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VULNERABILITY ASSESSMENT OF RC BUILDINGS DUE TO EARTHQUAKE INDUCED SLOW MOVING SLIDES

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ABSTRACT

The aim of the present study is to propose and quantify an efficient methodology for the vulnerability assessment of RC buildings exposed to earthquake induced slow moving slides. The vulnerability is defined through specific probabilistic fragility functions for specified limit states. The fragility curves are computed in terms of peak ground acceleration at the assumed seismic bedrock, given by the seismic hazard analysis, versus the probability of exceedance of each limit state, for all the building typologies considered. A two steps uncoupled analysis is conducted. In the first step, the differential displacements at the building's foundation level are estimated using FLAC 2D finite difference dynamic model (ITASCA Consultants, 2005). Gradually increasing acceleration time histories are applied at the base of the model to assess the building response and the associated ground displacements are computed accordingly. Then, the calculated differential displacements are applied as input to building's foundation model to assess the building's response for different ground landslide displacements induced by the earthquake. Limit states are defined in terms of a threshold value of building's material strain. The developed methodology is applied to single bay–single story RC buildings with varying strength and stiffness characteristics of the foundation system (isolated footings, continuous foundation), standing near the crest of a relative slow moving soil slide. The numerical analyses are performed using the fiber-based finite element code SEISMOSTRUCT (Seismosoft, 2005). Uncertainties related to the capacity of the building, the deformation demand and the definition of limit states are considered in the analysis.

Keywords: fragility curves, landslides, permanent displacements, RC buildings, foundation system

INTRODUCTION

Seismically triggered landslides represent one of the most important collateral hazards associated with earthquakes. They commonly account for a significant proportion of total earthquake damage related to human losses and damage to the built environment. Some of the most pronounced seismically induced landslides have occurred in Taiwan California, Japan, Italy, China and elsewhere, resulting to numerous casualties and tremendous (direct and indirect) damage to infrastructure.

Vulnerability assessment of the exposed elements to earthquake induced landslide hazards is a key component in a quantitative risk assessment (QRA) methodology. It may be defined as the degree of loss (expressed on a scale of 0 to 1) to a given element at risk resulting from the occurrence of a specified landslide event of given type and size. The vulnerability of human lives and properties is often different

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and it is strongly associated with the landslide type. For instance, a house may have a similar high vulnerability to both slow moving and rapid landslides, while a person living in it may have a low to negligible vulnerability in the first case (Dai et al., 2002; Fell et al., 2008). The vulnerability of elements at risk related to landslides is extremely difficult to assess for most landslide types as in most cases there is a scarcity of available damage data in quantitative terms. Furthermore, the magnitude or volume of a possible slide is difficult to foresee as it depends on the intensity, the frequency and the specific characteristics of the triggering event as well as on environmental factors (e.g., height of water table, pore water pressure built-up) at the moment of the event. Moreover, the slope geometry and the soil/rock material properties are not always properly defined.

Generally, the vulnerability to landslides may depend on (a) the landslide type and size (b) the triggering mechanism (intense rainfall, earthquake, etc), (c) the typology of the exposed elements (buildings, roads, pipelines, etc) and their specific structural characteristics,(d) their relative position to the potential unstable slope, and (e) the soil properties.

The physical vulnerability is generally described through fragility functions. Fragility relationships are essential components of quantitative risk assessment (QRA) studies. They are expressed in terms of relating landslide intensity with damage probabilities. Fragility curves provide for every element at risk, the conditional probability for the element to be in or exceed a certain damage state, under a landslide event of given type and intensity.

PROPOSED METHODOLOGY

Figure 1 illustrates the proposed framework in case of earthquake triggering relative slow moving slides.

