COMPDYN 2011
III ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, M. Fragiadakis, V. Plevris (eds.)
Corfu, Greece, 25–28 May 2011

AN ANALYTICAL APPROACH FOR THE VULNERABILITY ASSESSMENT OF RC BUILDINGS SUBJECTED TO EARTHQUAKE INDUCED GROUND DISPLACEMENTS

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Keywords: fragility curves, seismically induced landslide displacements, RC buildings, foundation system, structural damage index, corrosion of reinforcement

Abstract. *The present study aims at the development of an efficient analytical methodology for the vulnerability assessment of RC buildings subjected to earthquake triggered slope movements. The vulnerability is defined through specific probabilistic fragility functions for specified limit states. The fragility curves are numerically estimated in terms of peak ground acceleration at the "seismic bedrock", versus the probability of exceedance of each limit state, for the considered structure types. A two -step, uncoupled approach is performed. In the first step, the differential permanent displacements at the building's foundation level are estimated using an adequate finite difference dynamic slope model. Properly selected and corrected acceleration time histories are applied at the base of the model to assess the building's foundation response and the associated ground and foundation displacements are computed accordingly. Then, the calculated differential displacements are imposed to the fiber-based building model at the foundation level 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. Various sources of uncertainty concerning the capacity of the building, the deformation demand and the definition of limit states are considered in the analysis. The developed methodology is applied to both "low code" and "high code" RC frame buildings resting on shallow foundations with varying strength and stiffness characteristics (isolated footings, continuous foundation), standing near the crest of a relative slow moving soil slide. In case of the "low code" (usually old) RC buildings, the effect of corrosion of the reinforcement on the vulnerability estimation is also considered. The final goal of this research is to propose efficient fragility functions for a variety of RC building typologies.*

1 INTRODUCTION

Major landslide events occurred in Taiwan, California, Japan, Italy, China and elsewhere represent some of the most pronounced collateral hazards associated with earthquakes in terms of human losses and direct and indirect damage to the built environment. Seismically induced permanent ground deformation occurring as a result of landslides can adversely affect the likely performance of vulnerable engineering structures as it may account for a significant proportion of the total earthquake damage. Thus, predicting the expected performance of precarious slopes and affected facilities within the unstable area during or after an earthquake event, is of primary importance for design, urban planning, and for seismic hazard and risk studies.

Various methods of different complexity have been proposed to assess earthquake induced landslide hazards including the estimation of the probability of occurrence of a landslide and the slope permanent co-seismic displacement along a slip surface using Newmark-type displacement methods or advanced numerical approaches. However little work has been done on the quantification of the physical vulnerability of structures affected by earthquake triggered landslides. A major constraint to this may be considered the scarcity of accurate and reliable information on seismically induced landslide damage. HAZUS [1] multi-hazard loss estimation methodology may be considered an exception. Separate fragility curves, distinguishing between ground failure due to lateral spreading and ground failure due to ground settlement, and between shallow and deep foundations, were generated considering one combined Extensive/Complete damage state. However, the aforementioned methodology, exclusively based on expert judgment, involves a high degree of subjectivity and simplification as it does not account for the various landslide types and mechanisms, the soil type, the building typology, the stiffness of the foundation and the different damage states.

The present study, recognizing the need to improve the available background, aims at the proposition and quantification of an analytical procedure to assess the vulnerability of RC structures as a consequence of earthquake induced landslide displacements. The final goal of this research is to propose adequate fragility functions for a variety of RC building typologies.

2 METHODOLOGY

The proposed methodology [2], largely inspired from the seismic risk analysis, may be applied for the vulnerability assessment of RC buildings subjected to earthquake triggered relative slow moving soil slides. The proposed approach is principally based on a comprehensive set of numerical computations and statistical analysis. In terms of numerical simulation, a two-step uncoupled analysis is conducted. In the first step, the differential permanent displacements at the building's foundation level are estimated using the FLAC2D [3] finite difference dynamic slope model. The effect of ground shaking to the structure's response and the associated damages is not considered in the vulnerability of the building itself, which is assessed only for the differential movements due to slow moving seismically triggered landslides. Then, the computed differential displacements are applied as input to the building model at the foundation level, to assess the building's response for different differential ground landslide displacements induced by earthquake with progressively increased intensities. The numerical analysis of the building's response is performed through the fiber-based finite element code SEISMOSTRUCT [4]. Limit states are defined in terms of a threshold value of building's material strain.

The fragility curves are numerically estimated in terms of peak ground acceleration at the "seismic bedrock", (PGA) versus the probability of exceedance of each limit state considering various sources of uncertainty. The selection of the PGA against the differential displacements value is a key point of the present method and it is explicitly related to the main parameter of any seismic hazard assessment.

Figure 1 illustrates a schematic representation of the proposed framework. 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.

