GLOBAL PROGRAM FOR SAFER SCHOOLS - GPSS

GLoSI

The Global Library of School Infrastructure

Technical Report
on
Data Collection, Taxonomy, Index Buildings and Fragility/Vulnerability Assessment

<u>Load Bearing Masonry and Reinforced Concrete School Buildings</u>

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1. INTRODUCTION

The main aim of the Global Program for Safer Schools (GPSS) of the Global Facility for Disaster Reduction and Recovery (GFDRR) of the World Bank is to make the educational facilities and the communities they serve more resilient to natural hazards. In order to meet those objectives, within the program a collaboration platform was developed at global scale that enables sharing of risk information and measuring progress towards comprehensive safety of school facilities. At national level, the program supports and coordinates ongoing projects within each country in order to guarantee its success.

School buildings within a country or in different regions of the world usually present a great variation in terms of construction types, structural systems, non-structural components and functionality, among other physical characteristics. Further, in many cases, school construction follows a standard design in a country/region which evolves over time according to the development of new knowledge, construction technology, building regulations, cultural and social influences and educational policies, specific of that region. The development of a building classification system is an important step in the seismic risk assessment and risk reduction planning of these school infrastructure, as it introduces a common framework for seismic risk related to school infrastructure which delivers a unified approach at global level. On the other hand, some building construction characteristics become repetitive within a region (e.g. Latin America) or even at global level, and therefore a detailed and unified classification system for the seismic risk assessment of school buildings worldwide is technically feasible. Studies of seismic damages in past events show that some types of construction tend to be more vulnerable than others. For example, a building with unreinforced masonry walls can be expected to be much more vulnerable than those with confined masonry walls. Thus, preparation of a building type catalogue for the seismic vulnerability assessment of different buildings is a requirement to facilitate the program objectives (D'Ayala et al., 1997; Coburn and Spence, 2002). Moreover, some important characteristics of school buildings, such as geometry and layout, are strictly related to their function, such as minimum dimensions of classrooms or particular ventilation and lighting requirements, dictating the size of windows. These are recurring worldwide and irrespective of the structural form used. It is therefore logical to develop a building taxonomy, tailored to school infrastructure, which supports the identification and classification of school infrastructure at regional, national and global level. This tool is a key component of any risk assessment process related to school buildings at large scale and the basis for the design of risk mitigation programs.

Within the GPSS program, the Global Library of School Infrastructure (GLoSI) serves as a repository of data and information about the structural performance of school buildings and alternatives to reduce their seismic vulnerability. The purpose is to create baseline information on school infrastructure and its associated seismic risk by establishing, through the taxonomic classification a series of index buildings (IB) each representing a recurring type identified by studying national databases and surveying school infrastructure on site. A methodological approach is defined to derive both the seismic fragility and vulnerability of these school index buildings (IBs). Considering that each of those IBs represents a typology that can be found in several countries, a reliable analytical assessment of its expected seismic performance is an important contribution towards a robust seismic risk assessment process in any particular country or region worldwide. Finally, understanding the characteristics and components that control the vulnerability of any given building type, it is possible to establish targeted retrofitting measures which may effectively reduce the vulnerability and therefore generating the basis for a risk mitigation program in the school sector.

This report includes five main chapters and several annexes. Chapter 2 deals with the development of the GLoSI Taxonomy to classify school buildings. In the same chapter, the pre- and post-disaster data collection forms are also introduced. Chapter 3 establishes the criteria to define and characterize a group of building types (Index Buildings) that adequately represent the complete portfolio of school buildings worldwide. In Chapter 4, a methodological approach is defined to derive both the seismic fragility and vulnerability of selected school index buildings (IBs) and present the catalog of derived F/V functions. Chapter 5 presents some practical retrofitting options which will generate significant vulnerability reduction in the main IBs. Finally, Chapter 6 presents the main conclusions and recommendations at the present stage of development of the GPSS and the GLoSI. Annexes to the report include: Annex A – National Construction Typologies for Building Classification; Annex B – Pre-Disaster Collection Form; Annex C – Post-Disaster Collection Forms; Annex D – Taxonomy Excel Sheet; Annex E – Building Types Catalog; Annex F – Index building Catalog; Annex G – Suite of Seismic Records; Annex H – Software for seismic performance assessment using N2 Method; Annex I – Software for Least Square Method for Fragility Assessment; Annex J – Software IT-FUNVUL V2.0 for Component Based Vulnerability Assessment; Annex K – Fragility/Vulnerability Catalog and Annex L – Vulnerability Reduction Solutions.

2. GLoSI TAXONOMY

2.1 Introduction and Objectives

The response of a building construction to earthquakes is not only dependent on the seismic intensity it experiences during a particular event but is equally dependent on its construction characteristics such as the main structural system, the lateral load resisting mechanisms, the materials used, the building height, the quality of construction among others. Many school buildings in rural areas are poorly designed and constructed (i.e. non-engineered structures) by locals without any technical design using locally available materials and reflecting local construction traditions. Thus, it is very important to study the seismic vulnerability of school buildings to understand their seismic risk, and to plan and design the possible risk reduction strategies, the financing strategy and the recovery plan from potential future seismic disasters.

There are sufficient statistical evidences from past destructive earthquakes to conclude that some types of constructions are more vulnerable than others. For example, adobe buildings are likely to experience more damage than brick masonry buildings for the same seismic intensity. Since it is not feasible to study the seismic vulnerability of individual school buildings one by one, buildings having similar main construction characteristics (i.e. main structural system, building height and seismic design level) are grouped into distinct categories, called building types. A building type represents a large class of buildings having same attributes of the main construction characteristics (i.e. main structural system, height range and seismic design level) but variation in the attributes of other construction characteristics such as diaphragm type, structural irregularities etc. Identification and selection of the representative attributes of these characteristics then results in one or more index buildings for a building type, the detailed seismic analyses of which collectively define the seismic performance of the building type. Index building is a representative building of a building type which has fixed attributes for all the construction characteristics. Development of a structural taxonomy is thus important for the seismic vulnerability assessment, seismic risk assessment and risk reduction planning/implementation on school infrastructure for safer communities.

Most of school buildings worldwide are made of load bearing masonry (unreinforced, partially reinforced, confined, reinforced, etc.) and reinforced concrete (moment resistant frames with or without masonry infills, combined systems, etc.) construction. Other construction types of school buildings such as steel framed, timber framed, prefabricated structures and others are also present locally in some countries/regions but not at global level. Some of these buildings are very old and with very poor seismic performance while some others have been recently designed and constructed using the most up-to-date building codes and practices leading to a very good expected seismic behavior. Thus, the development of a taxonomy will make it easier to distinguish each school building in terms of its seismic performance and will help in the overall process of seismic risk assessment and intervention prioritization.

This chapter focuses on the review and evaluation of available information on the construction characteristics of school buildings in order to develop a global building taxonomy for different construction types of schools, initially limited to load bearing masonry (LBM) and reinforced concrete (RC) framed structural systems. This initial building taxonomy is developed based on the information on school buildings available from recent World Bank engagements in selected countries from different parts of the world viz. Nepal, El Salvador, Peru and Kyrgyz Republic.

The main objectives of the proposed building taxonomy can be summarized as follows:

- Having a common language for seismic vulnerability and risk communication with respect to school infrastructure.
- Identification of the distinct global construction types of LBM and RC framed school buildings.
- Ranking of the vulnerability parameter from the generic to the specific, but also by relative importance in defining and characterizing the seismic response.
- Identification and description of various taxonomy parameters (and their associated attributes) that affect the seismic performance of LBM and RC framed school buildings.
- Characterization of different LBM and RC framed school building types (building type catalogue).
- Identification and definition of different index buildings (index building catalogue) representing each different building typology for the seismic vulnerability assessment of a population of LBM and RC framed school buildings.
- Accelerating the risk assessment process of the school infrastructure by using the results for some building types that are already studied in past in detail.
- Development and adoption of the possible economical retrofitting options per building type of school buildings.

The building taxonomy proposed in this study is similar in principle to the GEM Taxonomy (Brzev et al., 2013) and thus results in a similar style of taxonomy string. However, the present taxonomy is simpler and is focused on the structural characteristics of school buildings and its non-structural components. Further, the taxonomy parameters that form the classification systems are detailed and well defined with respect to the variations in the attribute and the range of the attributes based on the existing school buildings in the case study countries.

2.2 Review of Existing Building Classification Systems

Several building classification systems have been developed and are in use, some being developed considering global construction types and hence globally applicable (e.g. Coburn and Spence, 2002; Jaiswal and Wald, 2008; Brzev et al., 2013, FEMA, 2013) and some being of national or regional reference (e.g. ATC, 1985; Grünthal, 1998). The structural characteristics used in the early classification systems such as ATC-13 (ATC, 1985) or EMS scale (Grünthal, 1998) are very limited, and the corresponding building types are very generic. In ATC-13, for instance, developed by the Applied Technology Council for the seismic vulnerability assessment of buildings in California, USA, the classification of building structures is based on the construction material, the building height, the structural load bearing system and the design and construction quality. The EMS scale building typology catalogue (Grünthal, 1998) classifies buildings into construction types and sub-types based on the construction material. Similarly, Coburn and Spence (2002) have developed a classification system in which structures are broadly grouped into non-engineered building and engineered buildings and are further classified into several types based on the construction materials used. Recently, additional classifications systems have been proposed including several other important parameters so as to more accurately define the seismic response, such as diaphragm flexibility, structural irregularities, openings, behavior of non-structural components and others. U.S. Geological Survey's Prompt Assessment of Global Earthquakes for Response (PAGER) program developed a global construction type catalogue (Jaiswal and Wald, 2008; Jaiswal et al. 2011) based on the analysis of database from different countries across the world. It captures most of the key structural aspects that affect the seismic performance (i.e. material and type of load bearing structure, lateral resisting system, diaphragm type and height of the structure). It has been used widely in different regions across the world, to forecast the level of damage in the immediate aftermath of main shocks. This classification system does not explicitly rank the typology parameters in terms of their influence on the seismic performance. On the other hand, the Global Earthquake Model (GEM) global taxonomy system (Brzev et al., 2013) is based on the concept of ordering the taxonomy parameters from more generic to more specific, so that for each additional parameter considered, the resulting class is a subset of the one determined without that parameter. The system has two main categories: primary parameters describing general building characteristics (e.g. height) and secondary parameters (e.g. height above grade, story height etc.) describing the characteristics in more detail. This classification system is more comprehensive than the previous classifications and results in a unique taxonomy string to each building structure.