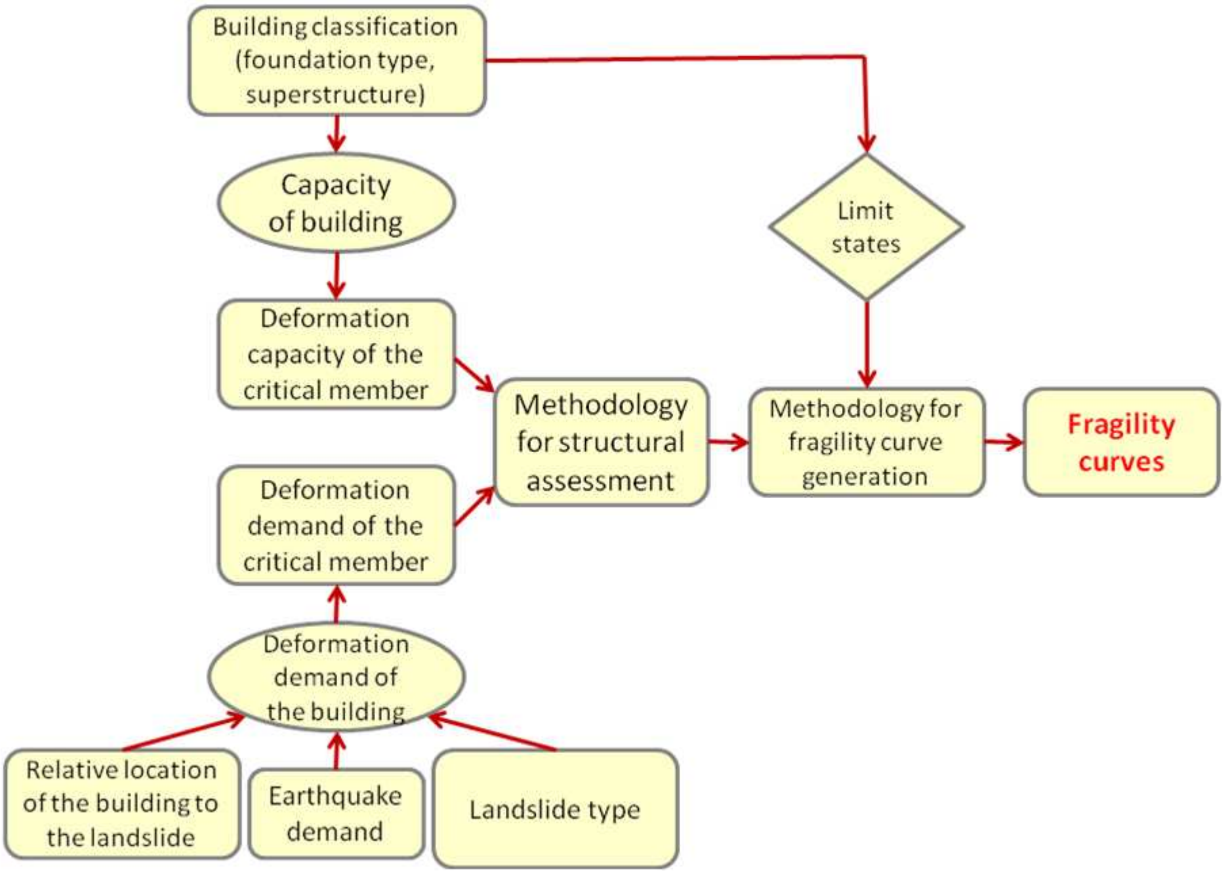


Figure 1. Flowchart for the proposed framework of fragility analysis of RC buildings

Building classification (foundation type, superstructure) constitute the capacity of the building. The earthquake demand, the landslide type and the relative location of the building to the potential unstable slope, constitute the deformation demand of the building. These two components (building capacity, deformation demand) can be considered as inputs to the simulation engine which is the third major component, i.e. the methodology for structural assessment. Structural response data obtained by analyzing the building capacity under the deformation demand is processed by the methodology for fragility curve generation to yield the results. Limit states, which are determined with respect to the building classification, properly selected empirical criteria and expert judgment, are required at this step. The final step of the methodology will result to the construction of the fragility relationships. Similar flowcharts may be defined for other triggering mechanisms (intense rainfall, erosion etc.). It is also possible to construct synthetic flowcharts combining different triggering mechanisms.

The landslide type is a crucial part of the proposed methodology as landslides of different types and sizes usually require different and complementary methods to estimate vulnerability. A relative slow moving slide that will produce tension cracks due to differential displacement to a RC building, exposed to the landslide hazard, is considered in the present study.

The relative location of the building to the landslide area is a very important contributing factor in estimating vulnerability. In large landslides, there are sensitive areas where damage in terms of total displacement or released energy will be more likely (or much higher). This occurs, for instance, in the landslide boundaries, such as the head or lateral sides, or at local scarps where tensile stresses develop with the result of cracks, surface ground depletion and local rotation (Fell et al., 2008). Landslides triggered by earthquakes tend to be clustered near ridge crests and hill slope toes. Peng et al. (2009) attributed this ridge- crest clustering to topographic effects, and the clustering at hill slope toes to dynamic pore-pressure changes in the water-saturated material of lower hill slopes. In the present study, a building standing near the crest where the seismic ground motion due to topographic effects is generally amplified is assumed (Bouckovalas & Papadimitriou, 2005; Ktenidou 2010).

The type of response to permanent total and differential ground deformation will depend primarily on the foundation type: for shallow foundations, the distinction is between rigid or flexible/unrestrained foundation systems. When the foundation system is rigid the building is expected rather to rotate as a rigid body and the damage states are defined empirically, as there is limited structural demand to the members of the building (apart from possible P-Δ effects at larger rotations). On the contrary, when the foundation system is flexible, the damage to the building members due to the various modes of differential deformation (differential horizontal, differential vertical and combination of them) can be estimated using an analytical procedure analogous to that of the response due to seismic ground motion (Bird et al., 2005; 2006).

