A simplified component-based methodology for the seismic vulnerability assessment of school buildings using nonlinear static procedures: application to RC school buildings

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Abstract:

Several earthquakes have affected school infrastructure, compromising the safety of students and all the educational community. These damages are not caused solely by the action of earthquakes, but also by the lack of adequate seismic design, deficient construction practices, and lack of regulations and normative to ensure an appropriate quality for infrastructure. Therefore, to analyze how is the expected infrastructure behavior in earthquakes, this study presents a simplified methodology for the seismic vulnerability assessment of school buildings. The methodology includes several components: data collection, the characterization of Index Buildings (IB), hazard definition, nonlinear numerical modeling of the structural response, seismic performance assessment and the vulnerability integration using a component-based approach. The novelty of the proposed methodology resides in the fact of its simplicity and robustness obtained by combining a simplified non-linear incremental static analysis together with a component-based vulnerability derivation methodology to assess the behavior of school buildings. This methodology is applied to a set of 11 Reinforced Concrete (RC) school building types representing common structural systems and seismic design levels. A number of sensitivity analyses are also carried out, varying the geometry, the foundation-soil flexibility, the mechanical properties of infill masonry walls, the non-structural elements and the analysis type, showing the versatility and reliability of the proposed methodology.

Key words: seismic vulnerability, non-linear static procedures, reinforced concrete buildings, school buildings

1 Introduction

Quality education is a priority established in the Sustainable Development Goals (SDG) in which the fourth goal corresponds to "Ensure inclusive and equitable quality education and promote lifelong learning opportunities for all" (United Nations 2015). One of the most important factors to achieve this goal is to ensure safer infrastructure, which is widely promoted by multilateral agencies through programs at the regional level with various objectives. Indeed, UNESCO has emphasized infrastructure safety by developing VISUS (UNESCO 2019), a methodology for assessing safety attributes in school facilities with applications in El Salvador, Laos, Indonesia, and Peru. The UNISDR (the United Nations Office for Disaster Risk Reduction), in collaboration with the Global Alliance for Disaster Risk Reduction & Resilience in Education Sector (GADRRRES), has also developed programs and action plans such as the Comprehensive School Safety program and the Worldwide Initiative for Safe Schools (UNDRR 2017). These two initiatives provide a global framework to support activities related to safe learning facilities, school disaster risk management, and risk reduction and resilience in education. Moreover, since 2014, the Global Program for Safer Schools (GPSS) of the World Bank is actively engaged in developing roadmap and guidelines, as well as assisting governments in developing countries, to reduce the disaster risk to school infrastructure. The main purpose of GPSS is to boost large-scale investments to enhance the safety and resilience of school infrastructure at risk from natural hazards and contribute to improving the quality of learning environments for children (The World Bank 2019).

School safety is threatened by several natural hazards such as earthquakes, cyclones, floods, wildfires, and landslides, imposing risk for children. Among these hazards that impact school infrastructure, earthquakes pose the greatest risk and thus the present study focuses on seismic vulnerability. Some of the most recent examples of damages due to earthquakes evidenced in Latin American infrastructure can be found in the Geotechnical Extreme Events Reconnaissance (GEER) reports for Ecuador 2016 and Mexico 2017 (GEER 2016, 2017). As reported by Earthquake Engineering Research Institute (EERI), earthquakes have also caused vast damage in other regions of the world, such as India, Indonesia, Peru, and Turkey (EERI 2019). In particular, in Nepal, the 2015 Gorkha earthquake showed the high vulnerability of school building with different structural systems, such as reinforced concrete frames, cement-bonded and mud-bonded masonry, and timber frames among others (Chen et al. 2017). However, it is essential to note that earthquakes themselves have not caused the disaster, but this occurred primarily due to the lack of adequate construction practices through regulations and infrastructure of appropriate quality (IADB 2014; UNDRR 2017; Nassirpour et al. 2018; The World Bank 2019). Based on the underlying weakness, or vulnerability condition of children due to their age and response capacities, governments have a direct responsibility in reducing the physical vulnerability of school infrastructure (D'Ayala et al. 2020).

The first step towards reducing the structural vulnerability of school infrastructure is the assessment of its structural and damage behavior to understand the expected performance during possible earthquakes. For this, simplified methodologies to develop safety index can be implemented such as the Rapid Visual Screening (RVS) method proposed by Ruggieri et al (2020). In this method, the authors propose to obtain in-field data related to structural and non-structural components, organizational characteristics, and the number of occupants to generate a composite safety index. RVS approaches however have limited applicability when needing to identify strengthening strategies for specific typologies. In this respect fragility functions and vulnerability functions, derived on the basis of analytical models, are more suitable tools for assessing the seismic safety of buildings (Masi 2003; D'Ayala 2013; Michel et al. 2014). Fragility functions represent the probability of exceeding a damage state of a specific structure type given the hazard intensity measure (D'Ayala et al. 2015). On the other hand, vulnerability functions represent the overall probability of damage for a structure, expressed such as the Mean Damage Ratio (MDR), and its variance given a hazard intensity measure, such as the Peak-Ground Acceleration (PGA) or the Spectral Acceleration (S_a) (Yamin et al. 2014). The MDR is usually expressed in economic terms, as the ratio of the expected total repair cost to the building's total replacement cost (Yamin et al. 2017). The building's total replacement cost is defined as the actual reconstruction cost of the building according to local price conditions in the region under analysis. Most common hazard intensity measures for seismic fragility and vulnerability assessment are the PGA or $S_a(T_1)$, while other intensity measures such as Spectral Displacement (S_d) are also used in practice. The choice of IM depends on the typology of the building under assessment (Yamin et al. 2017).

Recent studies have proposed diverse yet independent or isolated methodologies for the seismic Vulnerability or Fragility (V/F) assessment of representative buildings (Dolšek and Fajfar 2005; Dolšek 2012; Abo-El-ezz et al. 2013; D'Ayala et al. 2015; Del Gaudio et al. 2015; Hosseinpour and Abdelnaby 2017; Yamin et al. 2017; Cremen and Baker 2019). Approaches may consider empirical, expert opinion-based, analytical or hybrid methods to derive vulnerability or fragility functions (Porter et al. 2002; D'Ayala et al. 2015; Silva et al. 2018). The analytical vulnerability approach allows for an unbiased and consistent assessment that has proven to be applicable worldwide, independently of historical seismic damage data and local expertise on specific building performance (Silva et al. 2018). The analytical methods allow V/F functions to be easily updated, complemented, and modified as more refined data on exposure or refined analytical approaches become available. In addition, the analytical approach considers region-specific characteristics such as the hazard specifications, local geographical seismic conditions, and local characteristics, generating more reliable vulnerability curves.

Recent studies on vulnerability and resilience of school portfolios, increasingly apply analytical vulnerability approaches considering local characteristics of buildings. Indeed, Samadian et al (2019) present a new methodology based on regional economic conditions for loss estimation used to derive vulnerability functions in a broader framework to assess resilience in school buildings in Iran. Similarly, Gonzalez et al (2020) provide a methodology to quantify seismic resilience in Mexico City school facilities. In this methodology, the physical and human vulnerability functions are derived to develop, in a next step, a recovery function to assess resilience. An approach applied to Nepal school portfolio, developed by Giordano et al (2020), characterize the out-of-plane response of unreinforced masonry buildings. Despite these current developments and additional efforts such as methods proposed in literature (Michel et al. 2014; Rossetto et al. 2014; Yamin et al. 2014; Mora et al. 2015; Silva et al. 2018) there is a lack of a technically robust and comprehensive yet simple methodology to assess the vulnerability of school building aggregate at scale, for example at regional or national level, economically and reliably.

To address the need of a comprehensive yet simple methodology for assessing the seismic fragility/vulnerability of school buildings, this study presents an efficient and reliable methodology to assess seismic vulnerability in Reinforced Concrete (RC) school buildings, based on the definition of a comprehensive taxonomic characterization for the development of index buildings and estimation of simplified seismic vulnerability functions using a component-based approach. The proposed methodology is part of the developments made in the Global Library of School Infrastructure (GLOSI) framework of the GPSS, funded by the Global Facility for Disaster Reduction and Recovery (GFDRR) of the World Bank. GLOSI is a global library of data and information related to school infrastructure as well as methodologies and tools for assessing and reducing the associated vulnerability (The World Bank 2019). It is comprised by a global catalogue of school building types, generic vulnerability information and retrofitting solutions that can be implemented at large-scale level (The World Bank 2019). The paper is organized as follows: section 2 presents the details of the proposed simplified vulnerability assessment methodology. Section 3 presents a case study application in the global GLOSI index buildings. Section 4 develops a sensitivity analysis varying different parameters to understand the versatility and reliability of the methodology and its advantages for future use of vulnerability in risk assessments of school infrastructure. For the sensitivity analysis, the geometrical variations, foundation-soil flexibility, quality of masonry infills, non-structural vulnerable elements and the analysis type are considered. Even though we acknowledge that some buildings in a school facility may have horizontal and vertical irregularities in cases of kitchens, canteens, administrative or mixed-use buildings, the focus of this study are the RC schools buildings used for classrooms only, where the students spend most of their time. These buildings are mostly regular in plan and elevation, as identified in the SIDA report in Nepal (Digicon Engineering Consult and The World Bank 2016) and more globally in the GLOSI (The World Bank 2019). Finally, section 5 presents the summary and conclusions.

2 Proposed methodology for the simplified vulnerability assessment

The methodology presented herein aims to develop simplified seismic vulnerability functions using a component-based approach for low and mid-rise school buildings (1 to 3 stories). The methodology focuses on RC buildings, since this is one of the most common structural materials found in school infrastructure. However, the methodological approach is also applicable to other structural systems and materials. Indeed, a parallel study has been developed for masonry structures and details are available in Adhikari (2021), Vatteri & D'Ayala (2021) and GLOSI (The World Bank 2019). Fig. 1 summarizes the main steps of the proposed methodology to derive vulnerability functions. The methodology is comprised of six main steps: data collection, index buildings characterization, hazard assessment, numerical modeling, seismic performance assessment, and vulnerability integration. The data collection process and typology characterization is a substantial methodological endeavor in its own right, as reported in GLOSI (The World Bank 2019), while the present manuscript concentrate on the seismic performance assessment and vulnerability function derivation.