Although the above-mentioned classification systems include a wide variety of construction types and parameters, some (ATC-1985, EMS-98 scale, Coburn and Spence, 2000 etc.) have limited parameters and others (PAGER, GEM etc.) are too broad to be applied directly to typical construction types of school buildings and/or lack proper consideration/prioritization of different taxonomy parameters and the variations in the associated attributes. These classification systems are primarily focused on residential buildings, but the school buildings in many cases have different construction/architectural characteristics (e.g. large classroom size, large and many openings etc.) than that of the residential buildings. Moreover, some local construction types of school buildings (e.g. steel framed structure with masonry walls in Nepal, El Salvador and other countries) cannot be precisely categorized using the existing classification systems as these structures have two different and often independent structural systems (light steel framed structure and LBM wall system). Thus, a comprehensive building taxonomy specific to school buildings should be developed for the seismic vulnerability and risk assessment of school infrastructure.

2.3 Overall Process for the Development of the GLoSI Taxonomy

Since there are many different construction types of school buildings along with several variations in construction characteristics within a country and/or in different countries, the development of a global taxonomy for the building classification of these buildings is a complex process. Thus, it should be comprehensive yet with simple steps. The overall process of development of the GLoSI taxonomy follows the three main steps as shown in Figure 2-1, each of them is discussed in detail in the following sub-sections.

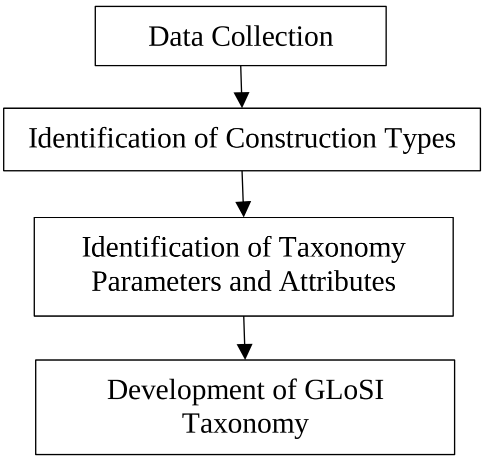


Figure 2-1 Overall process of development of the GLoSI taxonomy.

The first step involves the data collection and analysis of construction characteristics of school buildings at national level in each country and the comparison of construction types along with similarities and regional differences if any. This allows the identification of main global construction types of school buildings. **Annex A** presents a summary of construction types identified in different regions of the world based on the database on school buildings collected within the GPSS program which forms the basis of different types of main structural systems. The second step involves the identification and definition of the taxonomy parameters that affect the seismic performance of these school construction types. In each taxonomy parameters, the possible variations and ranges of the attributes at global level are also identified and are presented in 2.5. The final step involves the development of the comprehensive taxonomy (section 2.6) with all the taxonomy parameters and the associated attributes so that for each single school building, the selection of the appropriate attributes from each parameter ultimately results in a string, known as the taxonomy string which identifies the building classification of the school construction. This is further elaborated with example application of the GLoSI taxonomy to typical LBM and RC school buildings in section 2.8.

2.4 Data Collection

To assign a particular taxonomy to a school building, it is first necessary to obtain structural and architectural information in field or by photographs depending on the level of information available and the level of detail required by the taxonomy. This activity is best done by means of field inspections performed by technicians or engineers with some background in construction practices, structural systems and typical construction details that affect the expected seismic behavior of a building. Data collection process is a cumbersome task and is better to carry out in phases. Three different Tiers are proposed for data collection:

- Tier 1: school building information are collected to identify different building types on the basis of three main taxonomy parameters (discussed in detail later in section 2.3), data collection using the photographs collected by non-technical persons or rapid visual survey or the analysis of existing databases might be sufficient, although the analysis of these information should be carried out by experienced seismic/structural engineers.
- Tier 2: in order to completely define a taxonomy's string and hence to characterize a specific index building, more detailed level of structural as well as geometric information is required. The tier 2 data collection can be carried out by civil engineers with adequate trainings as it is mainly based on field observation and measurements. However, the evaluation of taxonomy parameters (section 2.5) requires seismic/structural engineering judgement and should be carried out by experienced seismic/structural engineers. Thus, the field data collection shall be done by trained civil/structural engineers in the field and then the evaluation of taxonomy parameters and classification of buildings shall be carried out by experienced seismic/structural engineers in office based on the collected data.
- Tier 3: it includes the collection of the intrinsic characteristics of building types which specifically refers to geometric dimensions, mechanical properties of material and structural details. A detailed assessment, including for example destructive tests to know the internal reinforcement details or non-destructive tests to establish material properties and detailed analysis using numerical models are usually required. This must be done for specific cases in order to define all relevant information to perform a reliable fragility/vulnerability assessment.

Figure 2-2 summarizes the data collection process and options within the overall process of building classification.

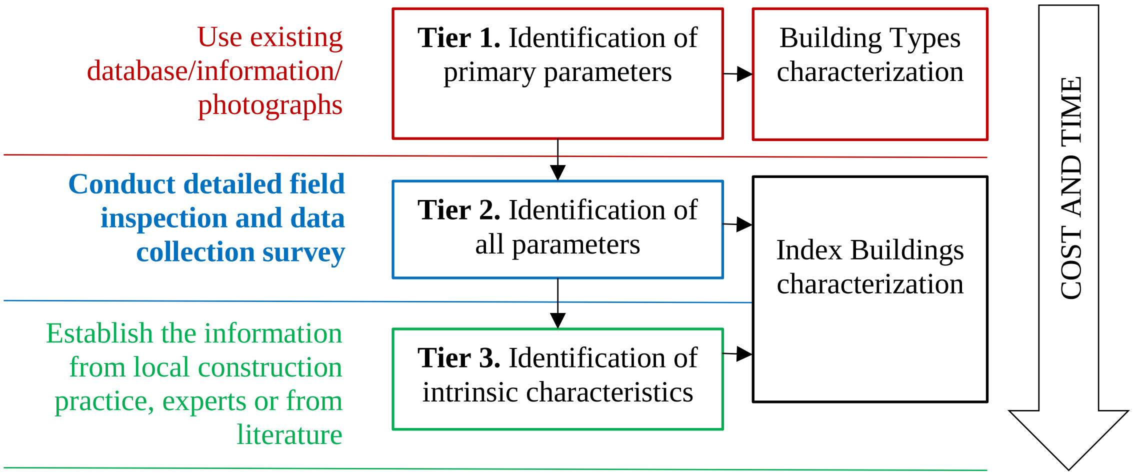


Figure 2-2 Overall process of building classification

In both cases the data collection should be consistent and standardized, with a flexible format applicable to a large number of building types, concise enough to require limited time on site, and recording observable quantities, rather than requiring judgement and interpretation, so as to avoid implicit biases by the surveyor.

In order to facilitate the data collection process using the proposed approach for Tier 2, a standardized, simple to use, self-explanatory and globally applicable Pre-Disaster Data Collection Form (PD-DCF) (along with a user manual) has been developed tailored specifically for school buildings (see **Annex B**). Table 2-1 summarizes the main sections contained in the PD-DCF. These correspond mostly to the taxonomy parameters explained in section 2.5.

Table 2-1. Main elements of the Tier 2 DCF.

Section	Identification
ID	Building ID
P0	Building category
P1	Main structural system
P2	Height range
P3	Seismic design level
P4	Diaphragm type
P5	Structural Irregularity
P6	LBM: wall panel length, RC: span length
P7	LBM: wall openings, RC: pier type
P8	Foundation type
P9	Seismic pounding risk
P10	Effective seismic retrofitting
P11	Structural health condition
P12	Non-structural components

In order to support the post-earthquake damage assessment in future events (e.g. to provide guidance on evacuation, further inspections or repair and retrofit etc.), to gather information for validating the fragility/vulnerability assessment in the framework of the GLoSI library as well as to collect the data and information on school portfolio in a country/region, a simplified Post-Disaster Data Collection Form has been developed and is included in **Annex C** along with a user manual. Such forms are useful in emergency response phase after an earthquake as well as a tool for collecting data on schools where no such information is collected in the past. For example, the SIDA database (SIDA, 2016) collected in Nepal after 2015 Nepal earthquake is very comprehensive and useful which collected data on the damaged as well as undamaged school facilities in the affected regions.

2.5 Identification and Definition of Taxonomy Parameters

The taxonomy parameters (i.e. seismic vulnerability parameters) are the construction and functional characteristics of a building structure that affect the seismic performance of the structure. Based on the literature review and the available information on schools in the case study countries, several taxonomy parameters that affect the seismic performance of school buildings are identified. These parameters can be broadly categorized into following three types:

a) Primary parameters
b) Secondary parameters
c) Intrinsic parameters

The primary parameters are the main parameters that highly affect and govern the expected seismic behavior of a school building and are: main structural system, height range and seismic design level. The attributes associated with these parameters collectively define a building type. The main structural system defines fundamental aspects of the expected seismic behavior such as the flexibility to horizontal loads, the lateral strength of the building and the capacity to deform into the inelastic range, better known as the ductility of the system. The height of the building is another important parameter which controls the vibrational characteristics of a building structure. Finally, the seismic design level corresponds to the quality of construction materials, level of workmanship, structural detailing and integrity of the structural elements in the construction of the building in terms of earthquake resistance. A poorly designed building will certainly have poor seismic performance and vice-versa.

The secondary parameters are a group of characteristics that will have a key role in modifying the usual expected behavior of a building that is already classified in a particular building type according to the three main parameters. These are: diaphragm type, structural irregularity, wall panel length/span length, wall openings/pier type, foundation type and flexibility, seismic pounding risk, structural health condition and non-structural components.