When building response to ground failure comprises structural damage, damage states can be classified using the same schemes used for structural damage caused by ground shaking. Limit states are defined in terms of limit value of a component's strain based on damage observation from previous earthquake events, the existing knowledge related to earthquake damage levels, and published tolerances for non-earthquake related foundation deformations (Crowley et al., 2004; Bird et al., 2006).

In a probabilistic approach applied herein, the uncertainties related to the capacity of the building, the definition of the limit states and the deformation demand (differential permanent displacement) should be considered. The uncertainty in the displacement capacity is a function of the material properties, geometric properties, and the yield strain of steel and post-yield strain capacities of the steel and concrete. The uncertainty in the demand includes all of the variability associated with the ground motion estimation plus the additional uncertainties associated with the landslide type and size, the relative position of the building to the landslide area, the variability in soil parameters and stratigraphy and the uncertainty within the assessment of ground deformations.

Finally, the fragility curves will be derived in terms of peak ground acceleration at the seismic bedrock, which is the trigger of the landslide event in terms of seismic hazard, versus the probability of exceedance of each limit state for all the building typologies considered.

APPLICATION

Deformation demand- Numerical analysis

An application of the proposed methodology is presented herein. It should be mentioned that the landslide type (geometry, soil properties) and the relative location of the building to the potential unstable slope are treated below as deterministic parameters. To estimate the input differential displacements at the building's foundation level, we applied FLAC 2D finite difference slope model (ITASCA Consultants, 2005) (fig. 2) using an elastoplastic constitutive model with Mohr-Coulomb failure criterion, able to simulate large deformations for slope stability assessments. A small amount of Rayleigh damping (1 to 3%) is assigned to the model to account for the energy dissipation in the elastic range. The center frequency of the installed Rayleigh damping is selected to lie between the fundamental frequencies of the input acceleration time histories and the natural modes of the system. In the slope area, a fine grid discretization of 1m x 1 m is adopted, whereas towards the lateral boundaries of the model, where the accuracy requirements loosen, the mesh is coarser (2 m x 1.6 m). The model is 300m wide and 100 m high. It contains approximately 12600 elements. The slope height and inclination are 20m and 30^{o} respectively. Free field absorbing boundaries are applied along the lateral boundaries while quiet (viscous) boundaries are applied along the bottom of the dynamic model to minimize the affect of reflected waves. In order to apply quiet boundary conditions along the same boundary as the dynamic input, the seismic motions must be input as stress loads combining with the quiet (absorbing) boundary condition. The soil type is selected to represent sand soil corresponding to soil category C of EC8; its material, physical and dynamic properties are provided in Table 1.
A building is assumed to be located 3m from the slope crest. The building is modeled only by its foundation (uncoupled approach). The foundation width is 6m. Two different foundation systems are considered (Table 2): isolated footings and a uniform loaded continuous slab foundation. In the first case, the foundation is simulated with concentrated loads at the footings' links. The soil-structure interaction can be neglected in this case due to the flexibility of the foundation system. In the second case, the foundation system is modeled as a deformable elastic beam connected to the grid through appropriate frictional interface elements that can approximate the potential Coulomb sliding and/or tensile separation of the beam. The static factor of safety of the slope is calculated through a limit equilibrium method as Fs=1.45.