The first two steps are the data collection and the definition of Index Buildings (IBs). The data collection is one of the most time and cost consuming tasks since institutional repositories do not usually collect information related to structural characteristics of school buildings. There are several methodologies to obtain information, such as field surveys, distance surveys, satellite images, proxies based on limited existing databases, or combinations of these (Aleskerov et al. 2005; Prasad et al. 2009; Gunasekera et al. 2015; Fernández et al. 2021). The identification of representative index buildings stems from the statistical analysis of the collected data. By applying the entire GLOSI taxonomy string to a large number of buildings, the most common combinations of parameters emerge, identifying recurring IBs. Once these are identified, intrinsic parameters, i.e., geometry and materials characteristics, should also be determined for each IB for modelling purposes. The GLOSI system has a number of predefined IBs that can be used to compare with the country specific data. Further information can be found in D'Ayala et al (2020) and The World Bank (2019).

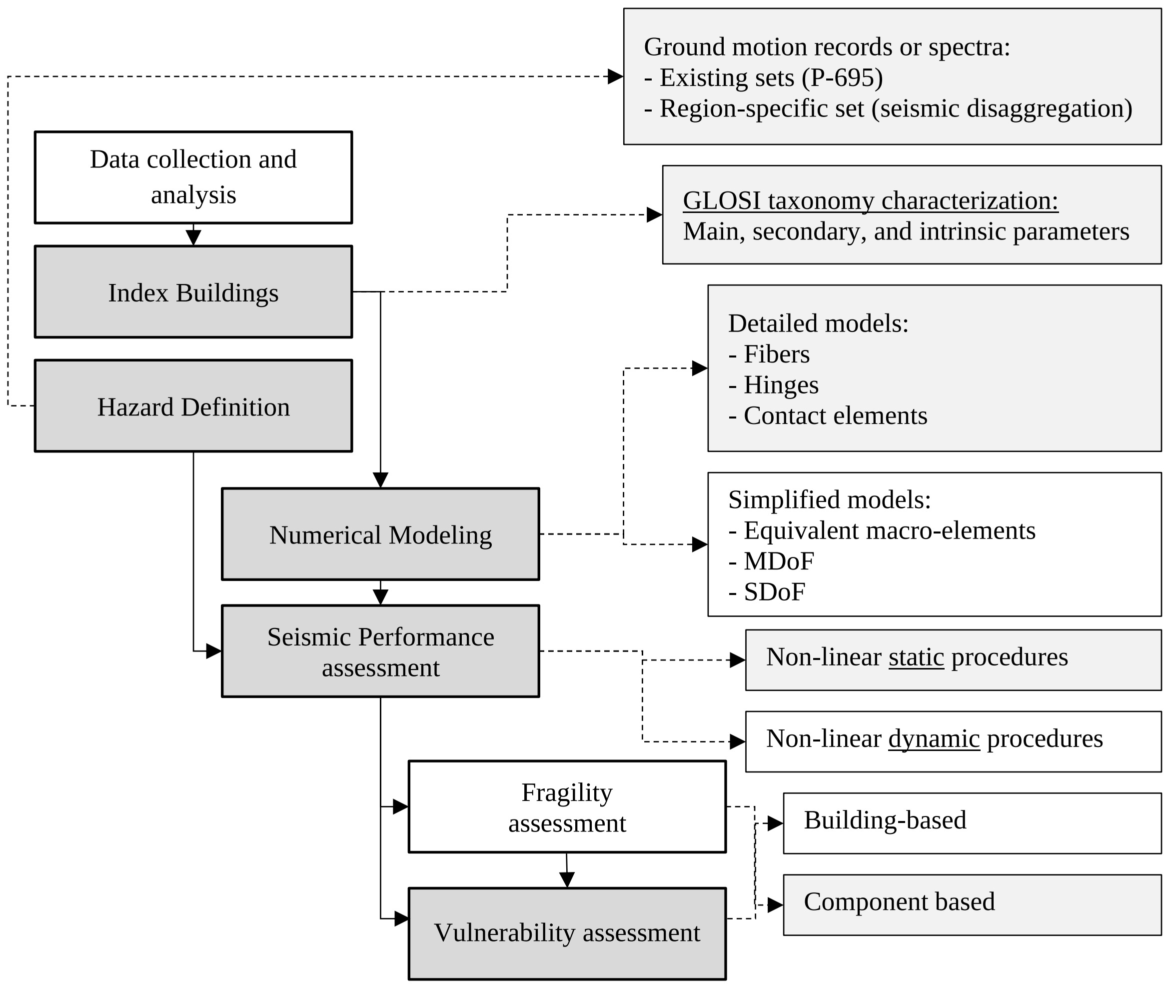


Fig. 1 Proposed simplified vulnerability assessment methodology – In dark grey the steps followed in this study

Once the IBs are fully characterized, the next step of the methodology is the seismic hazard definition. This may be selected using different techniques as the conditional mean spectrum (Baker 2011) or the uniform hazard spectrum (ASCE and SEI 2017a). However, these methodologies depend on a site-specific knowledge of the seismic conditions, which is seldom known in most countries. For a generalized and broader perspective, the hazard may be defined in terms of the acceleration/displacement spectra of the far-field set of earthquake ground motion records given by FEMA P-695 (ATC 2009) or other similar regional repositories. In particular, the FEMA P695 ground motion set is built to meet the following objectives: "to represent strong ground motions, to be statistically representative, broadly applicable for collapse evaluation of various structural systems and broadly applicable to structures at unknown location" (ATC 2009). Even though this set is primarily recommended by FEMA to assess the collapse fragility of buildings through an Incremental Dynamic Analysis (IDA), it can also be utilized to analyze lower damages states and corresponding losses, as well as to be used in an Incremental Static Analysis (ISA) (Dolšek and Fajfar 2004; Gogus and Wallace 2015; Ezzeldin et al. 2016). Also, its replicability, easy access and generic characteristics, not linked to a specific area, make suitable to be used in the present study and within the global remit of the GLOSI. Nonetheless, any set of appropriate ground motions specific to a location, obtained from literature databases such as the PEER Strong Ground Motion Databases (PEER 2020a), or by synthetic generation, can replace the current choice.

The next step of the methodology is the numerical modeling of the IBs. The modeling main objective is to develop the pushover curve of the IB in the most critical direction. This numerical model should be nonlinear and three-dimensional to characterize the structural behavior. To this end, depending on the complexity of the structure analyzed and the identified failure modes, both plastic hinges or distributed plasticity can be used, following internationally accepted methodologies (Mander et al. 1988; Elwood 2004; ATC 2005; Ibarra et al. 2005; ASCE and SEI 2017b; Di Trapani et al. 2018). Any acceptable software can be used for the pushover curve derivation. Common computer software available to perform this analysis are SAP2000, ETABS, Perform3D and OpenSees among others (CSI Computer & Structures Inc 2004, 2016, 2020; PEER 2020b). It is important to note that this methodology is limited to the most critical unidirectional analysis, the consideration of bidirectional or vertical effects are out of the scope of this paper.

The pushover curve developed in the previous step will be used to derive the seismic performance assessment. For this, an Incremental Static Analysis (ISA) is proposed which employs the latest version of the N2 method in a set of incremental ground motions spectra (as in IDA analysis) (Fajfar 2000; Vamvatsikos and Cornell 2002; D'Ayala et al. 2015). This ISA methodology (Mwafy and Elnashai 2001; Dolšek and Fajfar 2005), shows a good correlation with IDA methods for RC buildings with and without masonry infills. The N2 methodology may be summarized as follows: for each IB, the multiple degrees of freedom (MDOF) pushover curves are converted to a bilinear idealized pushover curve of the equivalent single degree of freedom system (SDOF) following standard engineering practices. This pushover is intersected with the inelastic demand spectrum for each different ground motion in the selected set (scaled to different intensity measure values) to generate several seismic performance points as Engineering Demand Parameters (EDP). The inelastic response is calculated for a 5% damped elastic response spectrum, considering a ductility factor. These results are expressed as a relation between the intensity measure (IM) and the corresponding EDP. This methodology has been recommended by the Eurocode-8 (European Committee for Standarization 2004) and have the advantage that its implementation for the determination of the performance point is relatively easy. More details on this can be found in D'Ayala et al. (2015).

Other simplified methodologies available in the literature may be used to perform the nonlinear seismic analysis, such as Fishbone (Nakashima et al. 2002; Qu et al. 2019), UMRHA (Chopra and Goel 2002; Li and Ellingwood 2005), and others (Miranda 1999; Miranda and Akkar 2006) which apply simplified models. The selection of one option over another depends on different factors, such as purpose of the analysis, the acceptable level of uncertainty, the availability of resources, and the data available (ATC 2005). An efficient nonlinear seismic analysis with reduced computational effort as well as resources, while able to minimize the effect of uncertainties is favored for simplified vulnerability assessment. The modelling uncertainty, according to FEMA 440, is due to the following two components: the seismic analysis and the structural model. In the first case, the seismic analysis could be performed using a pushover analysis or a response history analysis, where the highest accuracy is obtained with a non-linear response history analysis. The proposed ISA methodology is particularly efficient and accurate in the case of school infrastructure considering its geometric and structural characteristics (regular and low- to mid-rise). For the second type of uncertainty, the structural model could be represented as Single-Degree of Freedom (SDOF) or Multiple Degree of Freedom (MDOF) models. The main consequence of simplifying the structural model using a SDOF instead of a MDOF is that it lacks the representation of realistic failure modes or the interaction between components. For this reason, the proposed methodology includes MDOF detailed model to consider all these aspects, analyzed using the ISA methodology.

With the resulting EDPs, the final step is the derivation of vulnerability functions. It is important to note that the development of functions should consider a wide range of uncertainties that depend on the quality of the available information and the types of models and analyses used in each assessment (Porter et al. 2002; Wen et al. 2003; D'Ayala et al. 2015; Silva 2019). In seismic vulnerability assessment, the uncertainty is associated to seismic input, numerical modeling, material properties, damage states, costs modeling and others (Yamin et al. 2017). These uncertainties can be characterized as random or epistemic (Kiureghian and Ditlevsen 2009; D'Ayala et al. 2015). Random uncertainties are associated with the seismic input, the soil response, the frequency content of the seismic records used, and the variability in the materials and design of the building stock. Epistemic uncertainty is associated with a lack of knowledge from a physical or engineering point of view, limitation of the numerical modeling methodology, the estimation of the damage states, the repair cost estimation, and other analytical parameters used in the assessment (Yamin et al. 2017). All uncertainties are represented in the probability distribution function of each damage state of the fragility functions or in the variance function indicated for the vulnerability function (which also depends on the seismic intensity level). To consider and propagate the uncertainty through the vulnerability functions, different methodologies considering the variability of EDP are represented by a probabilistic distribution with an uncertainty β (ATC 2012; D'Ayala et al. 2015; Rincon et al. 2017). Therefore, the derivation is carried out using a component-based methodology following the approach proposed by Yamin et al. (Yamin et al. 2017). This entails a model based on damage-susceptible components, either structural or non-structural, which convolves the repair to replacement cost ratios and the cumulative probability of damage expressed through the fragility functions. The uncertainties and randomness of the variables are accounted for by performing a Monte Carlo simulation. The method is further improved by implementing a more efficient seismic analysis developed through the ISA assessment. The result of the methodology is a vulnerability function for the analyzed building, including the mean damage ratio and the corresponding variance at each intensity level. It is important to note that the building-based approach can also be used to develop the vulnerability functions even though the authors recommend using a component-based approach for its flexibility to include structural and non-structural components.