Intrinsic parameters are the building-specific characteristics such as the geometrical dimensions, architectural layout and the mechanical properties of the construction materials/structural elements. Even though these are not explicitly included in the taxonomy string, these parameters are required for the complete definition of index buildings and for the development of reliable analytical models. The seismic analysis on these index buildings allows the assessment of the seismic capacity with respect to different levels of intensity measures and hence to derive the representative fragility/vulnerability functions for different building types.

Twelve different taxonomy parameters (3 primary and 9 secondary parameters) are listed in Table 2-2 along with a brief description of each parameter and are discussed further in the sub-sections 2.5.1 to 2.5.12.

Table 2-2. Taxonomy parameters for building classification

No.	Taxonomy Parameter	Description
1	**Main Structural System**	Deals with the main construction material and lateral load resisting system
2	**Height Range**	Deals with the dynamic response of the structure and its fundamental period
3	**Seismic Design Level**	Deals with the quality of construction materials, level of workmanship, structural detailing and the inclusion of seismic enhancement measures
4	**Diaphragm Type**	Deals with the roof/floor diaphragm behavior (flexibility)
5	**Structural Irregularity**	Deals with the abrupt changes in strength or stiffness in plan and elevation
6	**Wall Panel Length/Span Length**[1]	Deals with the unrestrained wall panel length between two cross walls/buttresses in LBM construction Deals with the horizontal clear span of the typical bay in RC framed structures
7	**Wall Openings/Pier Type**[1]	Deals with the size and number of openings within a typical wall panel in LBM construction Deals with the vertical elements in the lateral load resisting system in RC construction
8	**Foundation Type**	Deals with the material and type of foundation structure as well as the soil type
9	**Seismic Pounding Risk**	Deals with the susceptibility to damage due to the different vibrational characteristics of adjacent buildings
10	**Effective Seismic Retrofitting**	Deals with whether the structure is effectively retrofitted or not in the past
11	**Structural Health Condition**	Deals with the condition of the building in terms of damage or deterioration
12	**Non-Structural Components**	Deals with the vulnerability/hazardousness of non-structural components

2.5.1 Main Structural System

The main structural system is the first parameter in the classification system. The structural system determines the structural behavior (brittle or ductile) and the collapse mechanisms. In LBM constructions, the unit and binding agent of the masonry fabric (e.g. field stone in mud mortar, bricks in cement mortar etc.) greatly affects the seismic performance as well as the vulnerability. For example, a field stone in mud mortar masonry construction has a poor seismic performance compared to brick in cement mortar masonry construction. Mud mortar is generally weaker than the cement sand mortar and provides poor tensile, cohesion and frictional resistances. Both bricks and concrete blocks have regular rectangular shape and size, thus these two are collectively known as rectangular block unlike dressed stone which often has larger and varying shape and size and hence is differentiated.

For RC framed structures, the collapse mechanisms are affected primarily by the presence or absence of stiff infill walls. Also, if the infills are not full story height, the configuration may cause a failure

[1] These parameters have different definitions and attributes depending on the main construction type (i.e. LBM or RC).

mechanism known as short column failure or captive column. In case the structural system is combined (reinforced concrete walls and frames) the behavior would be totally different as it would have seismic design in most cases, thus a ductile collapse mechanism is expected. The last case considered correspond to RC constructions with no clearly defined structural system, usually non-engineered and built by means of self-construction processes. The main structural systems (identified above in section 2.3) for the GLoSI taxonomy are summarized in Table 2-2.

For the particular case where a previously retrofitted building is to be classified, the proposed methodology requires that the present structural system and characteristics (and not the original ones) are considered in the classification process.

Table 2-3. Main Structural System

No.	Parameter	Attributes	Commentaries
1	**Main Structural System**	**For LBM:** **A** - Adobe **UCM/URM - Unconfined/Unreinforced Masonry** **UCM-URM1** - Dry Stone Masonry **UCM-URM2** - Rubble Stone in Mud Mortar Masonry **UCM-URM3** - Dressed Stone in Mud Mortar Masonry **UCM-URM4** - Rectangular Block in Mud Mortar Masonry **UCM-URM5** - Rubble Stone in Cement Mortar Masonry **UCM-URM6** - Dressed Stone in Cement mortar Masonry **UCM-URM7** - Rectangular Block in Cement Mortar Masonry **CM** - Confined Masonry with Rectangular Block in Cement Mortar Wall **RM** - Reinforced Masonry with Rectangular Block in Cement Mortar Wall **SFM – Light Steel Frame with LBM walls** **SFM1** - Light Steel Frame with Stone in Mud Mortar Wall **SFM2** - Light Steel Frame with Rectangular Block in Mud Mortar Wall **SFM3** - Light Steel Frame with Stone in Cement Mortar Wall **SFM4** - Light Steel Frame with Rectangular Block in Cement Mortar Wall **SFM5** - Light Steel Frame with Confined Masonry Wall **SFM6** - Light Steel Frame with Reinforced Masonry Wall **For RC frames:** **RC1** - Bare Frame **RC2** - Infilled Frame **RC3** - Short Column Frame **RC4** - Dual or Combined Frame **RC5** - Non-Engineered Frame	Description of each attribute is given after the table

A

These are generally unreinforced masonry buildings having adobe masonry (sun-dried mud bricks with mud mortar) walls as the main lateral load resisting system.

UCM-URM

These are masonry school buildings with unconfined/unreinforced masonry walls in the main lateral load resisting system. These are further sub-divided into different categories (see Table 2-3) depending on the type of units and mortar material used in the masonry considering that different combination of different units and mortars results in different seismic performance of the building.

CM

These are confined masonry school buildings in which the masonry walls are confined with RC columns and beams (known as tie-columns and tie-beams) of relatively small cross-section for improving the integrity of the walls. The level (density) of confinement can vary within a country or at regional level and affects the seismic performance. For example, in confined masonry school buildings in El Salvador, the confinement is applied around the openings as well, while in India, this construction type of school buildings in many cases lack confinements around the openings.

RM

These are reinforced masonry school buildings which have reinforced masonry walls in the main lateral load resisting system. The reinforcement material considered in this study is the steel reinforcement bars. The type of reinforcement (whether it is horizontal only, vertical only or both) and the density of reinforcement (amount and spacing of reinforcement) affects the seismic performance of these buildings.

SFM

This construction type of school buildings has light steel framed structure with load bearing masonry walls. This construction type is further sub-divided into different categories (see Table 2-3) depending on the type of load bearing masonry walls considering that different load bearing wall type results in different seismic performance of the building.

RC1

Reinforced concrete moment resistant frames with/without in-fill walls that do not contribute to lateral stiffness. Masonry infill walls are well separated from the columns by expansion joints. Joints are usually filled with elastic sealer. Also, when partitions are made using light and/or flexible infills such as drywall.

RC2

Reinforced concrete moment resistant frame with in-fill walls as stiffening element. In this kind of structures, the masonry walls usually go from the floor to the roof. The walls may have window openings. Infill walls are not separated from the RC structure. Since the masonry walls are not attached to the columns and usually have no internal reinforcement, walls may present an out-of-plane type failure fall.

RC3

Reinforced concrete moment resisting frames with masonry infill walls in contact with the structure. Masonry walls include uniform openings along the longitudinal direction of the building generating the possibility of a short column type of failure (“captive column”). This type of failure occurs when the lateral displacements concentrates in the free portion of the column, generating greater shear forces and hence an anticipated column failure mechanism. Since the masonry walls are not attached to the columns and usually have no internal reinforcement, walls may present an out-of-plane type failure fall.

RC4

Reinforced concrete combined or dual system. These are structures which includes two different main lateral load resisting system usually a reinforced concrete moment frame with steel braces or reinforced concrete walls that increase the stiffness of the system. The moment resistant frame can be designed to withstand only gravity loads or gravity loads and a percentage of lateral loads.

RC5

Non-engineered reinforced concrete structure. It usually includes a certain distribution of columns that may not correspond in all floors. Slabs usually consists of a solid or one direction joists slab without beams or girders. The structural elements may not conform standard or continuous moment resistant frames. Partition walls and facades are usually built with unreinforced masonry in contact with the structural elements, providing some initial apparent stiffness.

2.5.2 Height Range

Building height is one of the most important characteristics of a building controlling the dynamic behavior during earthquake ground motions. It affects the natural period of vibration as well as modes of vibration of a building during earthquakes. Under similar design and seismic intensity, high-rise buildings are subjected to more deformation (i.e. are more flexible) and higher modes come into play during seismic excitation, and hence become more vulnerable. The LBM school buildings are mostly single storied while few 2 – 5 stories masonry school buildings are also present, especially in urban areas. Most RC school buildings usually are 2 storied but there also exists up to 6 stories or more. In order to develop a uniform classification system and since most of the school buildings are usually 1 to 6 storied in height, the present study categorizes LBM and RC school buildings into low-rise (single-story), mid-rise (2 – 3 stories) and high-rise (4+ stories) which include buildings up to 6 stories.

Table 2-4. Height Range

No.	Parameter	Attributes	Commentaries
2	**Height Range**	**LR()** - Low Rise **MR()** - Mid Rise **HR()** - High Rise	-**LR**: single story -**MR:** 2 to 3 stories -**HR:** 4+ stories -Exact number of stories to be given in the bracket.