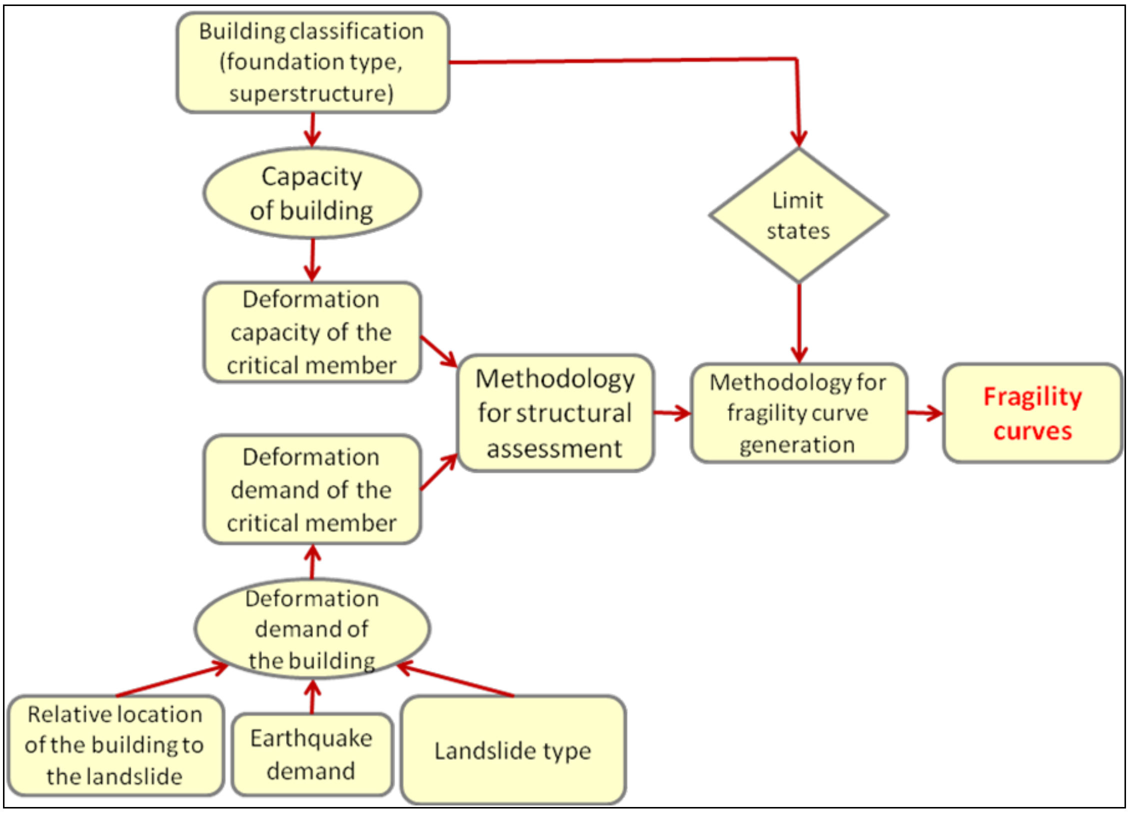


Figure 1: Flowchart for the proposed framework of fragility analysis of RC buildings.

The description of the methodological framework together with a simplified case study has already presented in [2]. In the present study an improved version of the methodology and the corresponding application is attempted, emphasizing at the variability on the earthquake ground motion and on the associated differential displacements, yielding a more descriptive characterization of the structural damage and considering buildings with different strength and stiffness characteristics and code design level. Some key points of the proposed framework are highlighted in the following paragraphs:

The landslide type is a crucial parameter of the proposed methodology as landslides of different types and sizes usually require different and complementary methods to estimate vulnerability. While damage to the built environment resulting from the occurrence of rapid landslides such as debris flows and rock falls is generally the highest and most severe, as it

may lead to the complete destruction of any structure within their path, slow-moving slides also have adverse effects on affected facilities [5]. The damage caused by a slow moving landslide on a building is mainly attributed to the cumulative permanent (absolute or differential) displacement and it is concentrated within the unstable area. A relative slow moving soil slide that will produce tension cracks due to differential displacement to a RC building, exposed to the landslide hazard, is considered in this study.
The characteristics (amplitude, frequency content and duration) of the earthquake ground motion in relation to the soil dynamic properties and stratigraphy can significantly influence the derived deformation demand for the building. Material damping, the impedance contrast between sediments and the underlying bedrock, and the characteristics of incident wavefield are considered to represent the governing factors for site amplification/attenuation [6,7]. A fundamental period of the earthquake close to the natural period of the site can lead to resonance phenomena and, consequently, to an amplified energy content of the ground motion. Combining a low-frequency seismic input motion together with a resonance phenomenon in the low-frequency range, the slope failure potential assumes its highest values [8].

The position of the building with respect to the landslide area is a very important contributing factor in estimating vulnerability. Landslides triggered by earthquakes tend to be clustered near ridge crests and hill slope toes. Peng et al. [9] 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 this study, a building standing near the crest where the seismic ground motion due to topographic effects is generally amplified is considered [10,11].

For a landslide of given type, mechanism and intensity, the typology of the exposed structure is also a key factor in the vulnerability assessment methodology. Geometry, material properties, state of maintenance, code design level, soil conditions, foundation and structure details, number of floors etc. are among typical typological parameters which determine the capacity of the building to withstand the specified co-seismic landslide displacement. The response to permanent total and differential ground deformation depends primarily on the foundation type. A structure on a deep foundation (e.g. piles) compared to shallow foundations often experiences higher resistance ability and hence a lower vulnerability. For shallow foundations, the distinction is between rigid or flexible/unrestrained foundation systems. When the foundation system is rigid (e.g. continuous raft foundation), the building is expected rather to rotate as a rigid body and a failure mainly attributed to the loss of functionality of the structure is anticipated. In this case, 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 (e.g. isolated footings), the various modes of differential deformation produce structural damage (e.g. cracks) to the building members [12,13] that can be estimated using an analytical procedure analogous to that of the response due to seismic ground motion.

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 [14,12].

In the probabilistic approach proposed 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.