3 Study case application

3.1 *GLOSI RC index buildings*

The methodology described above was applied to a set of reinforced concrete Index Buildings (IBs) in the GLOSI catalog (The World Bank 2019; D'Ayala et al. 2020). These IBs were selected to be two-story buildings in the mid-rise (MR) category (one story is considered low-rise and two- and three-story buildings are classified as mid-rise in GLOSI framework since school buildings usually do not have more than 4 stories). This category was selected based on the distribution of the number of stories of RC school buildings in developing countries such as Peru, Nepal, the Philippines and the Dominican Republic (Nassirpour et al. 2018; The World Bank 2019). Five RC structural typologies with different lateral resisting systems leading to various failure modes are considered for the analysis: RC moment resistant bare frames (RC1) with light infill panels; RC moment resistant frames with connected masonry infill walls (RC2); RC moment resistant frames with reduced height of masonry infill walls generating short column effects (RC3); RC moment resistant frames with steel bracing (RC4); and non-engineered RC systems where no frames are designed, usually a thin concrete slab connect the columns, and are usually built by the community without following any building code (RC5). Each structural system is evaluated for poor, low, or high design levels, as shown in Fig. 2. For illustration purposes, Fig. 3 presents the characteristic geometry of each structural system considered. It is important to note that RC1 typology does not include any masonry walls or partitions to present its moment-frame structural behavior as is. However, it is common to find actual RC1 typologies in the field with light partitions or masonry walls with sufficient separation from the structural elements. All structural models were defined with the same geometry characteristics for consistency in the results and comparisons.

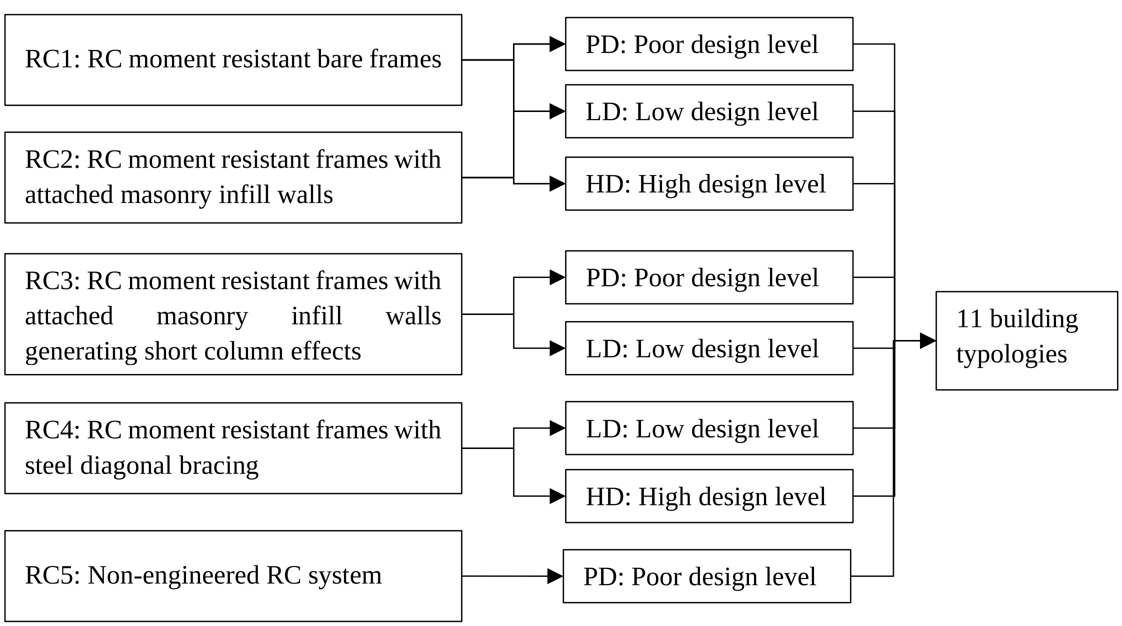


Fig. 2 Case study typologies

Fig. 3 Geometry of selected typologies

In addition to the structural characteristics presented above, each building was characterized by a taxonomy string using the GLOSI classification system, developed by the authors for The World Bank (The World Bank 2019). The secondary parameters were fixed for all the eleven buildings with the following considerations: rigid diaphragm (RD), no irregularities (NI), long span –corresponding to typical classroom geometric configuration– (LS), regular column –strong column weak beam frame configuration– (RO), rigid foundation (RF), no pounding risk (NP), original structure (OS), good condition (GC) and vulnerable nonstructural elements (VN) (such as parapets, gables, bookshelves and others to consider the additional damage cost, while nonstructural members which interact with the main structure are explicitly included in the structural system modelling). Table 1 presents the list of building typologies considered. The results described in this study are presented in reference to the Building Type string found in Table 1.

Table 1 Taxonomy of the RC Index Buildings currently available in the GLOSI library

ID	Building Type	Index Building
1	RC1/MR/PD	RC1/MR/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
2	RC1/MR/LD	RC1/MR/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
3	RC1/MR/HD	RC1/MR/HD/RD/NI/LS/RO/RF/NP/OS/GC/VN
4	RC2/MR/PD	RC2/MR/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
5	RC2/MR/LD	RC2/MR/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
6	RC2/MR/HD	RC2/MR/HD/RD/NI/LS/RO/RF/NP/OS/GC/VN
7	RC3/MR/PD	RC3/MR/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
8	RC3/MR/LD	RC3/MR/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
9	RC4/MR/LD	RC4/MR/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
10	RC4/MR/HD	RC4/MR/HD/RD/NI/LS/RO/RF/NP/OS/GC/VN
11	RC5/MR/PD	RC5/MR/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN

In relation to the seismic design levels, each building is modeled as indicated by the GLOSI taxonomy as follows. Poor design (PD) buildings are only dimensioned to withstand gravity loads so they have a very small capacity for lateral loads (no confinement stirrups). Low design (LD) buildings are designed for low lateral loads, so no seismic confinement stirrups exist in the plastic hinge zone of the elements (spacing between stirrups greater than $d/2$, where d is the distance from the extreme compression fiber of the section to the centroid of the reinforcement). The minimum dimension of structural elements at this design level is 200 mm. Fragile collapse mechanism and low lateral capacity are expected for this case. Finally, high design (HD) buildings are designed for a high seismic hazard zone with specific requirements as continuity in the longitudinal reinforcement of the elements, confinement zone with a separation of stirrups equal to $d/4$ and the minimum dimension for structural elements as 300 mm. The assumptions for each design level are based on the ASCE 7-10 (ASCE and SEI 2017a) and the ACI 318-14 (ACI 2014) but adapted with expert criteria for a more global application. Medium design was not considered in this study case since it differs considerably between countries and design codes. Non-structural elements are designed to withstand seismic forces unless specify otherwise. For this condition, it is expected ductile collapse mechanisms and very high lateral capacity.

3.2 Ground motions

Once the IBs have been characterized, the next step is the hazard definition for the non-linear static analysis. In this study, the possible hazard was based on an approach given by FEMA P-695 (ATC 2009). This approach considers a set of pre-selected, far-field and near-field real ground motions. In the present application far-field records are used to assess more globally applicable characteristics. This set has a PGA between 0.22 and 1.43g; a PGV between 30 and 167 cm/sec; a distance from the source between 1.7 and 8.8 km; a minimum M_w of 6.5 and is selected for soft rock and stiff soil conditions – e.g. soil types similar to NEHRP C & D (Building Seismic Safety Council 2003). The acceleration spectrums for this set are presented in Fig. 4. However, additional sets such as near-field or locally specific set can be used in other case studies. Given the global level application, no specific additional consideration to the type of soil is included in the present application. However, the results of a sensitivity analysis are discussed in section 4.2 to evaluate the effect of soil stiffness on the capacity curves obtained for two different types of foundation configuration, and the resulting vulnerability functions.