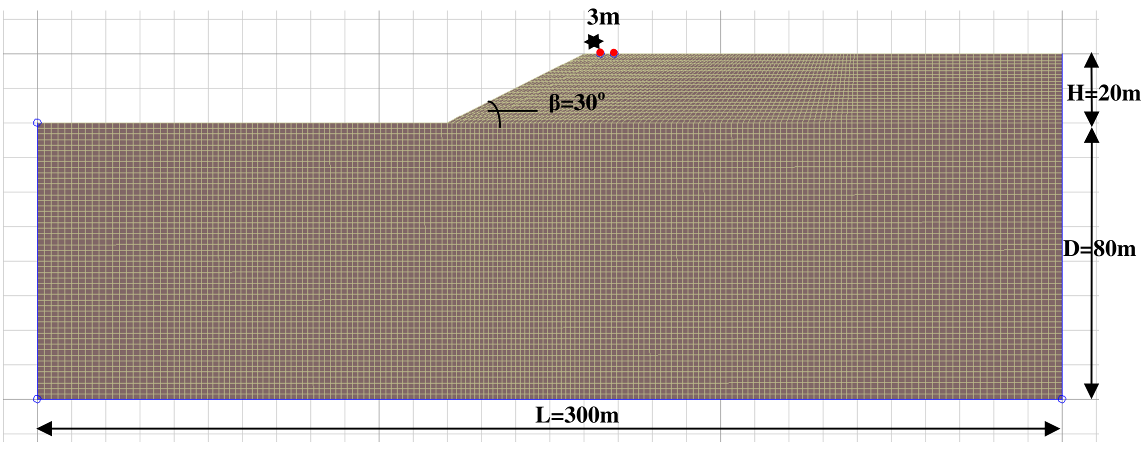


Figure 2. FLAC dynamic model

Table 1. Soil properties

Properties	SOIL C
Constitutive model	Mohr Coulomb
Dry density (KN/m^3)	18
Vs (m/sec)	250
Poisson's ratio	0.3
Cohesion (KPa)	0
Friction angle (degrees)	36
$N_{1(60)}$	21
Dr(%)	60

Table 2. Foundation properties

Properties	Foundation system	
	Stiff foundation	Flexible foundation
Element	beam	
Length (m)	6	
Density (KN/m3)	24	
Young's modulus (KPa)	2.90E+7	
Moment of inertia I (m^3)	0.0053	
Area (A) (m^2)	0.4	
Load (KN/m)	Uniform distributed q=25KN/m2	Concentrated load P=50KN/m

Four different earthquake records are used as input motion: (i) Valnerina (Cascia-L), Italy, Ms=5.8 , 1979, (ii) Athens (Kypseli-L), Greece, Mw=5.9, 1999, (iii) Montenegro-[TRA (EW)], former Yugoslavia, Mw=6.9, 1979 and (iv) Northridge (Pacoima Dam -L), USA, Ms=6.7, 1994. They all refer to free field rock soil conditions (soil category A in EC8). Before applied along the base of the model, they are subjected to appropriate correction (baseline correction, filtering and tapering) to allow for an accurate representation of wave transmission through the model.
The input accelerograms are scaled to five levels of peak ground acceleration (PGA=0.1, 0.3, 0.5, 0.7 and 0.9g) so as to assess the building response for different displacement magnitudes. This procedure will allow resulting in different damage states for the building and finally to be able to construct the corresponding vulnerability curves. It is worth noticing that when the soil structure interaction is considered, the differential horizontal displacements at the beam foundation are practically zero and the total differential displacement vector for the building is generally decreased.

Comparison with simplified displacement methods

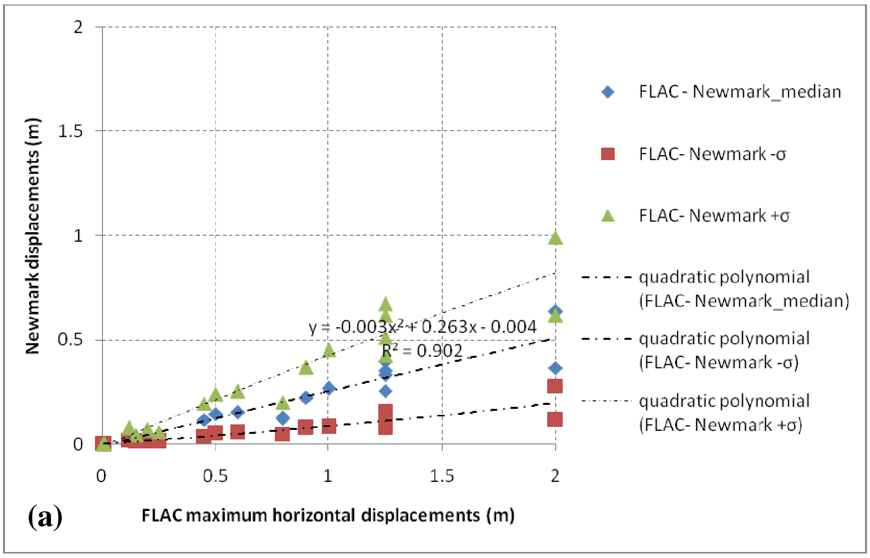
To validate the numerical results, they are compared, in terms of maximum permanent horizontal displacement, with the simplified Newmark-type displacement methods. The conventional Newmark rigid block model (Newmark, 1965; Jibson et al., 2003), as well as one of its improvements to account for the soil deformability using a coupled stick-slip deformable sliding block model (Bray & Travasarou, 2007), are used to calculate permanent displacements of the slide mass.
Bray & Travasarou (2007) displacement model captures the primary influence of the system's yield coefficient (k_y), its initial fundamental period (Ts), and the ground motion's spectral acceleration at a degraded period equal to 1.5Ts. The input accelerograms applied to both methods are the maximum

acceleration time histories at the slope area derived from the dynamic analysis. The yield coefficient, ky, is computed by applying the following relationship, as proposed in Bray (2007):

$$k_y=\tan(\varphi-\beta)+\ c/(\gamma\cdot H\cdot \cos 2\beta\cdot\ (1+\tan\varphi\cdot\tan\beta)) \qquad (1)$$

where φ= friction angle, c= cohesion, H= height of the critical sliding surface and β= slope angle.

