points were obtained for each velocity level and were fitted by a lognormal distribution function to derive fragility curves (Porter et al, 2007). This means that a set of fragility curves were obtained, each of them associated with a landslide impact velocity.



(a) (b)

Figure 5. Sequential nonlinear modelling and analysis: (a) earthquake ground shaking; (b) landslide impact.

Discussion of results

Figure 6 shows the fragility curves of the case-study building class, which are plotted under increasing landslide impact velocity from 0 (no landslide impact) to 10 m/s. Each fragility curve is defined by the median velocity η and logarithmic standard deviation β, which are outlined in Table 2 together with the coefficient of determination R^2 that characterises the goodness of fitting. Over all cases, R^2 reached very high values ranging between 0.88 and 0.99.

Analysis results show that earthquake-induced landslides can produce a significant increase in structural damage, as reflected by variations in the probability of failure P_f. This is particularly true if a flow velocity equal to or greater than 4 m/s is considered, highlighting that low velocity levels (1–3 m/s) have negligible consequences in terms of structural damage. At the same time, when high PGA values are assumed, earthquake ground shaking directly induces heavy damage to the structure, resulting in a failure probability that is more or less equal to unity.

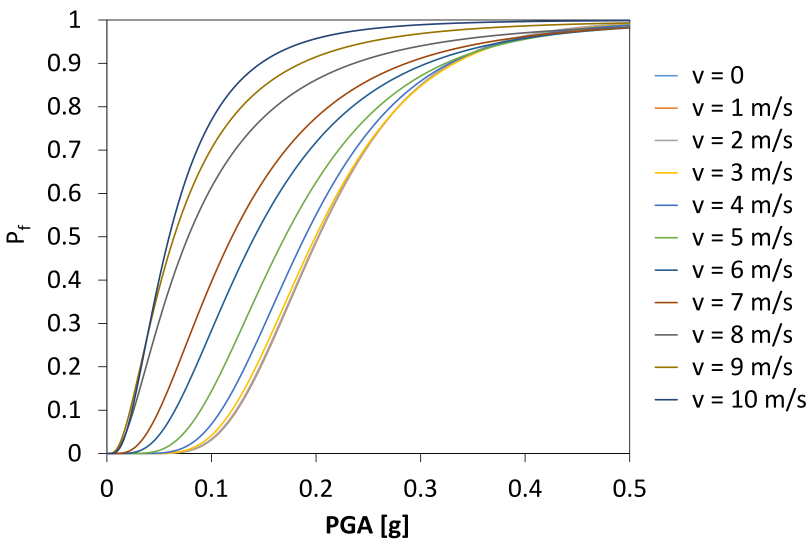


Figure 6. Fragility curves considering cumulative damage due to earthquake ground shaking and landslide impact.

v [m/s]	η [g]	β	R^2
0	0.20	0.38	0.9938
1	0.20	0.38	0.9935
2	0.20	0.38	0.9932
3	0.20	0.40	0.9891
4	0.19	0.44	0.9816
5	0.17	0.53	0.9642
6	0.14	0.65	0.9496
7	0.12	0.68	0.9667
8	0.08	0.87	0.9363
9	0.06	0.83	0.8848
10	0.06	0.70	0.9472

Table 2. Parameters and coefficient of determination of lognormal fragility curves.

In the case of flow impact velocity v = 0–3 m/s, the median acceleration was found to be approximately η = 0.20g with dispersion β = 0.38–0.40. By contrast, when v ranged from 4 to 10 m/s, the median acceleration significantly reduced from 0.19g to 0.06g with β ranging between 0.44 and 0.87. This last result reflects that the uncertainty level associated with structural vulnerability to both earthquake ground shaking and landslide is sometimes very high and should be taken into account in multi-hazard risk assessment.

Conclusions

A procedure for fragility analysis of RC framed buildings subjected to earthquake ground shaking and subsequent landslide impact has been presented for its use in probabilistic risk assessment of building portfolios. Monte Carlo simulation was integrated within a sequential nonlinear analysis that was aimed at evaluating cumulative damage due to horizontal seismic actions and lateral pressures acting at the ground floor of the building. Lumped plasticity inelastic modelling was combined with nonlinear static procedures to compare displacement demand and capacity for both the intact and seismically pre-damaged structures. Compared to previous studies, the additional novelty of this study is that fragility analysis was carried out by considering cases of diagonal landslide impact on the structure with any width of the flow.

Analysis results in terms of fragility curves associated with the attainment of the ultimate rotation in one or more ground-floor columns highlight that an earthquake-induced earth flow with moderate-to-high velocity can significantly increase the vulnerability of a RC framed structure designed only to gravity loads. This provides numerical evidence that seismic risk of structures located in landslide-prone areas should be assessed according to a multi-hazard approach, as demonstrated by strong earthquakes in several countries.

Future developments of this study will include the analysis of 3D structural models with masonry infill walls, considering potential benefits from seismic design rules and detailing.

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