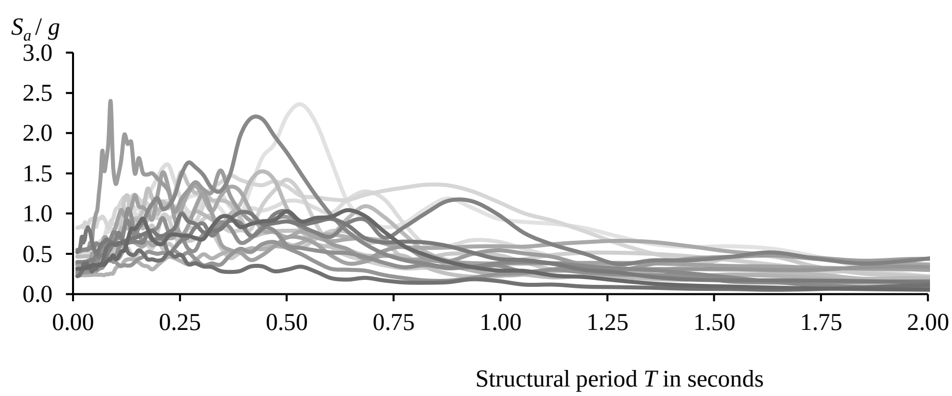


Fig. 4 Far-field ground motion response spectra

3.3 Structural modeling

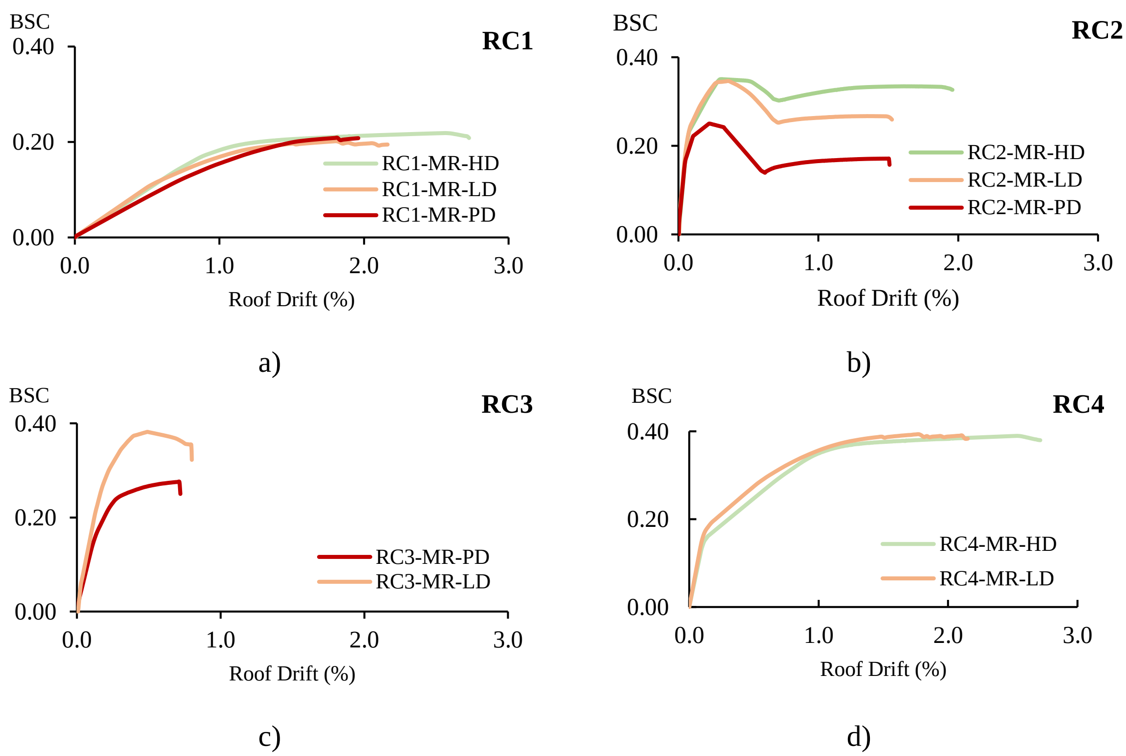
Details of the modelling considerations, material properties and general geometry details are presented in Table 2. These are based on information gathered from different countries, as documented in the GLOSI library (The World Bank 2019) and the recommendations given by the ASCE 41-17 (ASCE and SEI 2017b) for the non-linear analysis of RC buildings. The general geometric configuration is maintained in all IBs for consistency in the results. However, it is important to note that different buildings in reality may have slight variations in geometry, but this is out of the scope of this paper.

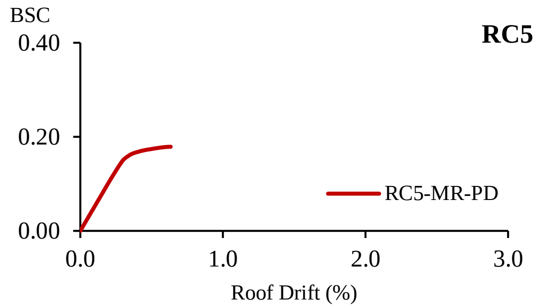
Table 2 Modeling considerations

General considerations	Model type	3-D
	Non-linearity considerations	Concentrated plasticity (hinges) based on ASCE 41-17 (ASCE and SEI 2017b)
	Structural elements modeling strategy	Concrete beams and columns as frames, masonry infills modeled as equivalent struts and steel bracing as diagonal elements only considering its tensile capacity.
	P-Delta effects	Yes
	Rigid zones in nodes	Yes
	Rigid diaphragms	Floors and roof
	Cracked sections	Main structural elements
	Foundation flexibility	fixed based condition is adopted
Loads considered in the analysis	Elements self-weight	Yes
	Additional dead loads	Slabs, nonstructural walls, non-structural elements, roofs, ceilings, etc.
	Live load	25% of the design live load is considered for the non-linear analysis
Material properties	Concrete f'_c	PD: 17 Mpa; LD and HD: 21 Mpa
	Reinforcement f_y	PD, LD and HD: 420 MPa
	Masonry f'_m	2.8 Mpa
Geometry	Building plane area	300 m^2
	Story height	3 m
	Number of spans in long direction	7
	Typical span length in long direction	4.5 m
	Number of spans in short direction	3
	Typical span length in short direction	3.5 m
	Typical column dimensions (cm x cm)	Poor design:20×20; Low design:25×30; High design: 40×30
	Typical beam dimensions (cm x cm)	Poor design:20×30; Low design:25×30; High design: 30×35

Including the above considerations, the buildings were modeled using Perform3D software (CSI Computer & Structures Inc 2016), chosen because it provides enough reliability in the non-linear pushover analysis and allows to also run dynamic nonlinear analysis, which is also the basis for the sensitivity analysis and the validation approve. Moreover, Perform3D being a commercial software, it demonstrates that the methodology can be used by professionals as well as researchers.

Fig. 5 presents the pushover analysis for each typology and each design level. These results show that the main structural system defines the pushover curve with substantial differences between the main typologies, which confirm the suitability of the main classification. On the other hand, the parameter design level controls the maximum strength and the ductility capacity, also clearly indicating that the construction characteristics modifiers used to determine the design levels are appropriate to determine the capacity. Indeed, the RC2 and RC3 IBs have different pushover progressive structural behavior, but for both typologies, the resulting curves have different strength and ductility depending on the design level and the effect of the masonry infills interaction. Fig. 5 also shows that the RC1 and RC4 typologies present similar pushover curves in shape, but with better structural behavior in RC4 in terms of initial stiffness and ultimate strength, while the ductility is preserved. The latter occurs since, in this case, the steel bracing was designed to fail at the drift level corresponding to the ultimate limit state of the concrete frames. However, other collapse sequences can be achieved when bigger or smaller sections of steel bracing are considered, which should be designed depending on the specific building code requirements of each case study. Finally, RC5 presents the lowest ductility and strength of a non-engineered structural system with low construction quality. It is also important to note that these results are obtained for fixed characteristic of geometry and design level, therefore results are significant for comparative purposes between typologies, but they do not capture the full range of behavior within a typology. To understand the effects of changes of the reference conditions chosen in this initial assessment, a sensitivity analysis varying the IB identifier parameters is presented in section 4. Also, damage state thresholds are not included in the pushover since the vulnerability derivation is component-based, and therefore, each component will have its own threshold.



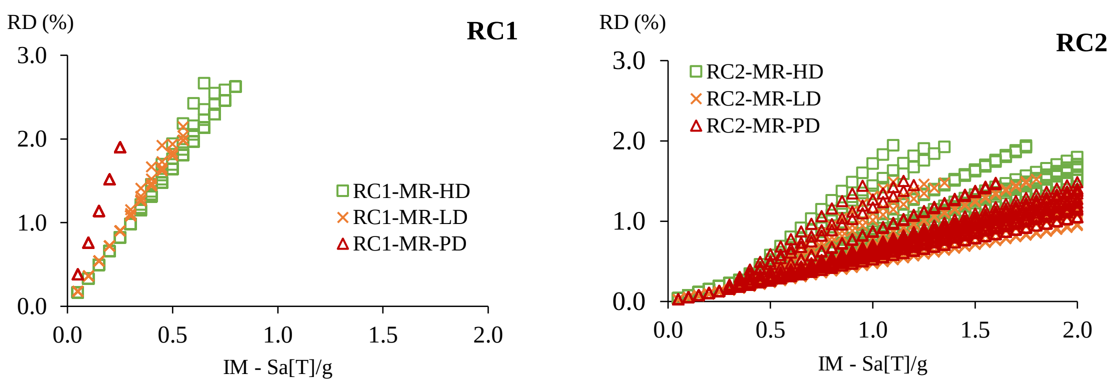


e)

Fig. 5 Pushover curves (BSC = Base Shear Coefficient)

3.4 Incremental static analysis

The integration of the hazard and the structural modeling is done by performing an incremental static analysis (ISA) based on the N2 method. This procedure results in the estimation of the engineering demand parameters (EDPs). These parameters were obtained by identifying the performance point, i.e., the intersection between the linearized pushover capacity curves shown in Fig. 5 and the non-linear response spectrum for each ground motion at each scaling stage. Further details of this methodology are explained in D'Ayala et al (2015). In this study, the analysis was developed to obtain the displacement for the roof and story level, as Engineering Demand Parameter (EDP). With these results, the roof drift and inter story drift are calculated using the story height. Fig. 6 presents the EDP results in terms of the roof drift for each structural system and each design level. These results show that the main structural system controls the structural behavior (as observed in the pushover analysis). The resulting EDPs are used to correlate with the damage state thresholds for each component included in the building, as presented in the following section.



a) b)

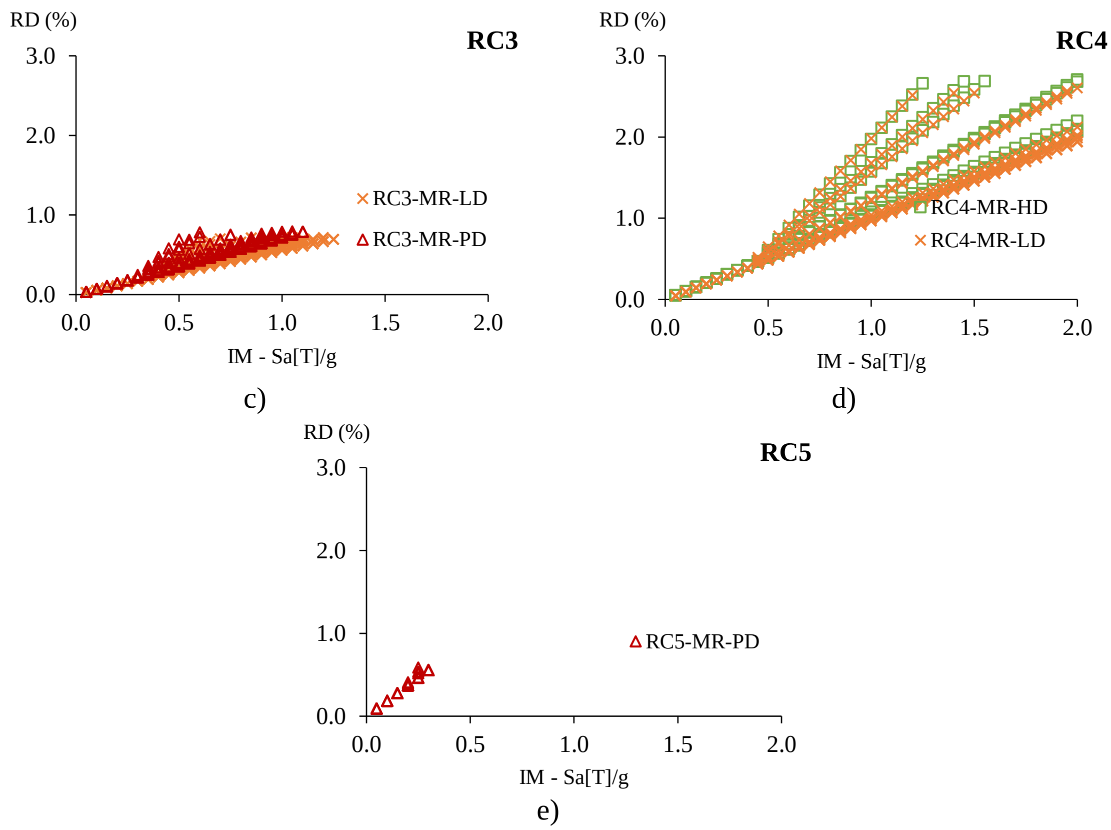


Fig. 6 RC mid-rise buildings engineering demand parameters – EDP (RD = Roof Drift)