The results of the above methods are summarized in figure 3 (a) and (b) respectively in comparison with the co-seismic displacements calculated using the numerical dynamic analysis. It is observed that the Newmark rigid block approach severely underestimates the computed displacements. This seems quite reasonable considering that Newmark's method treats the potential landslide block as a rigid mass that slides in a perfectly plastic manner on an inclined plane, which is not realistic in our case. The results of fully coupled stick-slip deformable sliding block model introduced by Bray &Travasarou (2007) are generally in good agreement with that of the dynamic analysis, yielding to slightly unconservative results for increased displacement levels.



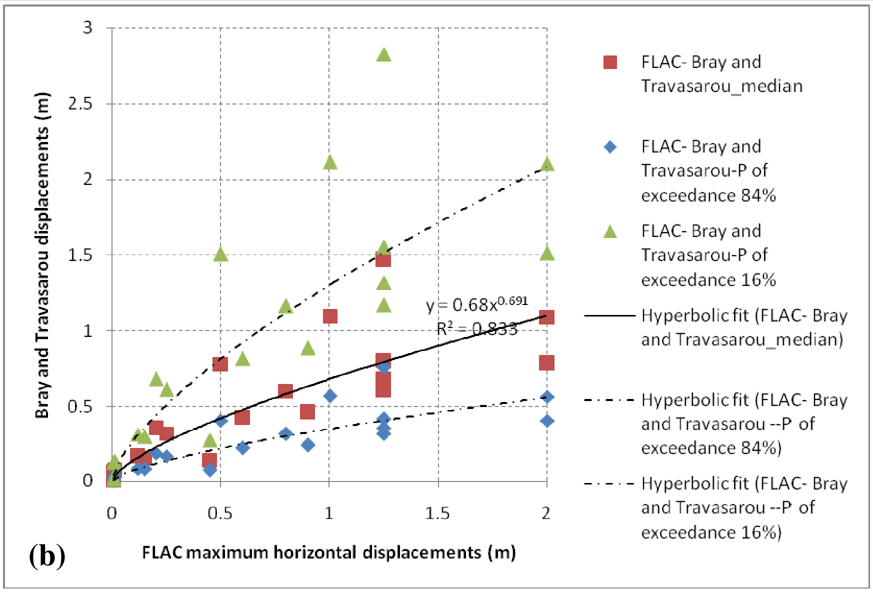


Figure 3. Comparison of Newmark (a) and Bray & Travasarou (2007) (b) displacements with maximum horizontal displacement from 2D dynamic analyses.

Numerical analysis of the building's response

The analyses of the buildings is conducted using the finite element code SeismoStruct (Seismostruct, Seismosoft 2005), which is capable of predicting the large displacement behavior of space frames under static or dynamic loading, taking into account both geometric nonlinearities and material inelasticity. Both local (beam-column effect) and global (large displacements/rotations effects) sources of geometric nonlinearity are automatically taken into account. The spread of material inelasticity along the member length and across the section area is represented through the employment of a fiber-based modeling approach, implicit in the formulation of SeismoStruct's inelastic beam-column frame elements. Static time-history analyses are performed for all numerical simulations. In this analysis type, the applied loads (displacements) at the foundation level vary in the pseudo-time domain, according to a load pattern prescribed as the differential permanent (ground or beam) displacement (versus time) curves directly extracted from the FLAC dynamic analysis.
Two reference single bay-single story RC buildings are considered that vary only in the foundation system: buildings with flexible foundation system (isolated footings) and buildings stiff but not completely rigid foundation system (continuous uniform loaded foundation of finite stiffness characteristics). The material properties assumed for the members of the reference RC buildings are described below. A uni-axial nonlinear constant confinement model (fig. 4 (a)) is used for the concrete material (fc=20MPa, ft=21MPa, strain at peak stress 0.002mm/mm, confinement factor 1.2), assuming a constant confining pressure throughout the entire stress-strain range (Mander et al., 1988). For the reinforcement, a uni-axial bilinear stress-strain model with kinematic strain hardening (fig. 4(b)) is utilized (fy=400MPa, E=200GPa, strain hardening parameter μ =0.005). This simple model is characterized by easily identifiable calibrating parameters and by its computational efficiency. The longitudinal reinforcement used is 8Φ14 (A=0.00123m^2) for all the cross sections considered. All columns and beams have rectangular cross sections (0.40x 0.40m). The reference building's height and length are 3m and 6m respectively.