3 APPLICATION

3.1 Deformation demand- Numerical analysis

An application of the proposed methodology to an idealized case study is presented herein. To estimate the input differential displacements at the building's foundation level, we applied FLAC 2D finite difference dynamic model [3] (fig. 2) using an elastoplastic constitutive model with Mohr-Coulomb failure criterion and non associated plastic flow rule, able to simulate large deformations for slope stability assessments. A small amount of Rayleigh damping (1 to 3%) is assigned 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 is adopted, whereas towards the lateral boundaries of the model, where the accuracy requirements loosen, the mesh is coarser. 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 a homogenous dry sand corresponding to soil category C of EC8 [15]; 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 with a width of 6m (uncoupled approach). 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. 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.

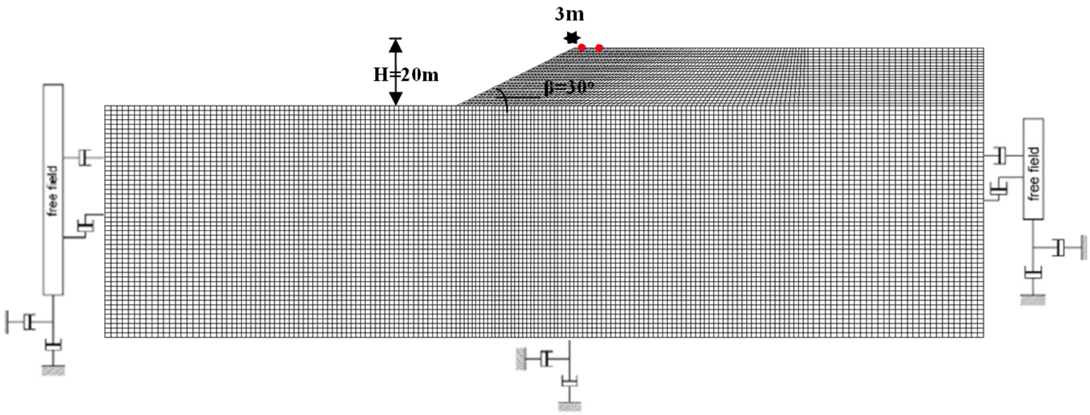


Figure 2: FLAC dynamic model

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
Dilation angle (degrees)	0
$N_{1(60)}$	21
Dr(%)	60

Table 1: Soil properties

Properties	Foundation system	
	Stiff foundation	Flexible foundation
Element	beam	
Length (m)	6	
Density (KN/m^3)	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

Table 2: Foundation properties

Prior to the dynamic simulations, a static analysis is carried out to establish the initial effective stress field throughout the model. The dynamic input motion consists of SV waves vertically propagating from the base. Six different earthquake records are used as excitation for the dynamic analysis: (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, (v) Campano Lucano (Sturno-L), Italy, Mw=6.9 , 1980 and (vi) Duzce (L), Turkey, Mw=7.2, 1999. They all refer to outcrop conditions. The selected records cover a wide range of seismic motions in terms of the seismotectonic environment, amplitude, frequency content and significant duration. Before applied along the base of the model, they are subjected to appropriate correction (baseline correction, filtering and tapering) to assure an accurate representation of wave transmission through the model. Figure 3 presents the normalized elastic response spectra of the input motions together with the proposed elastic design spectrum of EC8 [15] for soil type A (rock).

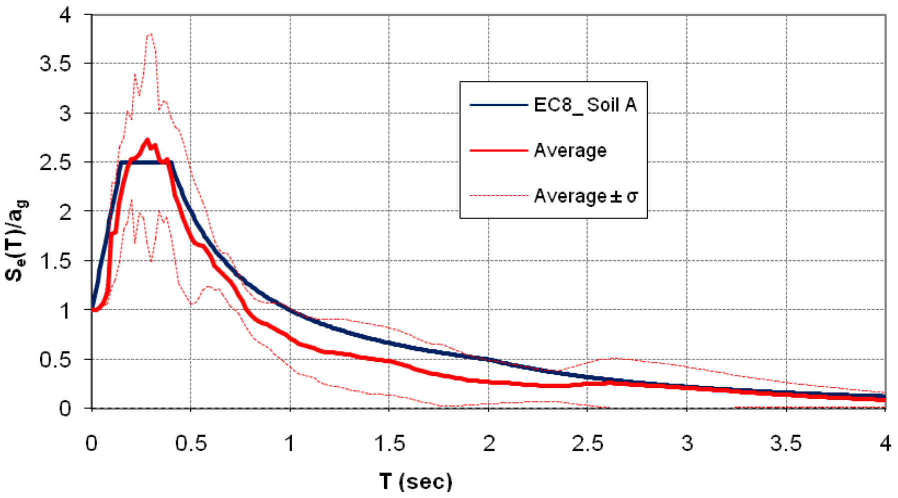


Figure 3: Normalized average elastic response spectrum of the input motions in comparison with the corresponding elastic design spectrum for soil type A (rock) according to EC8.

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. Figure 4 presents the maximum values of differential displacements for the building with flexible and stiff foundation system derived from the dynamic analysis by applying the different scaled accelerograms. It is observed that the specific characteristics (frequency content and duration) of the seismic ground motions can significantly influence the magnitude of the computed differential displacement at the foundation level. Moreover, 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.