2.5.3 Seismic Design Level

The seismic design level of a building structure highly affects its seismic performance. In the present classification systems, the seismic design level of a building structure represents the overall quality (workmanship) of construction, quality of materials used, level of connectivity within the individual elements and integrity of the overall structure which are often prescribed in seismic design codes as fundamental to attain a given level of seismic capacity. For LBM buildings, even though there might not be explicit seismic code provisions, if the building standards and good construction recommendations or bylaws are followed, a building is considered as a well-designed structure if it meets the following conditions: it has good workmanship in the construction of individual walls; the walls are properly connected to each other and the horizontal components (floors/roof) have sufficient in-plane stiffness as well as good connections to the walls. The seismic design at wall level includes the use of good quality mortar, provision of strong type of bond pattern of brick/stone, minimum openings, proper connection between wall leaves (e.g. using thorough stone in stone masonry) and other similar features (D’Ayala, 2008). Similarly, the level of connection between the walls can be made robust with the use of corner quoins or vertical reinforcements at the cross-wall corners, using seismic bands (ties) such as sill bands, lintel bands and floor bands etc. (Bothara et al. 2002). The connection of horizontal structures (floors, roof) with the walls can be made stronger by providing proper anchorage, ties, pegs, etc (Magenes, 2006). Poorly designed masonry buildings are prone to significant seismic damage as evidenced by the reconnaissance survey of recent earthquakes: e.g. 2009 L'Aquila earthquake, (D'Ayala and Paganoni, 2011), 2015 Nepal Earthquake (Bhagat et al., 2017) etc.

In RC case the seismic design level is defined as follows: for structures with no seismic design (only gravity loads) the design level is considered as poor. If the structure is designed for a low seismic hazard level (e.g. PGA<0.1g) but no seismic detailing is provided, then it would be a low design level. If the structure is designed for a medium hazard zone (e.g. PGA<0.2g) and there are some good considerations of steel detailing, then the design level is considered as medium. The last case is a structure designed for a high seismic hazard zone (e.g. PGA>0.25g) with conservative reinforcement detailing, in this case the design level is considered high. The poor and low designs are expected to have a fragile collapse mechanism and very low lateral capacity, opposed to medium and high design which have major capacity and ductile collapse mechanisms.

Very often, the date of construction is used as a proxy for the seismic design level, by considering it as an indication of the seismic design codes in force in the country at the time the building was built. As seismic codes' provisions have improved with time, it is generally assumed that more recent buildings will have better seismic design and therefore will perform better than older buildings. In the case study countries, many older school buildings (especially LBM construction) exist, which were not designed for seismic resistance (e.g. in Nepal) or were designed following earlier (now outdated) seismic design codes. However, local seismic enhancement measures (such as the use of through stones, timber tying elements, infill walls isolation, concrete walls addition, etc.) have been included in these older constructions in many cases, which should be accounted for in the assessment of the seismic design level. Moreover, in countries such as Nepal or Perú, it has been found that school buildings were mostly constructed by the local communities in the past without following the seismic codes or guidelines even after its existence in the country (Dixit et al., 2014; Yamin et al., 2015). Thus, several factors such as the designer and contractor (e.g. government, community, private contractor etc.), code enforcement capacity in the country, workmanship and level of quality control during construction influence the seismic design level and should be assessed prior to assign a design level class to a specific building. Notion of the seismic building culture of the country and its evolution is essential. In the present study, four different seismic design levels are defined: poor, low, medium or high seismic design level. These are summarized in Table 2-5.

Table 2-5. Seismic Design Level

No.	Parameter	Attributes	Commentaries
3	**Seismic Design Level**	**PD** - Poor Design **LD** - Low Design **MD** - Medium Design **HD** - High Design	For LBM: - **PD:** the building is quite old, quality of construction material and workmanship are poor, and there are none of the seismic enhancement measures - **LD**: the building is quite old, quality of construction materials and workmanship are fair, and there are few seismic enhancement measures (i.e. corner quoin, through stone etc.) mainly at wall level

			- **MD**: the building is quite new and quality of construction materials and workmanship are good, there are several forms of seismic enhancement measures (i.e. corner quoin, through stone, lintels above openings etc.) - **HD**: the building is new and quality of construction materials and workmanship are very good, there are several forms of major seismic enhancement measures (i.e. corner quoin, through stone, lintels above openings, floor level band beams, intermediate band beams/corner stitches etc.) For RC: - **PD:** Poor design. Building may have been only designed to withstand gravity loads and may have only a very small resistance to lateral loads. - **LD:** Low design. Building is designed for low lateral loads and hence no seismic confinement stirrups exists in the end of the elements (spacing between stirrups greater than d/2). Minimum dimensions in structural elements (>20cm). Fragile collapse mechanism expected and low lateral capacity. - **MD:** Medium design. Building is designed for a medium seismic hazard zone with specific requirements as continuity in the longitudinal reinforcement of the elements, confinement of stirrups at the ends of the elements with a separation equal to or less than d/2 and minimum dimensions in structural elements (>25cm). Nonstructural elements are designed to withstand seismic forces. Ductile collapse mechanism expected, high capacity and ductility. - **HD:** High design. Building is designed for a high seismic hazard zone with specific requirements as continuity in the longitudinal reinforcement of the elements, confinement of stirrups at the ends of the elements with a separation equal to or less than d/4 and minimum dimensions in structural elements (>30cm). Nonstructural elements are designed to withstand seismic forces. Ductile collapse mechanism expected, very high capacity and ductility.

2.5.4 Diaphragm Type

Since the floors and roof are key horizontal components of the seismic load resisting system, the seismic performance of a building is influenced by the flexibility of these elements. If the floors/roof have significant in-plane stiffness, several times greater than the lateral stiffness of the vertical resisting system and are properly connected to the vertical load resisting elements (e.g. LBM walls or RC frame system), it can be assumed that the lateral displacement at the floor/roof level is constant for all structural elements connected to that floor level. This in turns provide a more equal redistribution of the lateral forces among all structural elements in proportion of their stiffness, and hence the structures are best suited to resist the lateral forces and have a robust behavior. Such types of floors/roof structures are referred to as rigid diaphragms and they are realized by building reinforce concrete slab or steel beams system sufficiently braced to avoid relative in-plane displacements. The stiffness of the horizontal structure play an important role in controlling the global box-type seismic behavior (i.e. all the columns or walls acting simultaneously). On the other hand, a floor/roof structure with low in-plane stiffness

and/or poor connection to the lateral load resisting elements is unable to impose a common lateral displacement at the floor level and hence the individual lateral load resisting elements behave independently during the earthquakes. Such types of diaphragms are categorized as flexible diaphragms (e.g. unbraced timber/steel or prefabricated concrete slabs) and often induce poor seismic performance in a building structure. These are listed in table
Table 2-6.

Table 2-6. Diaphragm Type

No.	Parameter	Attributes	Commentaries
4	**Diaphragm Type**	**FD:** Flexible diaphragm **RD:** Rigid diaphragm	A rigid diaphragm should have: 1. Floors/roof structure with sufficient in-plane stiffness such as: • RC flat slab • Reinforced Brick Concrete (RBC) slab • Conventional slabs supported with concrete joists • Composite (steel and RC) deck if properly braced and connected • Braced timber or steel framework 2. Good connection of the floors/roof to the lateral load resisting system such as: • Monolithically connected to the walls or columns and beams with proper anchorage (e.g. tied with the reinforcing bars) - If a floor/roof structure doesn't meet both of the above-mentioned criteria, it is considered to be a flexible diaphragm.

2.5.5 Structural Irregularity

Structural irregularities (horizontal, vertical) tend to make structures more vulnerable than simple and regular structures. Horizontal (plan) irregularity describes the building's irregular (e.g. rectangular long, T-, C- or H-shaped) foot print or unsymmetrical positioning of lateral load resisting elements, whereas vertical irregularity includes the variation in story height or mass or stiffness over the building height. If the building plan shape is irregular or longer in one direction or openings are distributed unevenly, it experiences torsional effects and hence increased shear during seismic loadings (Bothara et al, 2002; Erberik, 2008). Different types of combination of irregularities and their consideration as the attributes of the structural irregularity parameter are summarized in Table 2-76.

Table 2-7. Structural Irregularity

No.	Parameter	Attributes	Commentaries	
5	**Structural Irregularity**	**NI** - No irregularities **HI** - Horizontal **VI** - Vertical **HV** - Both Horizontal and Vertical	**HI** might include -Torsional Irregularity -Reentrant corner -Diaphragm discontinuity -Out-of-Plane Offset -Non-parallel system	**VI** might include -Stiffness-Soft Story -Mass Irregularity -Vertical Geometric Irregularity -In-Plane Discontinuity in lateral load resisting elements -Weak Story

2.5.6 LBM: Wall Panel Length; RC: Span Length

LBM

In LBM buildings, wall panels are susceptible to out-of-plane damage under seismic loading. Their vulnerability is directly proportional to the unrestrained length of a wall. This is mainly due to the two-way bending the walls are subjected to as well as the low moment of resistance (i.e. flexibility) along the out-of-plane direction (Murty, 2003; Abrams et al., 2017). For example, in the 2008 Wenchuan earthquake, main building of a primary school collapsed, while the adjacent dormitory of the same construction type, which had smaller rooms and more cross walls survived the earthquake (Rodgers, 2012).

Masonry walls are generally restrained by cross-walls or piers or buttresses. Several studies and seismic design codes have thus suggested a limit on the permissible length as a function of thickness of the URM walls i.e. usually less than 12 times the wall thickness (e.g. NBC, 1994). Similarly, for confined masonry, it is recommended to be less than 4 m (Meli et al., 2011; AIS, 2001). Thus, the unrestrained wall panels are categorized into two types: long panels and short panel. In stone masonry school buildings, the thickness is generally higher (more than 400 mm) hence the walls are usually short panels in length, but in brick masonry construction where the thickness is generally low (250 mm to 400 mm), similar lengths of walls make them long panels. However, attention needs to be paid as to how good is the connection of the masonry leaves across the thickness of the wall for stone masonry walls. Indeed, if there are several leaves and they are not well connected with regularly spaced through stone then the wall slenderness should be computed with reference to one leaf only, not the whole thickness of the wall. For LBM walls, the unrestrained wall panel length thus has two possible attributes: Short Panel or Long Panel as presented in Table 2-8.

RC

The span length in RC structures is a very important indicator of general dimensions and vulnerability. It measures the distance between columns and classifies the flexibility of the frame by its beam length. It should be noted that short span beams tend to be stiffer and are more likely to attract high level of shear and fail in shear rather than bending. Conversely beam with long spans will tend to be very deformable and fail in bending. This parameter classifies span length as Short Span (SS) or Long Span (LS). Clear spans below 6 meters length is considered to be a short span.