3.5 Component and costs model

With the EDPs characterized for each building, the next step is the development of the component- and repair cost-based loss model, where estimated damage is expressed as repair/reconstruction vs. total cost. For this purpose, a component model, including structural and nonstructural elements, is assembled for each building under consideration, based on the FEMA P-58 vulnerable elements fragilities (ATC 2012). These set of fragilities were adapted for this specific case study to prove the global applicability of the proposed methodology by modifying the repair cost of each damage state to represent the conditions for low- or middle-income country based on previous studies in Latin America and the Caribbean (The World Bank 2018, 2020). However, further studies should be done to characterize the behavior of typical components for particular regions considering the local construction industry and details of the school infrastructure portfolio. The model includes all structural and non-structural components typical of school buildings, for each story. For each type of component, the measurement unit, the quantity of elements, fragility in terms of repair cost and time at different damage states, the controlling EDP, and correlation of damage between similar components at the same story is defined. Table 3 illustrates the typical component model for the buildings under consideration in this case study. The component model chosen depends on the architecture and construction characteristics as well as the context, region, or countries of which the IBs are representative. Therefore, for specific applications of this methodology the component model in Table 3 will need tailoring. To understand how this affects the resulting vulnerability function, a sensitivity analysis is performed in section 4.

Table 3 Component Model

Group	Description	Unit	Fragility specification code (FEMA P-58)	*EDP*	DS correlation between components	Structural typology
Structural	Columns and beam end nodes	Node	B1041.001a	Drift	No	All
Structural	Column and beam central nodes	Node	B1041.001b	Drift	No	All
Non-structural	Confined masonry facade	5mx3m	C1011.006b	Drift	Yes	All
Non-structural	Confined masonry partition wall (veneer)	5mx3m	C1011.005b	Drift	Yes	RC2 and RC3
Non-structural	Confined masonry[1] partition wall	5mx3m	C1011.004b	Drift	Yes	RC2 and RC3
Non-structural	Plastered ceiling	5mx5m	C3032.005a	Drift	No	All
Non-structural	Gas piping	22ml	D2022.025a	Drift	Yes	All
Non-structural	Electrical piping	110ml	D2021.011a	Drift	Yes	All
Non-structural	Water piping	62ml	D2022.011a	Drift	Yes	All
Contents	Contents (acceleration controlled)	5mx5m	E2022.010	Drift	No	All
Contents	Contents (drift controlled)	5mx5m	E2022.010a	Drift	No	All

Fragility functions were assigned to each component type in the model described above. They represent the probability of being in each damage state (usually slight, moderate, or extensive) as a function of the corresponding EDP (as defined previously). Each damage state is assigned a probability density function of repair cost and time, according to the FEMA P-58 (ATC 2012) catalogue. Further detail is given in Yamin et al (2017).

3.6 *Vulnerability functions results and discussion*

As described above, the integration of the previous results using a component-based method results in each IB's vulnerability function, which is the main result of the proposed methodology. The procedure to integrate the losses of each component in each EDP is described in Yamin et al (2017), which apply equally in this case, with the difference that the EDPs are obtained from an incremental static analysis instead of an incremental dynamic analysis. The vulnerability functions can be formulated using a beta distribution following Equation 1 (ATC 1985).

[1] Confined masonry refers to infill walls built with a secondary system of columns and beams ties in a framed RC building.

$$E[\beta] = E\left[1 - K^{\frac{V^{\rho}}{\gamma}}\right] \qquad (1)$$

where $E[\beta]$ is the expected MDR, V is the IM, K is the known MDR, γ the intensity for the known MDR K and ρ is the curvature parameter. The summary of the parameters for each function is presented in Table 4. The graphic results are presented in Fig. 7 (continuous line refers to the Mean Damage Ratio while the dotted line refers to the Variance in the results, this applies to all vulnerability functions presented herein).

Table 4 Vulnerability functions parameters

Structural system	Height range	Seismic design level	Intensity at which damage begins	Intensity for complete damage	Inflection point Mean Damage Ratio (K)	Inflection point Intensity (γ)	Curvature before inflection point (ρ)	Curvature after inflection point (ρ)
RC1	Mid rise	Poor	0.08	0.30	50.00	0.20	5.00	5.00
		Low	0.10	1.10	50.00	0.55	2.30	3.00
		High	0.10	1.50	45.00	0.70	2.50	3.00
RC2	Mid rise	Poor	0.10	5.00	50.00	2.00	2.00	2.00
		Low	0.10	5.00	45.00	2.00	2.00	2.00
		High	0.10	5.00	40.00	2.00	2.00	2.00
RC3	Mid rise	Poor	0.10	5.00	50.00	1.00	3.00	4.00
		Low	0.10	5.00	50.00	1.10	4.00	5.00
RC4	Mid rise	Low	0.10	5.00	35.00	2.00	2.00	2.00
		High	0.10	5.00	25.00	2.00	2.00	2.00
RC5	Mid rise	Poor	0.10	0.60	50.00	0.35	3.50	4.00

The first set of vulnerability functions is presented in Fig. 7-a for the RC1 IB and each design level. The maximum Mean Damage Ratio (MDR) of 1.0 is reached for an IM of 0.4g, 1.0g and 1.5g for poor, low, and high design respectively. These results imply that this structural system may experience total collapse, even when the high design level is considered. These results also show a loss variance (indicated by the dotted line) of around 10% for each typology centered around a MDR of 0.25g, 0.6g and 0.8g for poor, low, and high design respectively.

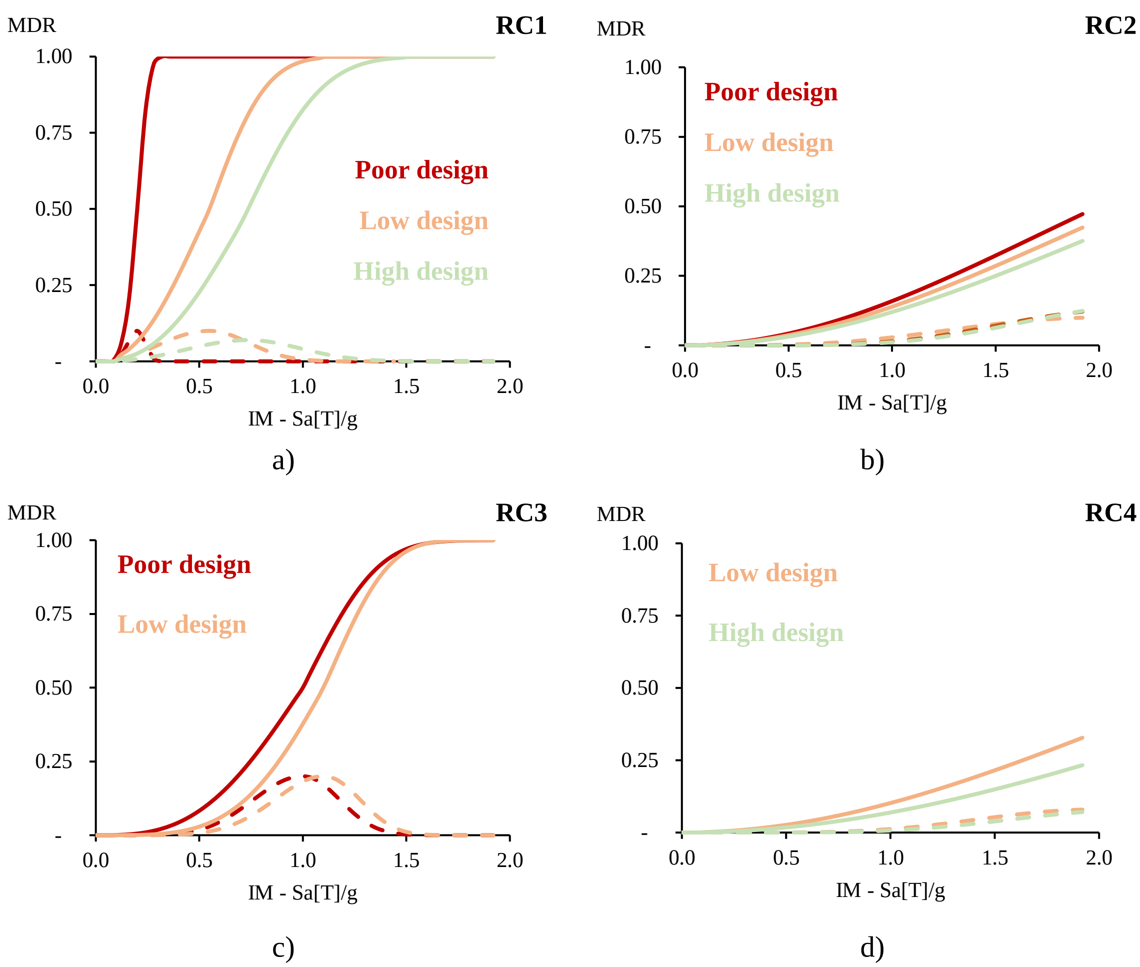
Fig. 7-b presents the second set of resulting vulnerability functions, showing the different design levels for the RC2 structural system. In contrast to the results obtained above, RC2 building typologies do not reach an MDR of 1.0 before a spectral acceleration of 2.0 g. These results indicate that this typology is less likely to collapse than RC1, however, it is important to note that variance in this range is usually larger than the one at the lower and larges IMs. Another change with respect to the previous results is that differences between design levels is not as noticeable as before, which is consistent with the resulting EDPs presented in Fig. 6. These minor differences suggest that in an RC2 typology, the design level does not control the vulnerability since the infills response governs the interaction between masonry walls and structural members.