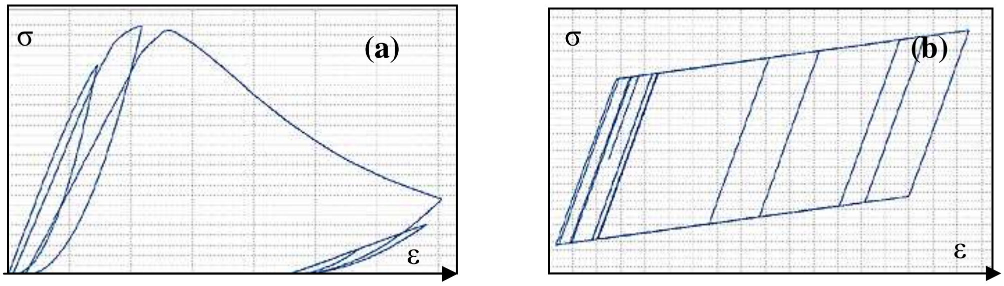
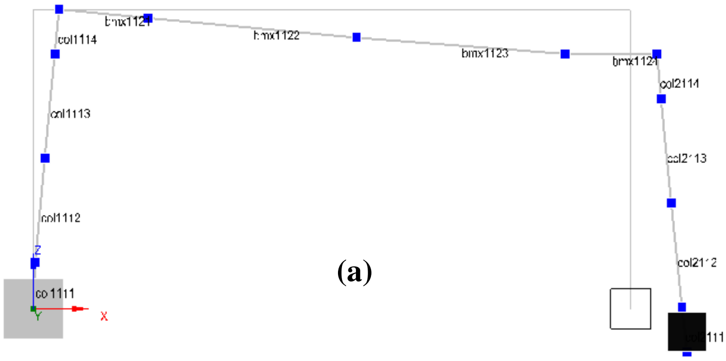


Figure 4. Stress-strain models for concrete (a) and steel (b).

A sensitivity analysis is performed for the two reference building cases by varying, in a range of reasonable values, the yield strength of steel (fy=210, 400, 500 MPa), the compressive (fc=16, 20, 30 MPa) and tensile (ft=2.0, 2.1, 3.0 MPa) strength of concrete, reinforcement bar size (Φ14, Φ16), the strain hardening parameter (μ=0.005, 0.01), the confinement factor (1.0, 1.2, 1.3) and the building height (H=3,4 m) and length (L=5,6m), together with the cross sections dimensions (30x30m, 40x40m, 50x50m), for progressively increasing levels of differential displacements extracted from the previous dynamic stress strain analysis for increasing level of input acceleration time histories. This analysis allows for indentifying the influence of different parameters on the structural response and proposing a preliminary probabilistic framework of the damage estimation.

The deformed shapes of buildings with flexible foundation system are essentially the same irrespective of the variability in the geometrical and strength parameters considered and the increase of displacement magnitude. The same trend is observed to the buildings with stiff foundation (Fig. 5). Moreover, in the case of buildings with flexible foundations, the applied differential displacement vector is mainly governed by the horizontal component (inclination angles vary from 37-38^{o} from the horizontal) that determines the deformation mode (fig. 5(a)). On the contrary, in buildings with stiff foundation system the applied displacements are practically vertical. Hence, it is concluded that the inclination of the applied differential permanent displacement constitutes a fundamental parameter in determining the deformed shape of the building when subjected to a permanent displacement at the foundation level.



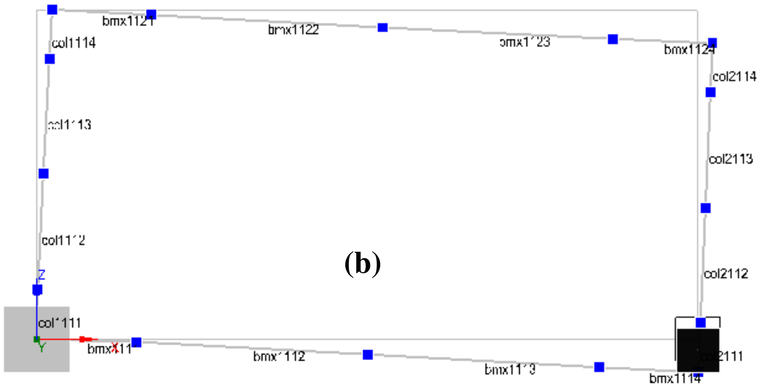


Figure 5. Deformed shapes for buildings with flexible (a) and stiff (b) foundations

Analytical fragility curves

Definition of damage indexes and limit states

We derived in this stage a set of analytical fragility curves for single bay- single storey RC buildings with varying stiffness of the foundation system. Each curve provides the conditional probability of exceeding a certain limit or damage state under a range of seismic induced landslide events of given type and intensity. The landslide intensity is expressed in this work in terms of peak ground acceleration at the seismic bedrock that is the initial trigger of the slow moving slide. This will result to permanent differential displacements at the foundation level.