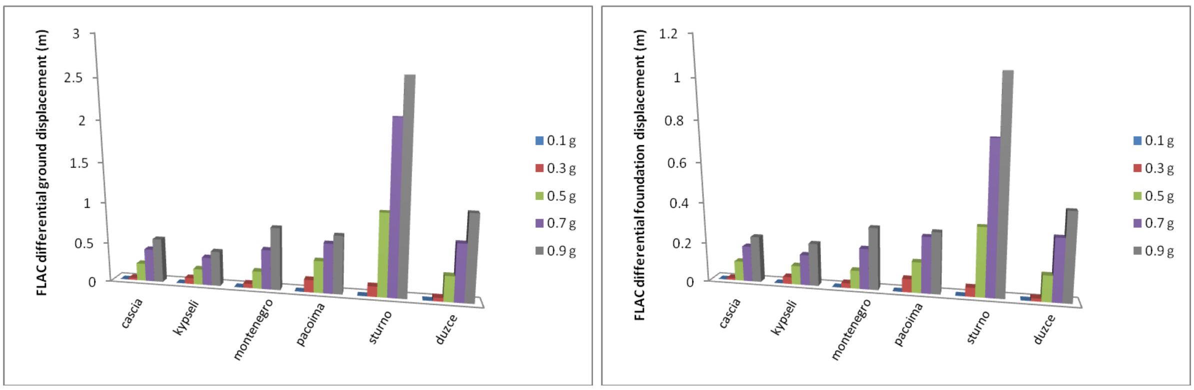


Figure 4: Maximum values of differential displacement vector for buildings with flexible and stiff foundation system

3.2 Comparison with simplified displacement methods

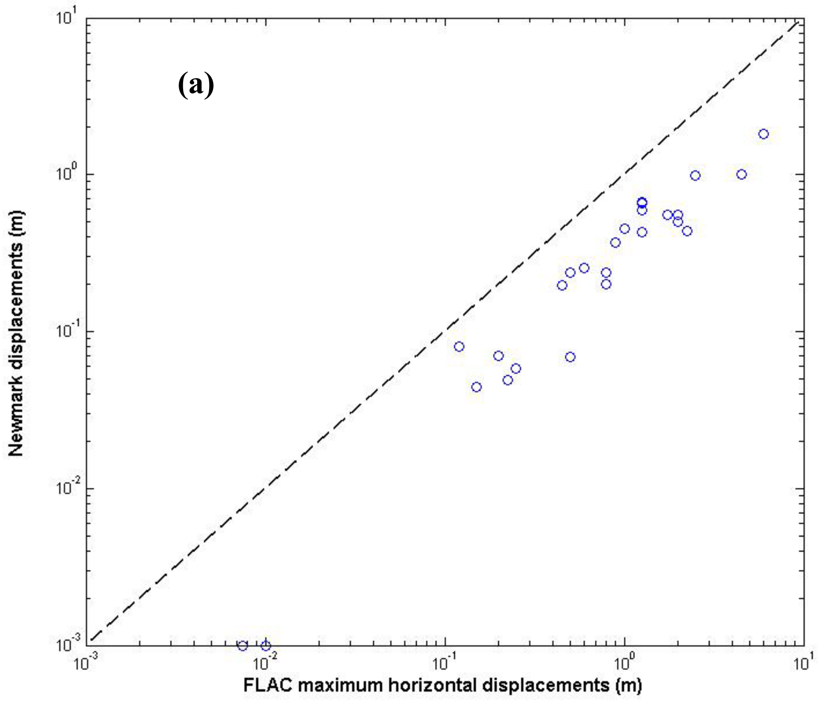
To validate the numerical results, they are compared, in terms of maximum permanent horizontal displacement, with the simplified Newmark-type displacement methods. The conventional Newmark rigid block model [16,17], as well as a recent improved version to account for the soil deformability using a coupled stick-slip deformable sliding block model [18], are used to calculate permanent displacements of the slide mass.

Bray and Travasarou [18] 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 scaled acceleration time histories recorded on rock multiplied by the site amplification factor S=1.15 (as proposed in EC8 for subsoil class C), in an effort to conservatively approximate the equivalent acceleration time histories acting on the potentially sliding mass. The yield coefficient, k_y, is computed by applying the following relationship, as proposed in [19]:

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

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 5 (a) and (b) respectively in comparison with the co-seismic numerical displacements calculated herein. The direct application of Newmark rigid block approach is found to underestimate the computed displacements. This can be regarded as relevant considering that Newmark's method treats the potential landslide block as a rigid mass (no internal deformation) 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 and Travasarou [18] are generally in good agreement with that of the dynamic analysis. In both cases, however, a large scatter on the predicted residual displacements is detected recognizing the need to adopt a fully probabilistic framework, as proposed in Bray and Travasarou [18].