Table 2-8. LBM: Wall Panel Length. RC: Span Length

No.	Parameter	Attributes	Commentaries
6	**LBM: Wall Panel Length** **RC: Span Length**	**SP** - Short Panel **LP** - Long Panel **SS** - Short Span **LS** - Long Span	- For LBM, if the wall length is less than 12 times the wall thickness, it is an SP, otherwise LP. - For RC, if the span length is less than or equal to 6 m, it is a SS, otherwise LS.

2.5.7 LBM: Wall Openings; RC: Pier Type

LBM

The size, number and distribution of openings in masonry walls largely affect the seismic behavior of an URM building, as it determines the shape and size of piers and spandrels and their relative stiffness and capacity. The presence of openings in an LBM wall reduces the in-plane capacity and stiffness and causes damage concentration in the areas around openings. The out-of-plane vulnerability also increases due to the presence of openings as the cracks initiating around the openings can easily trigger partial collapses (Abrams et al., 2017).

Openings in a wall cause the easier development and propagation of diagonal shear cracks and this effect is more pronounced when the openings are of different size and irregular distribution (Augenti and Parisi, 2010). An irregular distribution of openings often induces concentration of drift demands and damage in some particular regions of the wall which causes an increased seismic vulnerability, as observed in past earthquake damage surveys, for instance, conducted by Decanini et al. (2004) (2002 Molise earthquake); Kaplan et al. (2008) (2007 Cameli earthquake); D'Ayala and Paganoni (2011) (2009 L'Aquila earthquake) and experimental studies (e.g. Paquette and Bruneau, 2003; Bothara et al., 2010). To limit the seismic damage, openings are to be located at a specified minimum clear distance from the ends and top of the walls and if unavoidable, should be reinforced (Bothara et al., 2002).
In this study, the openings are considered to be either small or large opening. The opening is small if the combined width of the openings on a wall between two consecutive cross walls is less than 50% of the wall length and it is large when it is equal to or more than 50% of the wall length. This decision is based on the analysis of the opening characteristics in the school buildings from case study countries.

RC

In RC structures, the columns are equivalent to the masonry piers, and play a key role in the stiffness and capacity of the structure. For RC structures the minimum size of the column is recorded and is an indicator of the building behavior (building code usually limit columns and beams minimum dimensions (ASCE 7-16 & ACI 318-14) and indicates the capacity of the column in relation to the beams. Taking this into account, the Pier Type parameter allows to classify the structures in terms of its propensity to develop a weak floor collapse mechanism. Two classes are considered:

- Slender or weak column (SW): their inertia and cross section are smaller than the beam cross section. This can generate a weak or soft story and can trigger failure mechanism controlled by the columns instead of the beams
- Regular columns (RO): both column dimensions should be at least equal to the depth of the beam. In this case the frame is likely to comply with the Strong column-weak beam requirement. In this case it would be more probable a failure mechanism controlled by the beams in the upper stories which would present a more ductile type of failure.

Table 2-9 summaries the classification of openings for LBM and the column slenderness for RC structures.

Table 2-9. LBM: Wall Openings. RC: Pier Type

No.	Parameter	Attributes	Commentaries	
7	**LBM: Wall Openings** **RC: Pier type**	**For LBM:** **SO** - Small Openings **LO** - Large Openings **For RCF:** **SW** - Slender-Weak Column **RO** - Regular Column	- For RC frames, RO criteria meet when: *The column depth is at least the same as the beam *Three times the length divided by the depth of the column is less than 22 (ACI 318-14)	- For LBM, if the opening width in a wall between two consecutive cross walls is less than 0.35 and 0.25 times the wall length in single- and multi-story building, respectively, it is considered a SO; otherwise LO.

2.5.8 Foundation Type

The type of foundation influences the seismic performance of a building by controlling the settlements, cracking, deformations and overturning at the base of the main lateral load resisting systems. In fact, all the foundation structures have some flexibility. Depending on the foundation structure and the underlying soil properties, a foundation structure can be categorized as flexible or rigid compared to the flexibility of superstructure. A rigid type foundation usually prevents large foundation deformations as well as anticipated failures. On the other hand, flexible foundations contribute to the horizontal deformations of the building generating possibility of anticipated failures both at the foundation level and in upper structural elements.

Masonry walls usually have continuous stone masonry, brick masonry strip type foundation or reinforced concrete strip footings below the ground level. These foundation structures are usually thicker than the masonry walls. If these foundations are at least 1 m deep and the site soil is medium type or hard type, the foundation can be categorized as a rigid foundation type.

In the case of RC buildings, the foundation is usually built in reinforced concrete, but the behavior depends greatly on the soil type and on the presence of foundation beams. The most common foundation is isolated footings, which may be very rigid in hard soils but flexible in soft soils. On the other hand, a deep mat foundation can be considered rigid in hard or soft soil. Table 2-10 summaries the various types of foundations for both LBM and RC structures and classifies them in either flexible or rigid.

Table 2-10. Foundation Type

No.	Parameter	Attributes	Commentaries
8	**Foundation Type**	**FF** - Flexible Foundation **RF** - Rigid Foundation	The foundation type depends on two factors. 1. The foundation details: materials, structure and depth below the ground level. • The material can be RC, Brick Masonry or Stone Masonry or Dry-Stone Masonry • The structure type can be Isolated Footing, Combined Footing, Strip Footing, Mat Foundation etc. • Foundation depth can be Shallow, Medium or Deep. 2. The soil type in the site which can be soft, medium or hard type. The combination of these two factors decides the type of foundation. For example, a RC Mat foundation in a hard type soil is considered as a rigid type foundation.

2.5.9 Seismic Pounding Risk

Seismic pounding occurs when two adjacent building having different vibration characteristics collide with each other during earthquakes. Although this is not a significant issue in case of low-rise building structures, if the gap between the buildings is very small, it can cause damage to structural or even non-structural elements of a building due to hammering and eventually cause partial collapse. The minimum gap recommended by FEMA (FEMA 273, 1997) and other codes is at least 4% of the building height.

Table 2-11. Seismic Pounding Risk

No.	Parameter	Attributes	Commentaries
9	**Seismic Pounding Risk**	**PR** - Pounding Risk **NP** - No Pounding	Seismic gap between buildings at least 4% of the critical height. Critical height is the height of the shorter building where the expected collision occurs.

2.5.10 Effective Seismic Retrofitting

Effective seismic retrofitting is a process of strengthening a building structure, by which its seismic resistance is increased, thereby improving the seismic performance of the structure. Seismic strengthening can be mainly categorized into two types: strengthening of vertical load resisting system and strengthening of horizontal structures. The strengthening of vertical load resisting elements includes the different measures to increase the strength, ductility etc. of the vertical members such as walls or frames or improving the connections among the vertical load resisting elements. Some examples of these interventions are jacketing of LBM walls or RC columns, installation of bracings etc. On the other hand, the strengthening of horizontal structures includes the increase in in-plane stiffness or floors/roof as well as the improvement of connection of these with the vertical load resisting system. These retrofitting interventions can improve the seismic performance of poorly designed school buildings in the future earthquakes. For example, several retrofitted LBM school buildings in Nepal survived without any

damage during the 2015 Nepal earthquake. In the case study countries, the retrofitting interventions has been applied on very few school buildings.
As the retrofitting work is usually covered (with non-structural elements such as plaster or any other) after its application, it is often necessary to talk to the school administrators to know more about the seismic retrofitting history on the school building.

It is important to note that for each school building, the main structural system and all other taxonomy parameters will be classified for the retrofitted structures. The importance of knowing about previous retrofitting works is to recognize that this particular building is not of the same quality of an equivalent new one.

The attributes for this parameter are either original structure or retrofitted structure as shown in Table 3-14.

Table 2-12. Effective Seismic Retrofitting

No.	Parameter	Attributes	Commentaries
10	**Effective Seismic Retrofitting**	**OS -** Original Structure **RS -** Retrofitted Structure	-If a building has been retrofitted effectively so that the seismic behavior has been improved considerably with respect to its original situation, it is a retrofitted structure (RS). Minor non-structural improvements and/or maintenance don't make it a retrofitted structure.

2.5.11 Structural Health Condition

Structural health condition describes a building's current physical condition with respect to the material deterioration and existing damages in the structure. Masonry materials such as brick and mortar can deteriorate over time. Similarly, steel reinforcement bars in confined, reinforced masonry or reinforced concrete may get corroded or exposed over time due to the disintegration of the concrete cover. Existing damages (e.g. building out of plumb, delaminated walls, corner separation, cracks in the walls/columns etc.) contribute more to the seismic vulnerability of a building. Based on these analyses, buildings can be categorized in terms of the health condition as good or poor.

Examples of factors that determine the structural health condition for LBM buildings are:

- Deteriorated materials (units and mortar)
- Deteriorated connections among structural and non-structural elements (e.g. between walls and floors/roofs, between roof structure and roof covering tiles/sheets)
- Exposed reinforcement bars or corrosion in the reinforcement bars in reinforced or confined masonry
- Existing structural damages (cracks in the walls, corner separation, tilted building/walls etc.)

Examples of factors that determine the structural health condition for RC buildings are:

- Disintegration/deterioration of concrete
- Exposed rebars
- Corroded of rebars

- Existing cracks

Table 2-13. Building health condition

No.	Parameter	Attributes	Commentaries
11	**Structural Health Condition**	**PC** - Poor Condition **GC** - Good Condition	- Engineering judgement required. It refers to conditions that may affect the general building behavior. - See FEMA P-68 for further information

2.5.12 Non-Structural Components

In school buildings, several forms of non-structural components such as gables, heavy roof covering (e.g. tiles), parapets, in-class furniture and others may impose special vulnerability conditions during earthquakes. For example, if not secured properly, heavy masonry gables are one of the most vulnerable non-structural components as they act like cantilever walls and are subjected to higher levels of acceleration (Bothara et al., 2010). The vulnerability can be reduced using light gable materials (such as CGI sheet) or tying masonry gables using RC tie beams. Also, the proper tying of the roof tiles to the purlins greatly reduces the hazard that is inherent to the unsecured roof tiles during a seismic event (Bothara et al. 2010). Unsecured furniture, blackboards, covers, divisions, equipment, pipes, installations or windows can topple down during earthquakes and thus can be hazardous to the building occupants.
The presence, location, self-weight and connection details of non-structural elements maybe assessed and rated as vulnerable or non-vulnerable.