The probabilistic nature of the problem is treated by accounting for the variability associated with the building capacity (yield strength of steel, compressive and tensile strength of concrete, reinforcement bar size, strain hardening parameter, confining factor, building height and length and cross sections dimensions), as well as the variability in the demand, assuming different progressively increasing acceleration time histories that result in different permanent differential displacement magnitudes at the building's foundation links. In order to identify the building performance (damage) state and to construct the corresponding fragility curves, a damage index (DI) is introduced describing the steel and concrete material strains. Within the context of a fibre-based modelling approach, such as that implemented in SeismoStruct, material strains do usually constitute the best parameter for identification of the performance state of a given structure (Seismostruct, Seismosoft 2005). In all cases analyzed (480 in total), the steel material strain (ε_s) yields more critical results. Thus, it was decided to adopt only this parameter as a damage index hereafter for simplicity reasons. In this way, it is possible to establish a relationship between the damage index (ε_s) and the input motion intensity in terms of the PGA values at the assumed seismic bedrock, for the two different building typologies and consequently to assign a median value of PGA to each limit state.

The next step is the definition of the damage or limit states. Based on the work of Bird and co-workers (Bird et al., 2005; Bird et al., 2006) and engineering judgment, 4 limit states (SL1, SL2, SL3, SL4) are defined as presented in Table 3. These concern exceedance of minor, moderate, extensive and complete damage of the building. The first limit state is specified as steel yielding that is the ratio between yield strength and modulus of elasticity of the steel material.

Table 3. Definition of Limit states for RC buildings

Limit state	Steel strain (ε_s)
LS1	Steel bar yielding: 0.0011-0.0025
LS2	0.0125

LS3	0.04
LS4	0.06

Construction of fragility curves

In order to construct the fragility relationships, appropriate cumulative distribution functions, as the ones proposed in HAZUS (NIBS, 2004), that describe the fragility relationships have been generated. For structural damage, given peak ground acceleration PGA, the probability of exceeding a given limit state, SLi, is modeled as:

$$f(PGA) = \Phi\left[\frac{1}{\beta_i} In\left(\frac{PGA}{\overline{PGA_i}}\right)\right] \quad (2)$$

Where:

- Φ is the standard normal cumulative distribution function,
- $\overline{PGA_i}$ is the median value of peak ground acceleration at which the building reaches the limit state, i,
- βi is the standard deviation of the natural logarithm of peak ground acceleration for limit state, i.

The median values of peak ground acceleration that correspond to each limit state can be defined for the threshold values of the aforementioned damage indexes as the values that corresponds to the 50% probability of exceeding each limit state. The standard deviation values [β] describe the total variability associated with each fragility curve. Three primary sources contribute to the total variability for any given damage state (NIBS, 2004), namely the variability associated with the definition of the limit state value, the capacity of each structural type and the demand (seismic demand, landslide type, relative position of the structure to the landslide). Based on the work of Crowley et al. (2004), Bird et al. (2006) and HAZUS (NIBS, 2004) prescriptions, the uncertainty in the definition of limit states and the capacity are assumed to be equal to 0.4 and 0.25 respectively for both building typologies (with flexible and stiff foundation system) considered. The last source of uncertainty associated with the demand, is taken into consideration by calculating the variability in the results of numerical simulation carried out in FLAC 2D for the different input motions at each level of PGA in the range of 0.1g to 0.9g applied at the base of the dynamic model (5 levels in total). It should be mentioned that this variability is different for the two different building types. In particular, it is higher in the case of the buildings with stiff foundation system. It is increased by 15% to account for the uncertainty in the landslide type (geometry, soil properties) and the relative location of the building to the potential unstable slope. The total uncertainty is estimated as the root of the sum of the squares of the component dispersions. The median (expressed in terms of peak ground acceleration PGA) and beta values of each limit state for the building with flexible and stiff foundation system are shown in Table 4.

Table 4. Parameters of fragility functions

Limit State / Building type	Median PGA (g)				β_i
	SL_1 (g)	SL_2 (g)	SL_3 (g)	SL_4 (g)	
Flexible building	0.26	0.42	0.76	>0.9	0.73
Stiff building	0.34	0.60	>0.9	>0.9	0.93