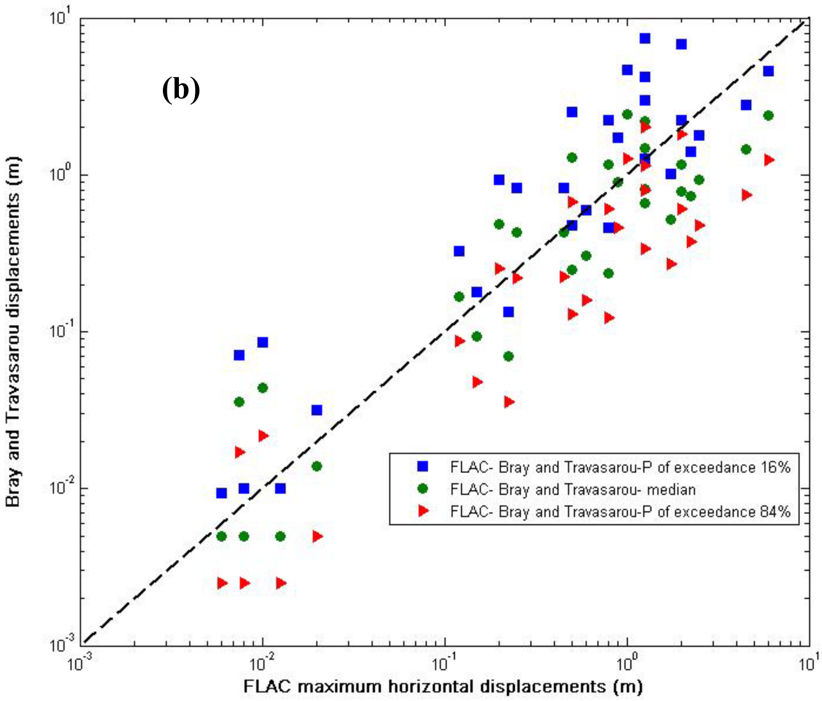


Figure 5: Comparison of Newmark [16] (a) and Bray and Travasarou [18] (b) displacements with maximum horizontal displacement from 2D dynamic analyses.

3.3 Analysis of the building's response

The analyses of the buildings is conducted using the finite element code SeismoStruct [4], which is capable of calculating 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. Nonlinear static time-history analyses are performed for all numerical simulations. In particular, the differential permanent (ground or beam) displacement (versus time) curves, directly extracted from the FLAC dynamic analysis, are statically imposed at one of the RC frame supports.

The studied buildings are single bay-single storey RC bare frame structures that vary in their foundation system and design code level. Regarding the foundation system, buildings with flexible foundation system (isolated footings) and buildings with stiff but not completely rigid foundation system (continuous uniform loaded foundation of finite stiffness characteristics) are considered. Both "low code" and "high code" design buildings that differ in the strength and stiffness characteristics and the corresponding assigned limit strain levels are examined. The reference building's height and length are 3m and 6m respectively. All columns and beams have rectangular cross sections (0.40x 0.40m). The longitudinal reinforcement used is 8Φ14 ($A=0.00123m^2$) for all the cross sections considered.

The use of such simple structures is justified from the observation that the number of storeys and bays do not seem to comprise crucial parameters in the determination of the building's performance subjected to permanent ground displacements. The latter is also confirmed in [12] for the vulnerability assessment of RC buildings due to liquefaction induced ground deformations. Hence, one bay-one storey RC structures despite their simplicity are found to be adequately representative of the performance of real low rise RC frame buildings.

The material properties assumed for the members of the reference RC buildings are described below. A uni-axial nonlinear constant confinement model (fig. 6 (a)) is used for the concrete material (fc=20MPa, ft=2.1MPa, strain at peak stress 0.002mm/mm, confinement factor 1.2), assuming a constant confining pressure throughout the entire stress-strain range [20]. For the reinforcement, a uni-axial bilinear stress-strain model with kinematic strain hardening (fig. 6(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.

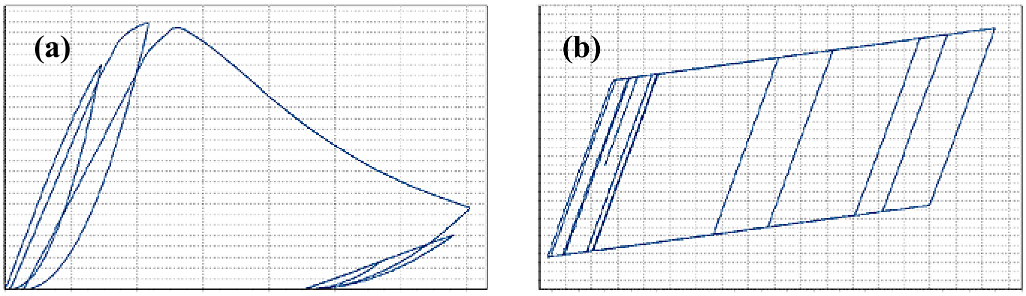


Figure 6: Stress-strain models for concrete (a) and steel (b).

A sensitivity analysis is performed for the reference building cases which allows for indentifying the influence of different parameters on the structural response and proposing a preliminary probabilistic framework of the damage estimation. The parameters selected to vary are: the yield strength of steel (fy=210, 400 MPa for “low code” and 400,500 MPa for “high code”), the compressive (fc=16, 20 MPa for “low code” and 20, 30 MPa for “high code”) and tensile (ft=2.0, 2.1 MPa for “low code” and 2.1, 3.0 MPa for “high code”) strength of concrete, the reinforcement bar size (Φ12, Φ14 for “low code” and Φ14, Φ16 for “high code”) and the confinement factor (1.0, 1.2 for “low code” and 1.2, 1.3 for “high code”) for progressively increasing levels of differential displacements extracted from the previous dynamic stress strain analysis for increasing level of input acceleration time histories. In case of the “low code” (usually old) RC buildings, the effect of corrosion of the reinforcement on the vulnerability estimation is also considered. The loss of area of steel due to corrosion of the RC elements is modeled as a reduction in longitudinal reinforcing bar cross sectional area compared to the elements in the initial nondegraded state. In this study, a 50% reduction of the area of reinforcing steel is assumed.

The deformed shapes of buildings with flexible foundation system are essentially the same irrespective of the variability in the strength parameters and the level of demand, observation that is in accordance with that of [12]. The same trend is observed to the buildings with stiff foundation (Fig. 7). In both building typologies, a column failure mechanism is detected (see also [21]). The reason is that the axial stiffness of the beams is generally much higher compared to the flexural stiffness of the columns. Moreover, in the case of buildings with flexible foundations, the applied differential displacement vector is mainly governed by the horizontal component that determines the deformation mode (fig. 7(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. It is worth noting that the deformed shape of building with flexible and stiff foundations and the associated computed displacements at the critical column are found to closely approximate the idealized response of a RC frame building subjected to liquefaction induced lateral movement (with an associated vertical component) and to vertical settlements respectively, as proposed in [12].