Table 2-14. Vulnerable non-structural elements

No.	Parameter	Attributes	Commentaries
12	**Non-Structural Components**	**VN** - Vulnerable Non-Structural Components **NN** - Non-Vulnerable Non-Structural Components	-It refers to components that can produce economic losses or human casualties such as: parapets, ceilings, tiles, pipes infill, etc. This parameter is rather qualitative, and the selection of associated attributes depends on the assessment of all the non-structural components with respect to the location, self-weight, connection to the main structural elements etc.

2.6 Building Classification System (GLoSI Taxonomy)

Based on the taxonomy parameters discussed in section 2.5, a building taxonomy is developed for two major construction types i.e. LBM and RC construction types of school buildings (GLoSI Taxonomy form provided in attached excel sheet, refer to **Annex D**). According to the proposed GLoSI taxonomy, an old low-rise adobe school building with no seismic enhancement features and flexible diaphragm etc. is identified with a string containing the corresponding attributes of the taxonomy parameters, ultimately resulting in a taxonomy string given as: A/LR(1)/PD/FD/…… The length of the string depends on the extent of information on the building characteristics; the more the information, the longer the string and vice-versa. Further, when limited information is available, any element in the string can be omitted or truncated depending on the availability of the information or priorities given to different taxonomy parameters.

2.7 GLoSI Building Type Catalog

As mentioned before, the building types are the construction types which are defined by the three primary parameters. Thus, for any school building, these three primary parameters should be known to identify its building type. The building type catalogue (**Annex E)** documents the common building types of load bearing masonry (LBM) and reinforced concrete (RC) school buildings found mainly in the four case studies countries viz. Nepal, El Salvador, Peru and Kyrgyz Republic. However, it is open and flexible to include more building types that are present in other countries. For each catalogue of different construction type, the building type is identified by the first three primary parameters (main structural system, height range and the seismic design level). Countries of occurrence of the building type are given. Then, the construction characteristics of the building type with respect to the primary parameters are briefly discussed with the similarities/differences at in-country or regional level. Finally, representative photographs of the building type are presented. Readers are referred to **Annex E** for the details of the building type catalogue.

2.8 Application Examples of the GLoSI Taxonomy

In this section, examples of the application of the GLoSI taxonomy is presented for an LBM and an RC school building, respectively.

2.8.1 <u>Application Example for LBM School Building</u>

An LBM school building from Nepal (shown in Figure 2-3) is taken as an example to show the application of the proposed GLoSI taxonomy. This load bearing masonry building is constructed in brick in mud mortar masonry, hence the main structural system is **'Unconfined/Unreinforced Masonry with Rectangular Block in Mud Mortar Walls i.e. UCM-URM4'**. This is a single-storied building, thus the height range is **'Low-Rise (Single Storied) i.e. LR(1)'**. This is an old building constructed by the communities without following seismic design guidelines and doesn't include any seismic enhancement measures such as seismic bands, corner ties etc. however the walls are thick (one and a half brick thick) and are constructed in English-bond masonry with proper interlocking with the cross walls. Therefore, the seismic design level is **'Low i.e. LD'**. The roof structure is a timber structure with low in-plane stiffness and it is poorly connected to the load bearing wall system as the timber elements simply rest on top of the walls. Thus, the roof diaphragm is **'Flexible Diaphragm i.e. FD'**. This building has a rectangular regular plan shape and there are no horizontal or vertical irregularities, making it a regular structure. Hence the attribute for structural irregularities is **'No Irregularity i.e. NI'**. The unrestrained wall panels between two consecutive cross walls are longer than 12 times the wall thickness, hence the wall panel is a **'Long Wall Panel i.e. LP'** types. Similarly, the door and window openings are large and in a wall between two consecutive cross walls, the combined width of openings exceeds 50% of the wall length. Hence the wall opening is **'Large Opening i.e. LO'** type. These brick in mud masonry wall in Nepal are usually laid on stone in mud masonry work which is a strip foundation with depth about 1.5 m. The site is located in a medium soil. The foundation is thus assumed to be a **'Rigid (RF)'** type. This is an isolated building with sufficient clear distance from other buildings and hence there is **'No Pounding (NP)'** risk. There is no effective retrofitting intervention later after the original construction, hence it is an **'Original Structure i.e. OS'**. The mud mortar used in the construction have deteriorated over time and there are existing seismic damages (cracks wider than 5 mm in the masonry walls, partial collapse of gables). Thus, the structural health condition is **'Poor i.e. PC'**. The heavy masonry gables are standing freely and are very vulnerable during earthquakes. The connections of roof coverings to the roof structure is poor. Hence the non-structural components are **'Vulnerable i.e. VN'**. All of these attributes of different taxonomy parameters are summarized and listed in Table 2-15

Figure 2-3 Front (left) and side (right) view of a brick in mud mortar school building in Nepal (Copyright: The World Bank)

Table 2-15. Attribute identification for different taxonomy parameters for the selected school building.

Taxonomy Parameters		Identified Attributes
I.	Main Structural System	UCM-URM4
II.	Height Range (No. of Stories)	LR(1)
III.	Seismic Design Level	LD
IV.	Diaphragm Type	FD
V.	Structural Irregularity	NI
VI.	Wall Panel Length	LP
VII.	Wall Opening	LO
VIII.	Foundation Type	RF
IX.	Seismic Pounding Risk	NP
X.	Effective Seismic Retrofitting	OS
XI.	Structural Health Condition	PC
XII.	Non-Structural Components	VN

Thus, the identified attributes (referring to Table 2-14) collectively result in a unique taxonomy string for the building shown in Table 2-16. The first three attributes in the string are given in bold for that they define the building type of a given school building.

Table 2-16. Building type identification and the GLoSI taxonomy string for the selected school building.

Building Type	GLoSI Taxonomy String
UCM-URM4 Low-Rise with Poor Seismic Design Level	**UCM-URM4/LR(1)/PD**/FD/NI/LP/LO/RF/NP/OS/PC/VN

2.8.2 Application Example for an RC School Building

An RC school building (shown in Figure 2-4) is taken as an example to show the application of the proposed GLoSI taxonomy. This reinforced concrete building is a framed structure with masonry infills, hence the main structural system is **'RC moment resisting frame with masonry infills i.e. RC2'**. This is a two story building, thus the height range is **'Mid-Rise (Two Stories) i.e. MR(2)'**. This seems to be an engineered design but the dimensions of the structural elements (columns and beams) indicate it was designed for a low seismic hazard zone. Therefore, the seismic design level is **'Low i.e. LD'**. The floor and roof appear to be made with a system of beams of reinforced concrete. Thus, the diaphragm is **'Rigid Diaphragm i.e. RD'**. This building has a rectangular regular plan shape and there are no horizontal or vertical irregularities, making it a regular structure. Hence the attribute for structural irregularities is **'No Irregularity i.e. NI'**. There is no exact measure of the span length, but it can be defined from the photo which is less than 6 m, hence the span length is classify as **'Short Span i.e. SP'**. Similarly, the columns are bigger than the beams and no weak column mechanism should appear, thus the Pier Type is **'Regular Column i.e. RO'** type. These structures are usually built with a concrete foundation, composed of isolated footings connected by beams, this type of system is commonly accepted in modeling as rigid. In this case the foundation type is classified as **'Rigid Foundation (RF)'**. This is an isolated building with sufficient clear distance from other buildings and hence there is **'No Pounding (NP)'** risk. There is no effective retrofitting intervention later after the original construction, hence it is an **'Original Structure i.e. OS'**. The building seems to be in a fair condition, the concrete and masonry elements don't present any proof of bad condition. Thus, the structural health condition is **'Fair i.e. FC'**. The gables don't have any concrete element in top, this make them a vulnerable component. Hence the non-structural components are **'Vulnerable i.e. VN'**. All of these attributes of different taxonomy parameters are summarized and listed in Table 2-17.

Figure 2-4 Front view of a reinforced concrete school building (Copyright: The World Bank)

Table 2-17. Attribute identification for different seismic taxonomy parameters for the selected RC school building.

Taxonomy Parameters		Identified Attributes
I.	Main Structural System	RC2
II.	Height Range (No. of Stories)	MR(2)
III.	Seismic Design Level	LD

IV.	Diaphragm Type	RD
V.	Structural Irregularity	NI
VI.	Span Length	SP
VII.	Pier Type	RO
VIII.	Foundation Type	RF
IX.	Seismic Pounding Risk	NP
X.	Effective Seismic Retrofitting	OS
XI.	Structural Health Condition	FC
XII.	Non-Structural Components	VN

Thus, the identified attributes (referring to Table 2-17) collectively result in a unique taxonomy string for the building shown in Table 2-18. The first three attributes in the string are given in bold for that they define the building type of a given school building.

Table 2-18. Building type identification and the GLoSI taxonomy string for the selected RC school building.

Building Type	GLoSI Taxonomy String
RC2 Mid-Rise with Low Seismic Design Level	**RC2/MR(2)/LD**/RD/NI/SP/RO/RF/NP/OS/FC/NN

3. INDEX BUILDINGS

3.1 Introduction

As part of the GLoSI initiative, relevant information is evaluated and reviewed in order to establish the criteria to define and characterize a group of building types that adequately represent the complete portfolio of school buildings worldwide in terms of their expected seismic behavior. Considering the high number of school buildings in a country, usually about 1000 schools per million inhabitants or more, it is mandatory to identify in each country or region of interest the most representative building typologies in order to simplify the risk assessment process and scale up the implementation of the mitigation strategies.