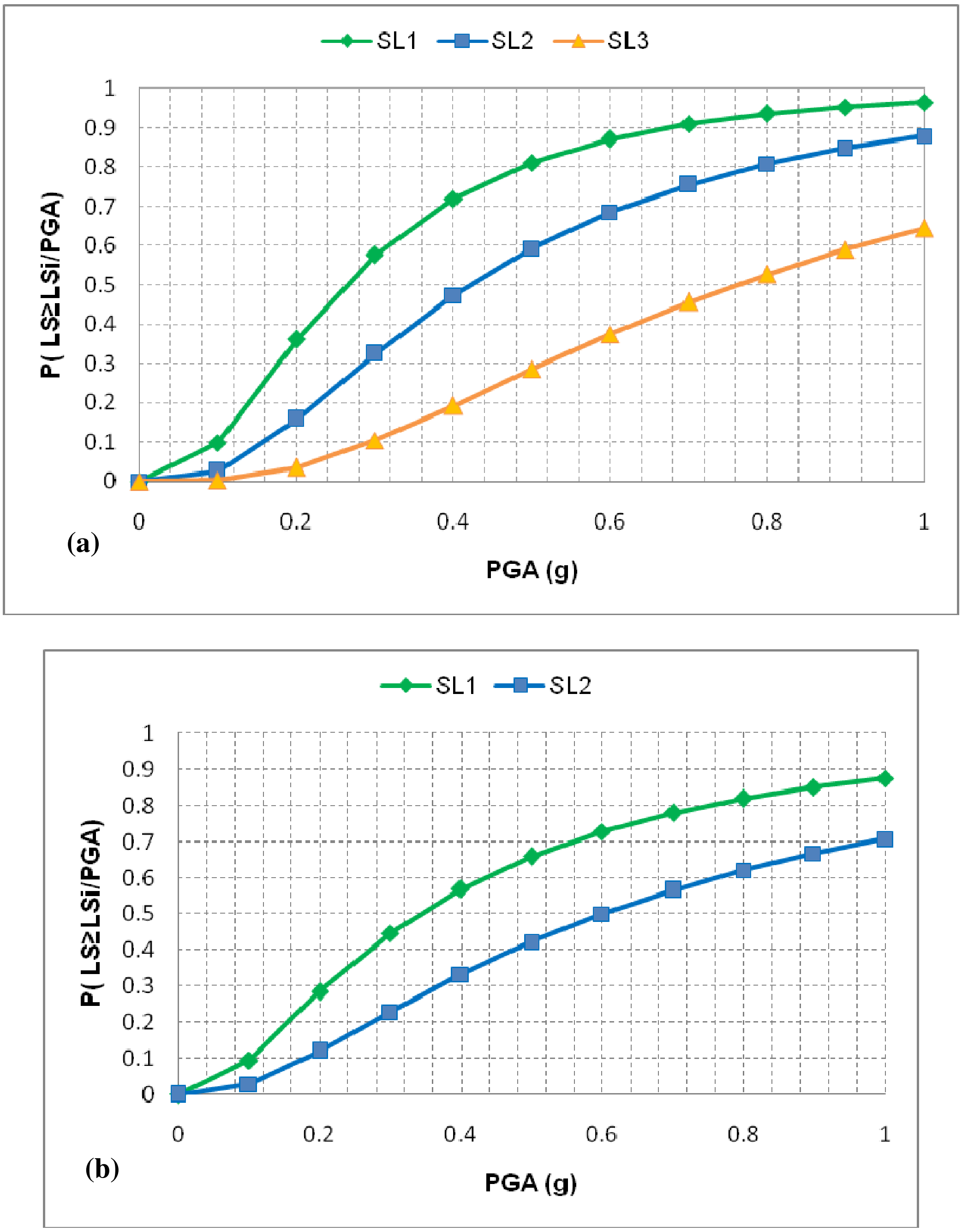


Figure 6. Fragility curves for one bay- one storey RC buildings with flexible (a) and stiff (b) foundation system

Figure 6 illustrates the derived fragility curves for the two building typologies. As expected, the building with stiff foundation system sustain less damage due to earthquake induced slow moving slides compared to the building with the flexible foundation system. More specifically, only minor and moderate damages are possible for the former for the specified levels of deformation. It should be noticed that only the structural damage of the building members is considered in this study. The total damage (structural and non-structural) will be quite different (certainly larger) in case of the building with the stiff foundation as a considerable amount of damage may be attributed to the rotation of the whole building as a rigid body. In the latter, the damage can only be defined using empirical criteria and expert opinion. Finally, it is worth pointing out that the influence of ground shaking to the building response is not considered in this study. The authors are planning to include this in a future work.

CONCLUSIONS

An analytical methodology to estimate physical vulnerability of RC buildings due to earthquake induced slow moving slides has been presented. The analysis results to the construction of fragility relationships for two reference single bay-single story RC buildings that differ only in the foundation system (isolated footing and continuous foundation). Various uncertainties, related to the capacity of the building, the deformation demand and the definition of limit states are considered in the analysis. It is observed that buildings with stiff foundation system are expected to suffer less structural damage compared to the buildings with flexible foundations.

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