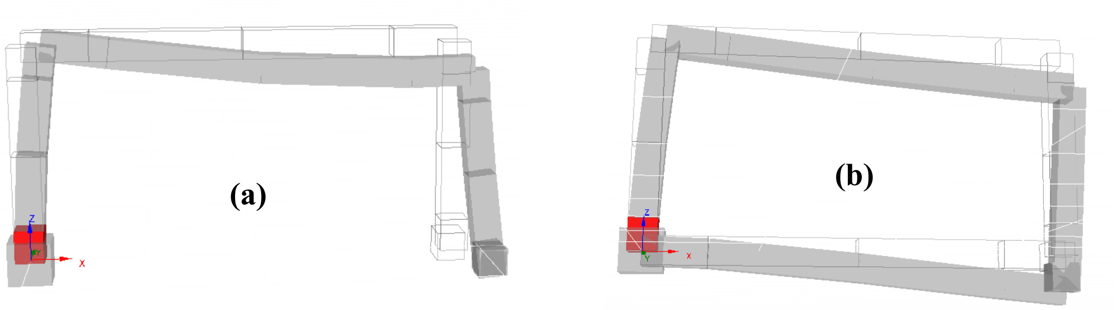


Figure 7: Deformed shapes for buildings with flexible (a) and stiff (b) foundations. The failure mechanism at the critical column is also shown.

3.4 Analytical fragility curves

We derived in this stage different sets of analytical fragility curves for low rise (single bay-single storey) RC buildings with varying stiffness of the foundation system and different code design level. Fragility curves for "low code" corroded RC buildings are also constructed. 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, confining factor), 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 [4]. In all cases analyzed (600 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 different building typologies and consequently to assign a median value of PGA to each limit state. Figure 8 presents representative PGA - damage index curves for low rise, "high code" design RC frame buildings with stiff and flexible foundation system.

The next step is the definition of the damage or limit states. Based on the work of Crowley et al. [14], Bird et al. [12,13] and engineering judgment, 4 limit states (LS_1, LS_2, LS_3, LS_4) are defined. Considering that low code RC buildings are poorly constructed structures characterized by a low level of confinement, the limit steel strains needed to exceed post yield limit states should have lower values compared to high code, properly constructed RC buildings. As a consequence, it was decided to adopt different limit state values for excedance of extensive and complete damage for low and high code frame RC buildings. A qualitative description of each damage band for reinforced concrete frames is given in Table 3 while the limit state values finally adopted in quantitative terms are presented in Table 4. 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.

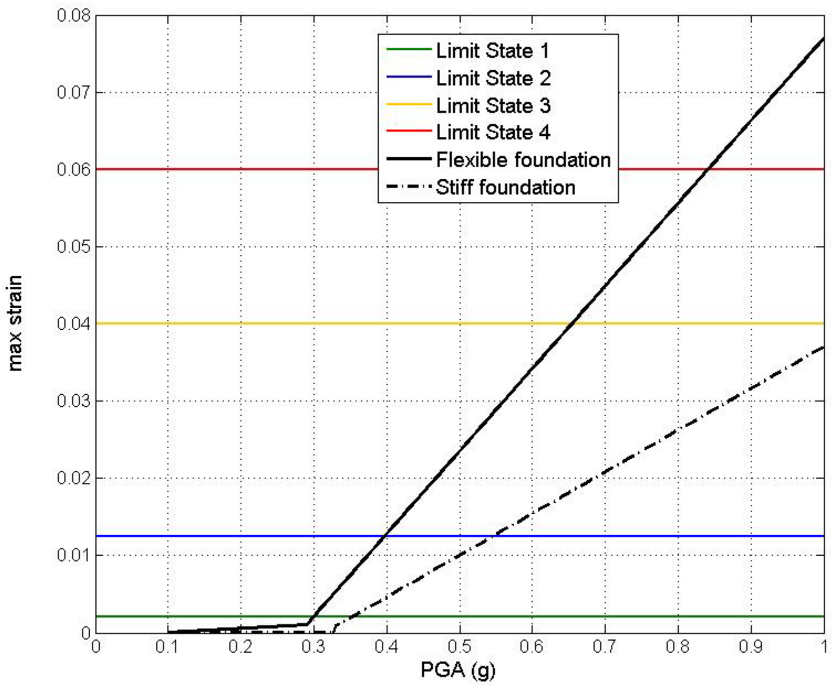


Figure 8: PGA–damage index relationships for 1story-1story “high code” RC frame buildings with stiff and flexible foundation system

Structural damage band	Description
None to slight	Linear elastic response, flexural or shear type hairline cracks (<1.0 mm) in some members, no yielding in any critical section
Moderate	Member flexural strengths achieved, limited ductility developed, crack widths reach 1.0 mm, initiation of concrete spalling
Extensive	Significant repair required to building, wide flexural or shear cracks, buckling of longitudinal reinforcement may occur
Complete	Repair of building not feasible either physically or economically, demolition after earthquake required, could be due to shear failure of vertical elements or excessive displacement

Table 3: Structural damage state descriptions for RC frame buildings (after [14])

Limit state	Steel strain (ε_s) –low code	Steel strain (ε_s) – high code
LS1	Steel bar yielding	Steel bar yielding
LS2	0.0125	0.0125
LS3	0.025	0.04
LS4	0.045	0.06

Table.4. Definition of Limit states for “low” and “high” code design RC buildings

In order to construct the fragility relationships, appropriate cumulative distribution functions, as the ones proposed in HAZUS [1], 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:

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

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 [1], 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 [14], Bird et al [13] and HAZUS [1] prescriptions, the uncertainty in the definition of limit states, for all building types and limit states, is assumed to be equal to 0.4 while the variability of the capacity is assumed to be $\beta = 0.3$ for “low code” and $\beta = 0.25$ for “high code” buildings. 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 applied at the base of the dynamic model. 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 flexible foundation system. 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 5.