An Index Building is a characteristic model of a building type, whose seismic behavior represents a group of buildings, with uniquely defined geometry, loads, materials, characteristics and dynamic behavior (D'Ayala, 2015). The catalogue of index buildings, real or fictitious, shall be representative of the school building portfolio at global level. Each index building will be uniquely defined with a set of parameters in order to have a clear reference of its representativeness for applications in any regions and/or countries (Yamin, 2013). Those parameters will identify the main structural and non-structural characteristics and the general condition of the building that, at the end, will define the seismic behavior of the represented buildings. Each index building will be associated to a particular collection of strings selected from the GLoSI taxonomy parameters (see Chapter 2). In addition, a collection of key parameters is defined, and ranges of values are assigned on order to have a clear understanding of the applicability of the index buildings vulnerability to individual buildings which are part of the school building portfolio of any country.

For each index building, a detail assessment of the physical fragility and vulnerability will be conducted. As a result, specific fragility and vulnerability functions will be proposed for risk assessment purposes. Considering that each index building can eventually represent a vast group of buildings, the uncertainty of the assessment will also be reported. In addition, sensitivity analysis will be performed in order to give some quantitative indicators about the expected variations of the F/V assessment, indicating the relative importance of each parameter.

It is important to note that the collection of index buildings that are proposed for a particular country or region, are a result of the taxonomy parameter screening, i.e. they are not pre-codified, even though for the definition and selection of relevant parameters and their attributes, prior knowledge and literature is carefully examined. The selected index buildings refer mainly to the countries where specific studies have been conducted and they are proposed with intend to be used globally. It is expected that each new country vulnerability assessment would probably generate additional index buildings, populating in this way the GLoSI database.
This chapter has the following main objectives:

a) Outline the general criteria for the selection of representative index buildings for (load Bearing Masonry) LBM and (Reinforced Concrete) RC framed school buildings.
b) Select the main vulnerability parameters that control the seismic performance and vulnerability of the LBM and RC framed school index buildings.
c) Identify, define, characterize and list the index buildings per typology according to the identified taxonomy parameters for subsequent F/V assessment.
d) Define all the information required in order to develop the F/V assessment for each one of the defined index buildings.

3.2 Criteria for defining index buildings

3.2.1 Parameters for the definition of Index Buildings

The vulnerability parameters required to clearly identify a unique expected seismic behavior of an Index Building can be divided in three main groups as follows:

a) Primary parameters
b) Secondary parameters
c) Intrinsic parameters

The main parameters are the parameters that control the overall performance of the building and include the main structural material and system, the height range and the seismic design level. The secondary parameters are a group of characteristics that will have an important role in modifying the usual expected behavior of a building that is already classified according to the three main parameters. And finally, the intrinsic parameters include the definition of the building geometry and details and the material properties which are required to adequately characterize the expected seismic building behavior.

Figure 3-1 presents the organization of all vulnerability parameters in the previous three groups of parameters. The Index Buildings are defined primarily by the most probable combinations of the Main Parameters. The secondary parameters are selected as representative attributes of the different analyzed catalogues and usually the most common value or a range of values are defined for each particular Index Building.

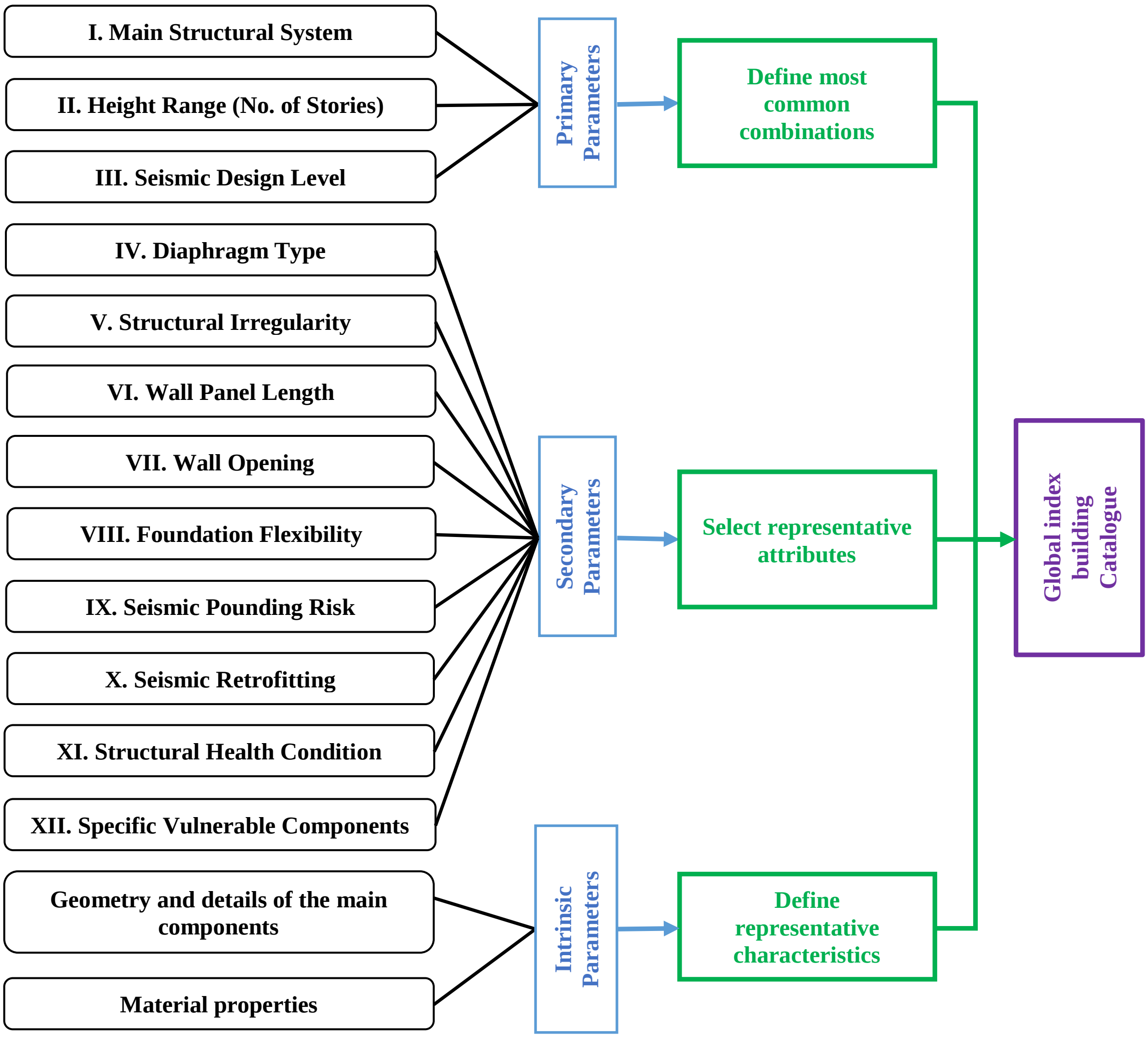


Figure 3-1. Taxonomy parameters

3.2.2 Primary parameters

The main parameters are the attributes that control the overall performance of the building. They include the first 3 categories of the taxonomy as shown in Figure 3-2. The attributes of each one of the parameters are indicated in the figure and are explained in detail in the Taxonomy report (Chapter 2).

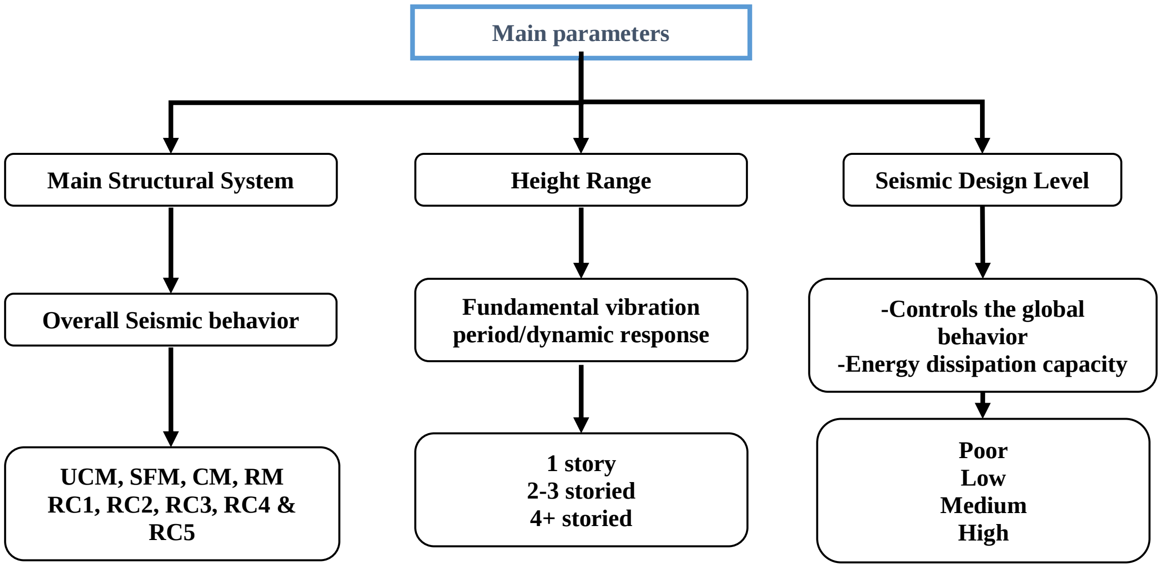


Figure 3-2. Main parameters

3.2.3 Secondary parameters

The secondary parameters describe a more specific set of buildings characteristics that may modify or affect the expected seismic behavior. The attributes for each one of the parameters are explained in detail in Chapter 2. The F/V functions for the GLoSI library are computed with the most common or less vulnerable condition for each parameter as shown in Figure 3-3. In some particular cases, specific values are selected for these attributes in order to generate particular F/V functions according to local conditions in any one country or region.

Secondary Parameters
Conditions assumed for the initial F/V assessment

Diaphragm Type	FD-RD
Structural Irregularity	HV-HI-VI-NO
Wall Panel Length/Span Length	SP-LP SS-LS
Wall Openings/Pier Type	SO-LO SW-RO
Foundation Type	FF - RF
Seismic Pounding Risk	PR-NP
Effective Seismic Retrofitting	OS - RS
Structural Health Condition	PC-GC
Non-Structural Components	VC - NV

Figure 3-3. Secondary parameters

3.2.4 Intrinsic characteristics

Intrinsic characteristics correspond to parameters that allow a clear and final definition of the Index Building and include the geometry and the material quality.