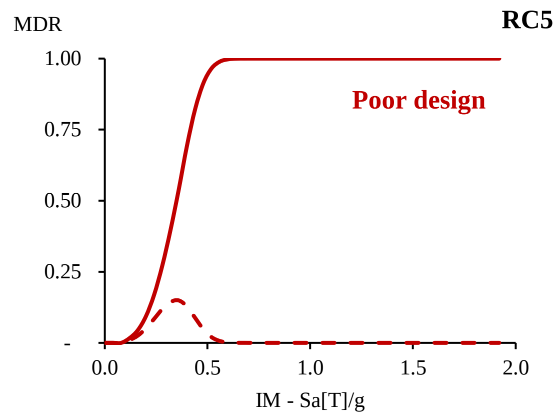
Fig. 7-c presents the results for the RC3 typology for poor and low design. These results show that this structural system presents a higher vulnerability to collapse (MDR = 1) than RC2 for intensity measures of about 1.5 g. This vulnerability condition is similar for both design levels, suggesting that the short column collapse mechanism controls the structural behavior. This conclusion is similar to

the one obtained before, which shows the high impact of the masonry infills in determining the structural behavior of a school building.

Fig. 7-d shows the resulting vulnerability functions for the RC4 typology for low and high design levels. These results indicate a low vulnerability in general for this typology, which is usually well built and designed. The resulting vulnerability functions also suggests that this typology can be used as a retrofitting option for the above IBs, reducing the MDR from 100% (total collapse) to only 25%.

Finally, as demonstrated in Fig. 7-e, the RC5 structural systems have a high vulnerability, presenting collapse for intensity measures around 0.5 g. This level is only comparable with the vulnerability presented for RC1 poor design level buildings, which is also a highly vulnerable typology.





e)

Fig. 7 RC mid-rise buildings vulnerability functions (MDR = Mean Damage Ratio)

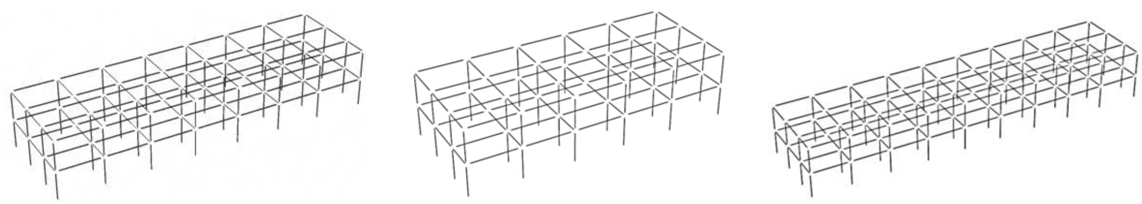
The vulnerability functions presented above are compared directly for illustration purposes. However, it is essential to note that two similar curves of different IBs can lead to different risk results when integrating hazard and exposure. This difference occurs because different typologies may have different structural periods, resulting in a different hazard level (IM) and, hence, a different level of damage. This type of analysis is out of the scope of the present study but should be conducted for understanding the risk results at a regional level and for the development of effective retrofitting programs.

4 Sensitivity analysis

Different construction characteristics in a portfolio of buildings represented by one IB pose a number of uncertainties, from actual material properties of masonry components to variation in geometry and layout. Also, the consideration of non-structural elements and the modelling strategy affects the final vulnerability functions of IBs. To analyze this uncertainty, a sensitivity analysis is performed, including variations in five different parameters. The first one is geometry, which is included since there is high variation between countries and inside each country. For example, buildings usually include three classrooms, but in smaller schools there exists submodules of two or even one classroom, while in larger school facilities modules of four or five classrooms can be found. The second parameter is the foundation-soil flexibility, which is included mainly due to the spatial variability, and therefore uncertainty, of the soil conditions. The third parameter is the quality of masonry infills, that varies among countries. For example, in El Salvador and the Dominican Republic, it is very common to find infill walls of reinforced concrete blocks while in Colombia such walls are unreinforced clay bricks. The fourth parameter analyzed is the effect of including the losses associated with non-structural vulnerable elements. Finally, the analysis type is also considered since it can have an important influence on the results. These analysis are done independently following the One Factor at a Time (OFT) method (Porter et al. 2002).

4.1 Geometrical variations

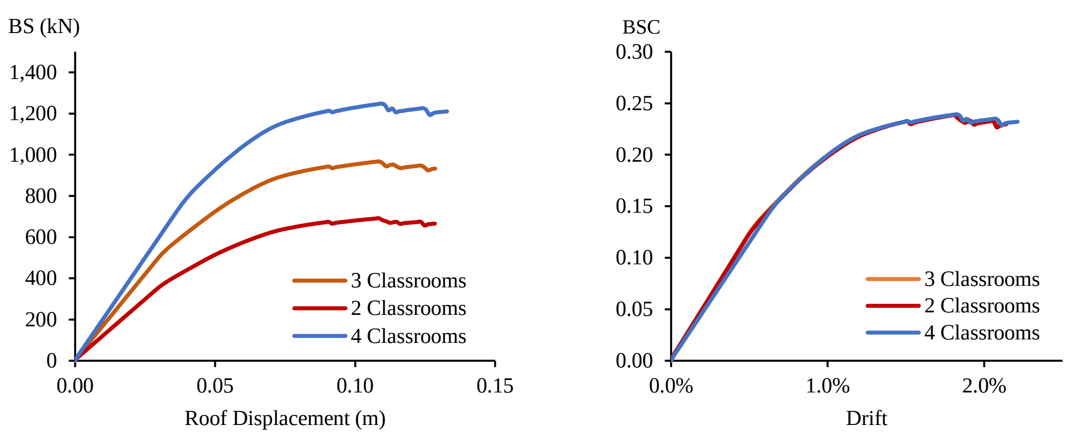
To understand how geometrical variations of the school buildings layouts affect the vulnerability functions, an RC1 mid-rise building with low seismic design (RC1/MR/LD) was analyzed. Three different layouts were selected for the analysis as illustrated in Fig. 8, representing a plan with three (the most common), two and four typical classrooms. All models are two-story buildings, and the frame dimensions and reinforcement details are maintained constant.



a) Three classrooms (IBRC-2) b) Two classrooms c) Four classrooms

Fig. 8 School buildings modules

Fig. 9 presents the capacity curves relating the maximum roof displacement associated to different total base shear forces. Normalized pushover curves for the three models, shown in Figure 13b in terms of the Base Shear Coefficient (BSC), do not present significant variations. Considering that the Engineering Demand Parameters (EDPs) are obtained using the N2 method, no significant variations are expected in the final vulnerability functions for the three models. Therefore, it is concluded that the vulnerability function for the three-classroom model is representative of other general plan layouts, as long as no irregularities or other critical structural behavior is generated with alternative layouts.



a) b)

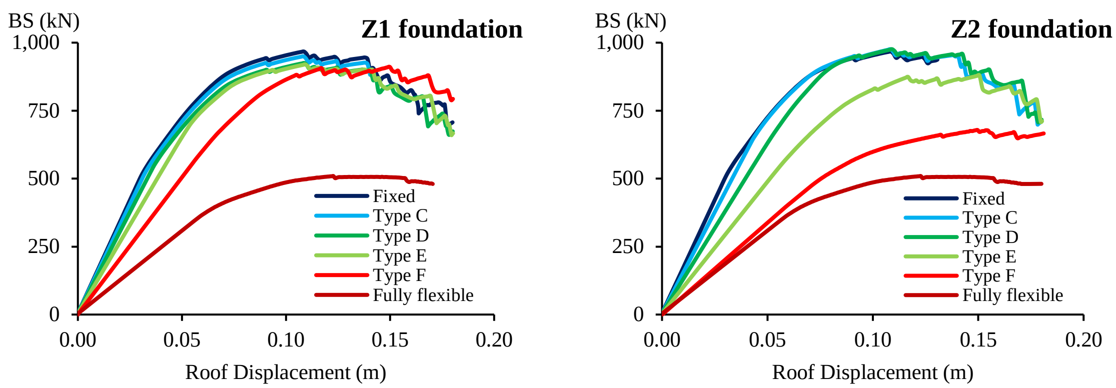
Fig. 9 Capacity curves for different geometries (BS = Base Shear, BSC = Base Shear Coefficient). a) Capacity curves. b) Normalized capacity curves

4.2 Foundation-soil flexibility

To assess the possible variations in the vulnerability functions for different soil-foundation stiffness, the RC1 mid-rise building with low seismic design (RC1/MR/LD) was analyzed using two different foundation configurations: 1.0 m by 1.0 m (Z1) and a 0.5 m by 0.5 m. (Z2) isolated footings. Each of these configurations was combined with four different soil types as indicated in Table 5 (Wald and Allen 2007; ASCE and SEI 2017a). Resulting capacity curves are presented in Fig. 10 for all possible combinations of foundation and soil type. Corresponding vulnerability functions are presented in Fig. 11, including the results for the full rigid and full flexible cases which are the same for both Z1 and Z2 configurations.