Figures 9-11 illustrate the derived sets of fragility curves for the different building typologies, design code level and considering the effect of rebar corrosion for an assumed deterioration scenario in case of “low code” buildings. 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 “high code” stiff buildings while minor, moderate and extensive damage are expected for “low code” stiff buildings for the specified levels of landslide intensity. Low code, poorly designed and constructed buildings are associated with a rapid transition from low levels of damage to collapse. This can be seen from the closeness of the four limit state curves and the steepness of their slopes. In contrast, well-designed buildings generally present a more ductile behavior, allowing for larger distinction between the curves. The consideration of corroded structural members in case of low code RC buildings reveals a significant increase in the system's vulnerability. This is more evident for buildings with flexible foundations and low levels of damage.

Foundation type	Design level	Median PGA (g)				β_i
		LS_1 (g)	LS_2 (g)	LS_3 (g)	LS_4 (g)	
Flexible	High	0.3	0.4	0.67	0.85	0.8
	Low	0.3	0.395	0.51	0.69	0.81
	Low_corroded	0.2	0.365	0.455	0.59	0.81
Stiff	High	0.36	0.55	>0.9		0.74
	Low	0.345	0.525	0.74	>0.9	0.76
	Low_corroded	0.25	0.5	0.71		0.76

Table 5: Parameters of fragility functions

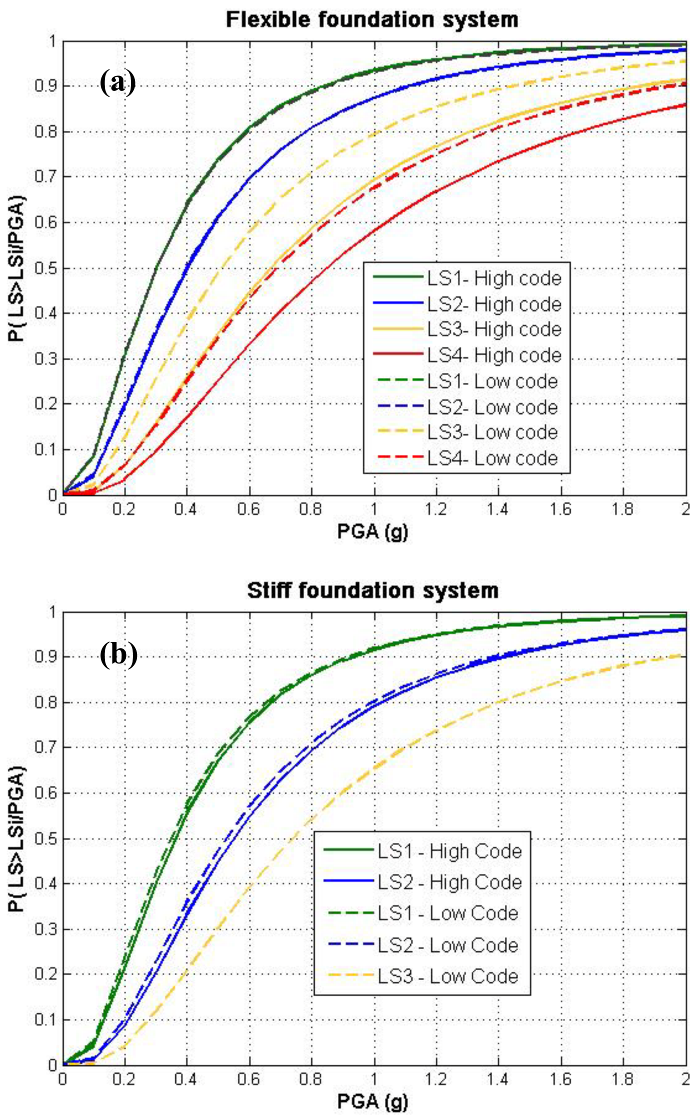
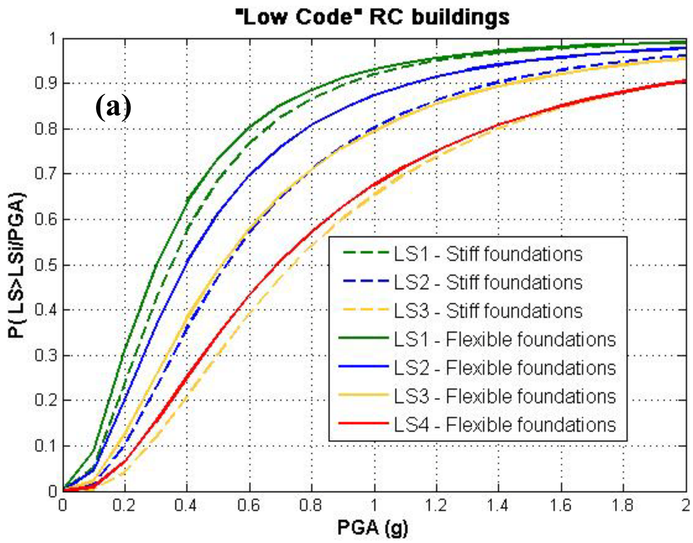