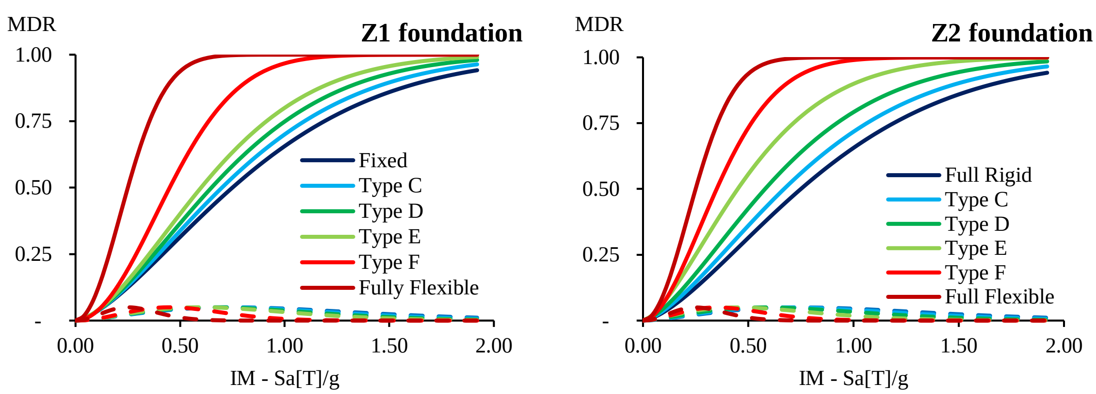
Table 5 Soil properties for foundation stiffness calculation

Type	G/G_0	Soil Type	Density $_{\text{sat (kN/m3)}}$	Mean V_{s30} (m/s2)	ν
C	0.9	Lime	22	500	0.35
D	0.81	Clay	18	300	
E	0.47	Clay	18	200	
F	0.32	Clay	18	100	



a) b)

Fig. 10 Capacity curves with foundation in different soil types (BS = Base Shear)



a) b)

Fig. 11 Vulnerability functions with different foundation stiffness (MDR = Mean Damage Ratio)

From the results it is possible to conclude that good foundation configurations (represented by Z1 footings) will generate pushover curves and vulnerability functions showing a behavior closer to the rigid base model assumption, showing an IM of around 0.7 g for the fix-end assumption to 0.56 g for the Type E soil for 50% MDR. However, for a considerable flexible soil (type F), the vulnerability is substantially increased, presenting an IM of 0.44 g for 50% of damage and total damage at an IM of 1.00 g. For relatively weak foundation configurations (represented by Z2 footings), higher vulnerability curves are obtained with considerable variations for different soil types. For stiff soil profiles (soil types C or D in the previous table) the expected behavior will approximate the fixed base assumption. On the other hand, for flexible soil profiles (soil types E, or F) the expected behavior will approximate the hinged base assumption. It is important to note that this combination may not be found in reality but it is included in the analysis as an illustrative case for comparison. In conclusion, the most common assumption of rigid base behavior can be sustained only when a relatively good foundation configuration is expected in medium or stiff soil profiles. In the cases where there is evidence of soft soil profiles with probable deficiencies in the foundation configuration, flexible support conditions shall be considered in the assessment, given that those conditions will generate a higher vulnerability for the building under consideration.

4.3 Masonry infills quality

To test the relevance of masonry infills quality in the resulting vulnerability function, different masonry properties were selected, additional to the ones selected in section 3, as summarized in Table 6. These masonry properties were selected by the authors based on the test developed by Carrillo (2004) and with new materials available for construction in Colombia. In this case, the variations in masonry were analyzed in the IB model RC2, mid-rise building with low seismic design (RC2/MR/LD), for which infill are explicitly modelled to quantify their contribution to structural response.

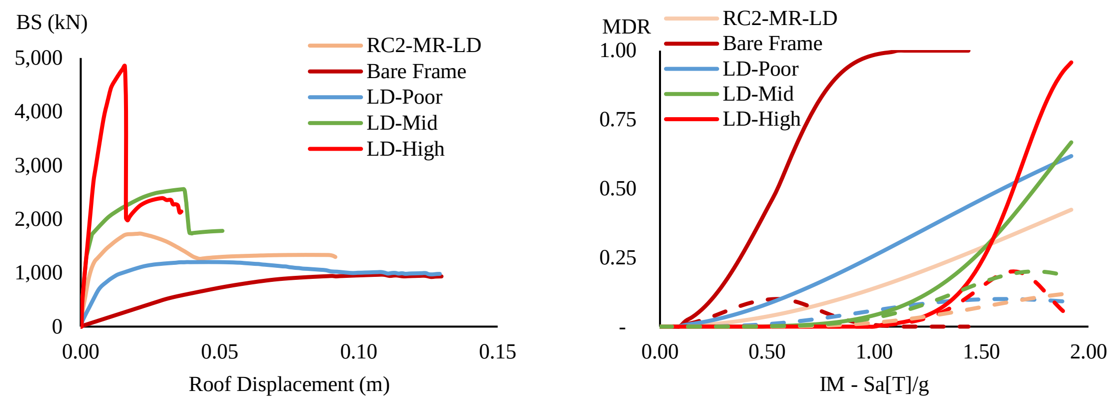
Table 6 Masonry properties

Quality	Block Material	Dimensions (bxlxt)	f_v (Mpa)[2]	f'_m (Mpa)[3]	Friction coefficient
High	Clay brick	10x20x6	0.9	12	0.7
Medium	Clay brick	10x28x6	0.1	4.7	
Poor	Clay tile	11x30x20	0.1	2.0	
Reference	Clay brick	10x20x6	0.6	2.8	

Fig. 12a presents the capacity curves for the three assumptions of masonry quality as compared to the bare frame (no masonry infills) conditions and the reference IB (RC2-MR-LD). From the figure, it is clear that masonry infills, when not isolated from the structure, can heavily affect the expected structural behavior of the building. Also, the collapse mechanism of the building can significantly change, as more resistant but fragile behavior can be obtained. For the cases of High, Medium and Reference quality infills, weak floor failure mechanism can be generated when the ground-floor infill walls fail under horizontal seismic loading.

[2] f_v = cohesion
[3] f'_m = compressive strength of the masonry wall

Fig. 12b shows the great variability in the results expected for the range of masonry infills qualities considered. It is worth noting that the curves are not directly comparable because the building structural predominant period (T_1) will significantly change depending on the quality of the masonry infills and therefore different intensity parameter will be used for the risk assessment (for direct comparison between the vulnerability functions, they should be transformed –or derived from the beginning– for PGA or another equivalent IMs for all functions). In conclusion, the quality of the masonry infills in a school building (if not isolated from the main structure) will have a significant impact in the final vulnerability of the building. Therefore, it is highly recommended to consider the quality of the masonry infills as a critical variable for the assessment.



a) b)

Fig. 12 a) capacity curves using different masonry qualities (BS = Base Shear). b) vulnerability functions using different masonry qualities (MDR = Mean Damage Ratio).

4.4 Non-structural vulnerable elements

The objective of this sensitivity analysis is to identify the effect of considering different types of non-structural elements (NSE) in the loss calculation process. For this, mid-rise RC1 IB with high seismic design level (RC1/MR/HD) is selected since masonry walls do not interact directly with the reinforced concrete frames, which as showed in the previous section, highly affects the results. The following three conditions are considered: (i) no consideration of non-structural elements, (ii) poor quality fragile non-structural elements and (iii) high quality ductile non-structural elements. Table 7 to Table 9 present the component models for these three conditions. It is important to clarify that the non-structural elements for façade and internal partitions walls, presented in Table 8 and Table 9, are isolated from the structure and therefore does not affect the structural behavior of the RC moment resistant frames (RC1).

Table 7 Only structural elements component model

Group	Description	Quantity	Fragility specification code	*EDP*
Structural	Column-one beam	8	B1041.091a	Drift
Structural	Column-two beams	21	B1041.091b	Drift

Table 8 Poor quality component model

Group	Description	Quantity	Fragility specification code	*EDP*
Structural	Column-one beam	8	B1041.091a	Drift
Structural	Column-two beams	21	B1041.091b	Drift
Non-structural	Unreinforced Masonry (URM) facade	14	C1011.006a	Drift
Non-structural	URM wall	6	C1011.006b	Drift
Contents	Contents	13	E2022.010a	Drift

Table 9 High quality component model

Group	Description	Quantity	Fragility specification code	*EDP*
Structural	Column-one beam	8	B1041.001a	Drift
Structural	Column-two beams	21	B1041.001b	Drift
Non-structural	Confined masonry (CM) facade	14	C1011.001a	Drift
Non-structural	CM wall	6	C1011.001a	Drift
Contents	Contents	13	E2022.010a	Drift

Fig. 13 presents the vulnerability curves for each one of the cases explained above. From these results it can be concluded that variations on the order of 20% in the mean damage ratio could be expected when considering fragile NSE as compared with a building with no NSE for the lower ranges of seismic intensities. In addition, lower relative variations are expected in the higher range of seismic intensities, since global building collapses would control the losses in that intensity range.

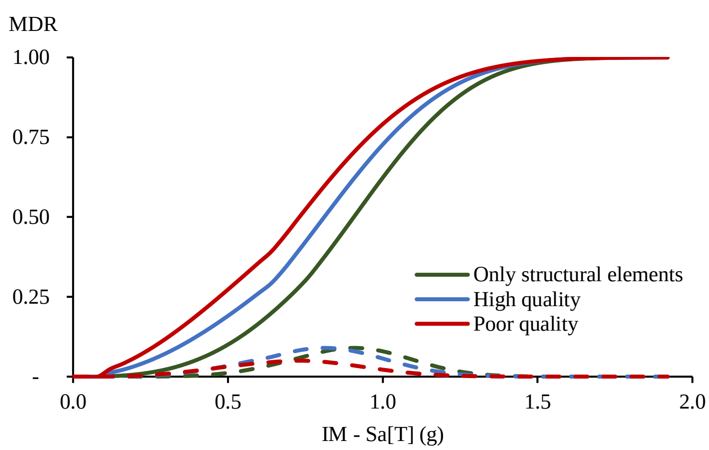
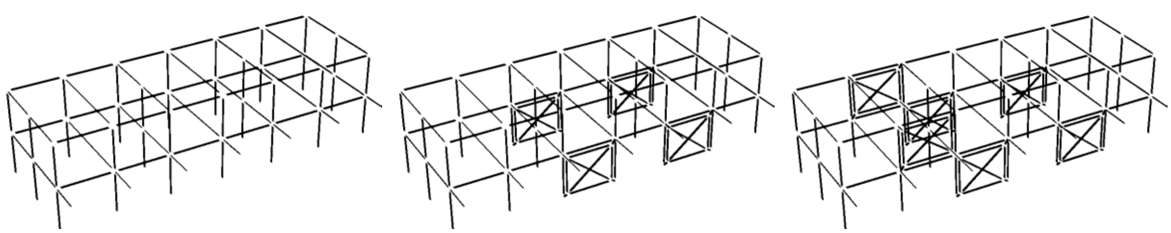


Fig. 13 Vulnerability functions using different component models (MDR = Mean Damage Ratio)

As a general recommendation, NSE shall be included in the vulnerability assessment when they represent a significant replacement value as compared to the structure itself, and when they present fragile behavior and significant damage during an earthquake (no seismic design). The consideration of the NSE in those cases will generate a significant increase in the mean damage ratio of the global building especially for the low range of seismic intensities and will therefore affect significantly the expected annual losses in the risk assessment process.

4.5 *Analysis type*

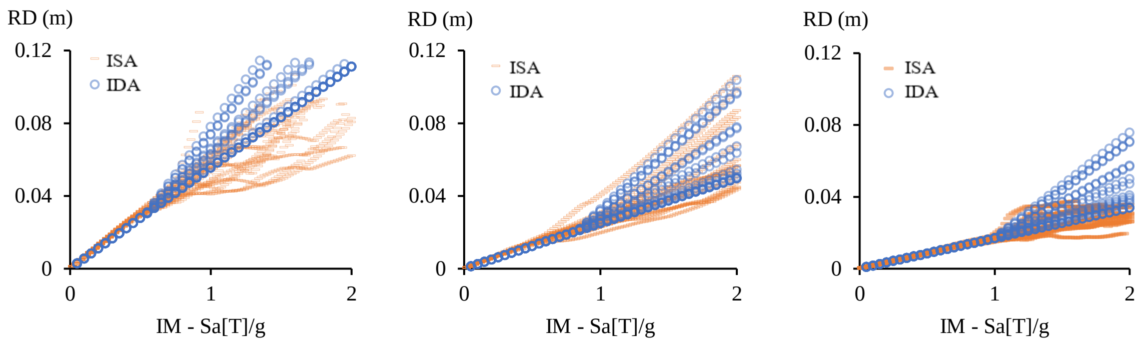
To establish the reliability of using the non-linear static N2 method, finally a comparison is carried out with results obtained by applying incremental dynamic analysis (IDA) approach to an IB without and with retrofitting. Literature highlighting non negligible differences between the nonlinear dynamic methods and nonlinear static procedures usually focuses on high rise buildings, whereby upper modes have greater contribution (Han and Chopra 2006; Reyes and Chopra 2011). However, the proposed methodology focuses on low and mid-rise school buildings, usually regular in height and floor plan. Available calibrations and results for these types of buildings show good correlation between the simplified methodology and more complex procedures like incremental dynamic analysis (Mwafy and Elnashai 2001; Dolšek and Fajfar 2004, 2008; Faella et al. 2008; Bhatt and Bento 2011; Causevic and Mitrovic 2011; Gehl et al. 2014; Rossetto et al. 2014). As an additional comparison for validation purposes, results from both incremental dynamic analysis (IDA) –obtained through a non-linear time history analysis using the commercial software Perform3D with the modelling considerations presented in Table 2– and the incremental static analysis (ISA) proposed in this methodology are compared. Fig. 14 presents the three selected buildings models, which are based on a common school typology found in Peru (Fernández et al. 2019). The Basic IB is a RC1, a mid-rise building with low design level (RC1-MR-LD). The second model is the same building with a retrofitting system of steel diagonals in the first story which shift is taxonomy to a RC4-MR-LD. The third model includes the retrofitting system at both stories obtaining a type RC4-MR-HD.



a) b) c)

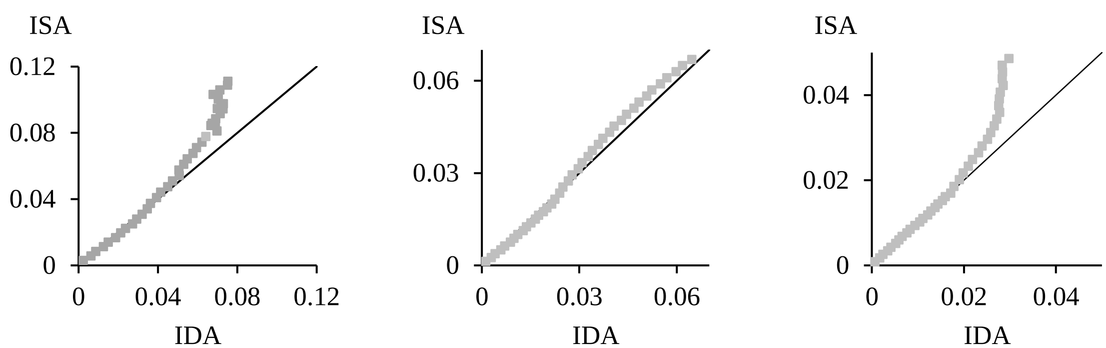
Fig. 14 School buildings analyzed for the IDA vs. ISA comparison. a) RC1-MR-LD, b) RC4-MR-LD, and c) RC4-MR-HD

Fig. 15 shows the resulting Roof Drift (RD) obtained with ISA and IDA methodologies using the set of ground motions presented in section 3.2. The results show a high correlation between the mean roof drift found with IDA and ISA procedures. Results show that the maximum roof drift found in each building is similar for both methodologies. In addition to the above, Fig. 16 presents the mean RD obtained with each methodology for all three typologies. Based on these results, the mean square error (MSE) between ISA and IDA results in 0.016, 0.002, and 0.007 for RC1-MR-LD, RC4-MR-LD, and RC4-MR-HD, respectively. It is important to note that the MSE is very small for all the buildings, particularly for RC4-MR-LD. Additionally, the mean absolute percentage error (MAPE) between methodologies is about 19.1%, 8.4%, and 16.3% for RC1-MR-LD, RC4-MR-LD, and RC4-MR-HD, which is relatively low considering the simplification of the ISA methodologies. The obtained relative error may be explained on the simplifications of the ISA method, particularly with working with the resulting spectrum and not with the ground motion itself. This comparison suggests that both methodologies yield similar results, despite being limited to the regular buildings analyzed herein. This type of results will not necessarily be obtained if the analyzed buildings are taller than three stories or have any irregularity. For these types of buildings, modifications of the N2 method have been proposed (Kreslin and Fajfar 2012; Magliulo et al. 2012), but are beyond the scope of the present study.



a) b) c)

Fig. 15 EDP using ISA and IDA methodologies (RD = Roof Displacement). a) RC1-MR-LD, b) RC4-MR-LD, and c) RC4-MR-HD



a) b) c)

Fig. 16 Mean Roof Drift (m) using IDA vs. ISA. a) RC1-MR-LD, b) RC4-MR-LD, and c) RC4-MR-HD

Fig. 17 presents the resulting vulnerability functions for the three buildings, using both methodologies. From these results it can be concluded that the N2 method, which in general is much simpler and faster to run, gives comparable results with the more refined and time-consuming IDA method of analysis. It is also important to note that the N2 method considers the non-linear behavior of buildings (hysteretic behavior, pinching and buckling of rebar among others) in a simplified way through the ductility and the pushover of the building. However, from the results it is possible to establish that both methodologies generate similar mean and dispersion values of the vulnerability function. For the vulnerability assessment of typical school buildings, the N2 method is clearly a reliable option to determine structural response. Caution shall be exerted when considering non–typical school buildings whose behavior is influenced by irregularities, variations in height, combined structural systems or any other special characteristic.

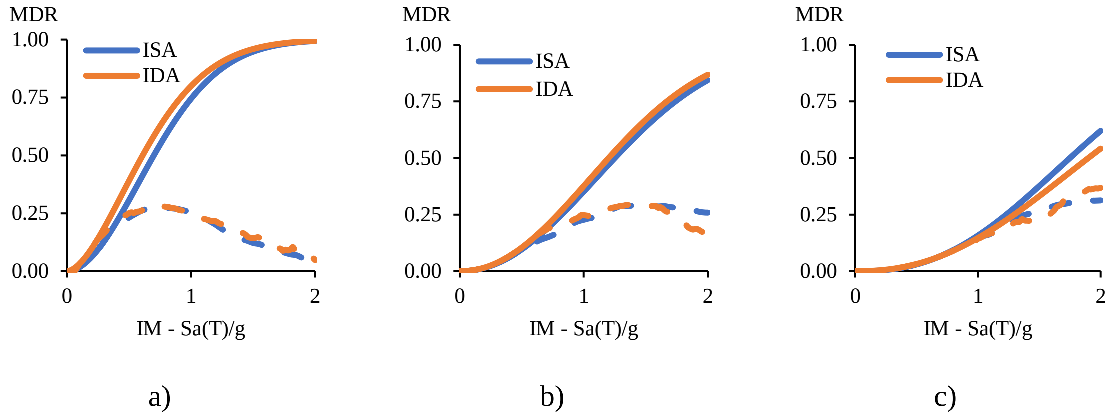


Fig. 17 Vulnerability functions (MDR = Mean Damage Ratio): a) RC1-MR-LD, b) RC4-MR-LD, and c) RC4-MR-HD

5 Summary and conclusions

The present study shows how by classifying typical global RC school buildings according to their construction characteristics, and focusing on these when developing structural modelling, their specific structural system and their corresponding design level, their seismic response and vulnerability can be clearly characterized and quantified with good levels of confidence. As a first conclusion from the work presented in this study, the structural system is the main parameter affecting seismic response and hence, economic losses. As a matter of fact, typologies such as RC1, RC3 and RC5 can reach 100% of damage for low IMs (<1.5g), while RC2 and RC4 reach a maximum level of damage between 20 and 50% for a maximum IM of 2.0g. Another relevant result is that the vulnerability functions depend also on the design level, with a higher impact in this regard for the RC1 than in the RC2, RC3 and RC4. Indeed, the variation between design level for the RC1 IBs can increase until 80% for the same IM, while for RC2, RC3 and RC4 IBs it reaches a maximum value of 10% for the range of IMs analyzed. This impact can be explained by the infills high contribution in the final structural behavior of the entire system, even in ductile structures. Ductility also plays an important role in determining seismic response and vulnerability, as seen in the RC4 typologies and the RC1 high design IB. In general terms, since a higher vulnerability was identified for RC1, RC3, and RC5 compared to the results for RC2 and RC4, the former systems may be prioritized in a risk reduction retrofitting program, although this type of preliminary assessment for prioritization should weigh other factors as well, such as economic and technical viability, relative importance (e.g., terms of student demand), or other functional and operative considerations.

The proposed methodology provides an efficient and reliable procedure to assess the seismic vulnerability in school buildings and may be widely applied in different contexts for school infrastructure worldwide. In addition, the methodology explicitly quantifies the uncertainty associated with the vulnerability value, allowing for an unbiased and consistent strategy to assess the vulnerability of school buildings. This assessment is the first step to quantify the risk level of school infrastructure in a specific region and to develop a set of strategies with the aim of reducing vulnerability and protecting the students and all the community associated to school infrastructure. These vulnerability results can be used and helps for the developing intervention strategies at scale, such as the incremental retrofitting. As presented in section 3 the main parameters like structural system, height and design level highly impacts the resulting vulnerability of buildings. However, secondary parameters such as the infills type or the foundation also affects the vulnerability as showed in section 4, and therefore should be considered as they directly impact the economic losses and costs. Other factors such as variation in story height, vertical or horizontal irregularities were not considered in this study and further analysis is needed to understand the full spectrum of vulnerability of other type of RC school building.

Acknowledgments

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Declarations

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