Figure 9: Fragility curves for low rise RC buildings with flexible (a) and stiff (b) foundation system



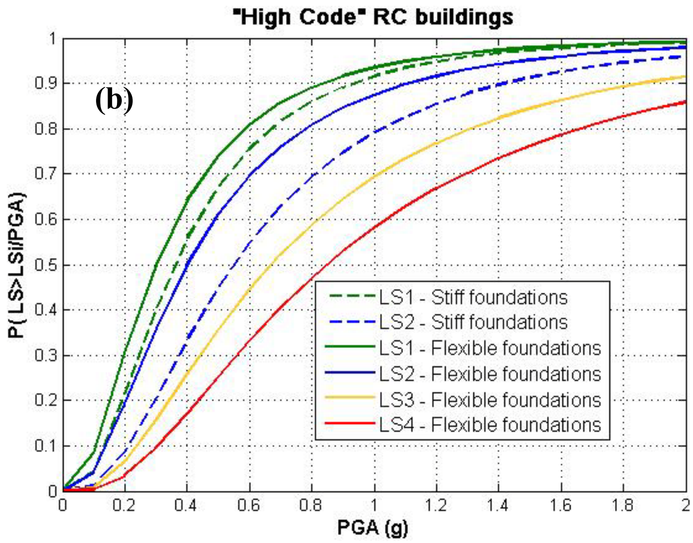
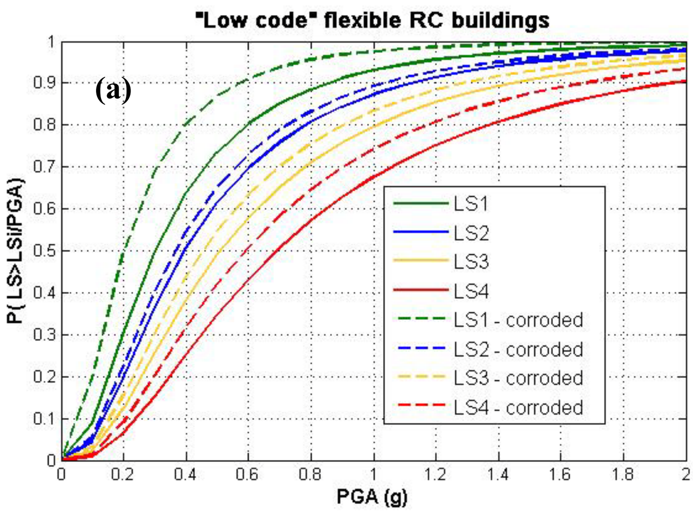


Figure 10: Fragility curves for low rise RC buildings, “low code” (a) and “high code” (b) design



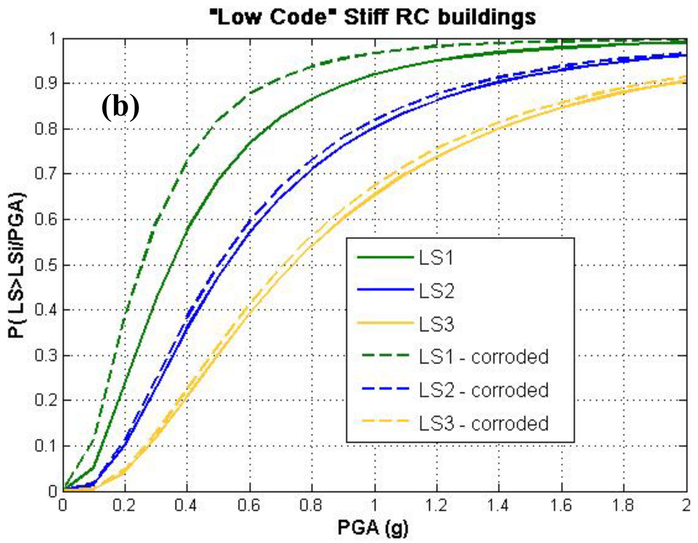


Figure 11: Fragility curves for low rise "low code" flexible (a) and stiff (b) RC buildings

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 [12]. Furthermore, it is worth pointing out that the complex issue of combined ground shaking and ground failure due to landslide is not taken into account in the evaluation of the building 's vulnerability. The authors are planning to include this in a future work. Finally, although the results are limited by some of the idealizations and assumptions of the analysis, they should provide a useful starting point in the vulnerability assessment of affected buildings standing near the crest of precarious slopes, providing the basis for more sophisticated numerical analysis for the particular governing conditions in selected real case studies.

4 CONCLUSIONS

An analytical-numerical methodology to estimate physical vulnerability of RC buildings subjected to co-seismic landslide displacements has been presented. The analysis results to the construction of fragility relationships for single bay-single story RC buildings that differ in the foundation system (isolated footing and continuous foundation) and code design level. For low code, usually old, structures the effect of corrosion of the reinforcement on the vulnerability estimation is also investigated. 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 "high code" buildings with stiff foundation system are expected to suffer less structural damage compared to the other building typologies examined. The performance of "low code" RC structures is found to be degraded due to the consideration of corroded structural members, revealing a substantial increase in the vulnerability of buildings resting on flexible foundations. Finally the proposed fragility curves can be used to estimate the seismic risk and to design appropriate mitigation measures at building or aggregate scale.

AKNOWLEDGEMENTS

The work described in this paper was supported by the project SafeLand "Living with landslide risk in Europe: Assessment, effects of global change, and risk management strategies" under Grant Agreement No. 226479 in the 7th Framework Programme of the European Commission. This support is gratefully acknowledged.

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