3.2.4.1 Geometry and details of the building and components

For LBM Index Buildings, the following geometrical characteristics and dimensions are necessary to define the numerical model.

- Overall architectural plan with dimension
- Floor/roof system dimension (e.g. RC slab dimensions) and the measurement of connection details.
- Inter-story height
- Foundation dimensions and characteristics
- Wall thickness
- Brick size and mortar thickness
- Reinforcement details (size, number, spacing etc.) in reinforcement masonry walls.

- Confinement tie-beam/tie-column arrangement, dimension and reinforcement details in confined masonry walls.
- Dimension of seismic enhancement features such as corner quoin, through stone, lintel beams, and seismic band beams etc.
- Dimension of the non-structural elements and their connections.

Similarly, for RC Index Buildings, following geometrical characteristics geometrical dimensions are necessary to define the numerical model.

- Building plan area
- Building total area
- Number of stories
- Story height
- Number of spans in X direction
- Typical span length in X direction
- Number of spans in Y direction
- Typical span length in Y direction
- Foundation system
- Typical column dimensions
- Typical beam dimensions
- Typical shear wall dimensions
- Typical bracing member section

3.2.4.2 Materials

For LBM Index Buildings, following mechanical properties of materials are necessary to define the numerical model.

- Mechanical properties (modulus of elasticity, compressive strength, tensile strength, coefficient of friction, shear strength etc.) of the masonry material.
- Mechanical properties of the steel reinforcement in reinforced masonry walls.
- Mechanical properties of the reinforced concrete material in confined masonry walls.
- Mechanical properties of the seismic enhancement measures such as lintel beams, seismic band beams.

For RC Index Buildings, the following material properties of materials are necessary to define the numerical model.

- Mechanical properties of the concrete in main structural elements.
- Mechanical properties of the steel reinforcement.
- Expected cyclic degradation characteristics (strength and stiffness).
- Mechanical properties of partitions that could interact with the structure.
- Mechanical properties of steel structural members.

3.3 GLoSI Index Building Catalog

3.3.1 Selection of Index Buildings

A collection of Index Buildings is defined using the available information for several countries. Only the most probable combinations are included in the selected group of Index Buildings. In addition, some combinations may have similar performances or overlap with others and therefore only the more representative ones are selected. Finally, in some cases the combinations of parameters generate an Index Building which classifies as highly vulnerable and therefore no analytical solution is worth it. Some of the Index Buildings are not included herein considering that their presence is comparatively low. Further IBs shall be included in the database as new countries are included in the GPSS program.

3.3.2 List of Index Buildings

Table 3-1 present the selected index buildings for LBM school construction types. 14 different index buildings pertaining to different LBM building types are selected.

Table 3-1 List of Index Buildings for LBM School buildings for the GLoSI library.

S.N.	Building Type	Number of Index Buildings*	Index Building
LBM-1	**A/LR/LD**	1	**A/LR(1)/LD**/FD/NI/LP/LO/RF/NP/OS/PC/VN
LBM-2	**UCM-URM1/LR(1)/LD**	1	**UCM-URM1/LR(1)/LD**/FD/NI/SP/LO/RF/NP/OS/GC/VN
LBM-3	**UCM-URM2/LR(1)/PD**	1	**UCM-URM2/LR(1)/PD**/FD/NI/SP/SO/RF/NP/OS/PC/VN
LBM-4	**UCM-URM3/LR(1)/LD**	1	**UCM-URM3/LR(1)/LD**/FD/NI/SP/LO/RF/NP/OS/PC/VN
LBM-5	**UCM-URM4/LR(1)/LD**	1	**UCM-URM4/LR(1)/LD**/FD/NI/LP/SO/RF/NP/OS/PC/VN
LBM-6	**UCM-URM4/MR(2)/LD**	1	**UCM-URM4/MR(2)/LD**/RD/NI/LP/SO/RF/NP/OS/PC/NN
LBM-7	**UCM-URM5/LR(1)/PD**	1	**UCM-URM5/LR(1)/PD**/FD/NI/SP/LO/RF/NP/OS/GC/VN
LBM-8	**UCM-URM6/LR(1)/MD**	1	**UCM-URM6/LR(1)/MD**/FD/NI/SP/SO/RF/NP/OS/GC/VN
LBM-9	**UCM-URM7/LR(1)/LD**	2	**UCM-URM7/LR(1)/LD**/FD/NI/LP/LO/RF/NP/OS/PC/VN
LBM-10	**UCM-URM7/LR(1)/LD**		**UCM-URM7/LR(1)/LD**/RD/HI /LP/LO/RF/NP/OS/PC/NN
LBM-11	**UCM-URM7/MR(2)/LD**	1	**UCM-URM7/MR(2)/LD**/RD/NI/LP/LO/RF/NP/OS/PC/NN
LBM-12	**CM/LR(1)/HD**	1	**CM/LR(1)/HD/FD**/NI/SP/LO/RF/NP/OS/GC/NN
LBM-13	**RM/LR(1)/HD**	1	**RM/LR(1)/HD/FD**/NI/LP/LO/RF/NP/OS/GC/NN
LBM-14	**SFM4/LR(1)/LD**	1	**SFM4/LR(1)/LD**/FD/NI/LP/LO/RF/NP/OS/PC/VN

Similarly, Table 3-2 present the selected index buildings for RC school construction types. 23 different index buildings pertaining to different RC building types are selected.

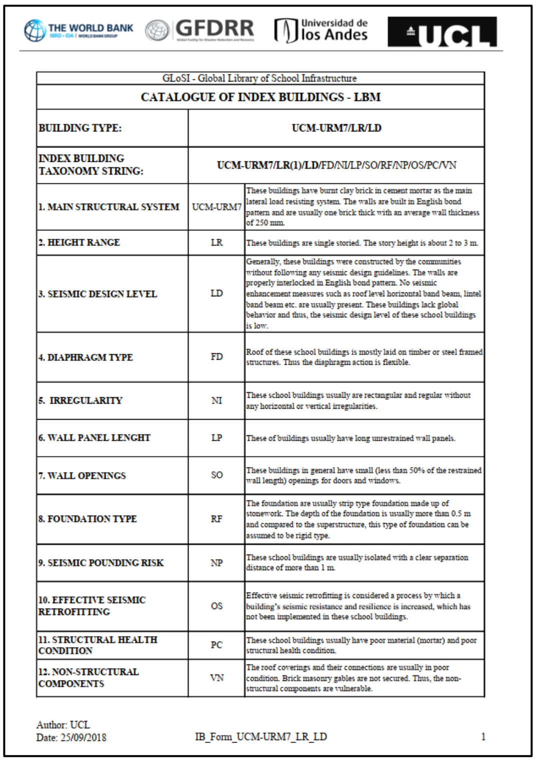
Table 3-2 List of Index Buildings for RC School buildings for the GLoSI library.

S.N.	Building Type	Number of Index Buildings*	Index Building
RC-1	RC1/MR(2)/PD	1	RC1/MR(2)/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-2	RC1/MR(2)/LD	1	RC1/MR(2)/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-3	RC1/MR(2)/HD	1	RC1/MR(2)/HD/RD/NI/LS/RO/RF/NP/OS/GC/NN
RC-4	RC1/HR(5)/PD	1	RC1/HR(5)/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-5	RC1/HR(5)/LD	1	RC1/HR(5)/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-6	RC1/HR(5)/HD	1	RC1/HR(5)/HD/RD/NI/LS/RO/RF/NP/OS/GC/NN
RC-7	RC2/LR(1)/LD	1	RC2/LR(1)/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-8	RC2/MR(2)/PD	1	RC2/MR(2)/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-9	RC2/MR(2)/LD	1	RC2/MR(2)/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-10	RC2/MR(2)/HD	1	RC2/MR(2)/HD/RD/NI/LS/RO/RF/NP/OS/GC/NN
RC-11	RC2/HR(5)/PD	1	RC2/HR(5)/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-12	RC2/HR(5)/LD	1	RC2/HR(5)/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-13	RC2/HR(5)/HD	1	RC2/HR(5)/HD/RD/NI/LS/RO/RF/NP/OS/GC/NN
RC-14	RC3/MR(2)/PD	1	RC3/MR(2)/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-15	RC3/MR(2)/LD	1	RC3/MR(2)/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-16	RC3/HR(5)/PD	1	RC3/HR(5)/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-17	RC3/HR(5)/LD	1	RC3/HR(5)/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-18	RC4/MR(2)/LD	1	RC4/MR(2)/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-19	RC4/MR(2)/HD	1	RC4/MR(2)/HD/RD/NI/LS/RO/RF/NP/OS/GC/NN
RC-20	RC4/HR(5)/LD	1	RC4/HR(5)/LD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-21	RC4/HR(5)/HD	1	RC4/HR(5)/HD/RD/NI/LS/RO/RF/NP/OS/GC/NN
RC-22	RC5/LR(1)/PD	1	RC5/LR(1)/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN
RC-23	RC5/MR(2)/PD	1	RC5/MR(2)/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN

3.3.3 Index Building Catalog

A form has been designed in order to summarize the relevant information of each one of the Index Building identified for the GLoSI library. Figure 3-4 presents examples of the catalogue of an LBM and an RC IB with the specific choice of attributes for each taxonomy parameter. **Annex F** presents the forms for all the LBM (14 different) and the RC (23 different) IBs.

LBM Index Building



GLoSI - Global Library of School Infrastructure		
CATALOGUE OF INDEX BUILDINGS - LBM		
BUILDING TYPE:	**UCM-URM7/LR/LD**	
INDEX BUILDING TAXONOMY STRING:	**UCM-URM7/LR(1)/LD/FD/NI/LP/SO/RF/NP/OS/PC/VN**	
1. MAIN STRUCTURAL SYSTEM	UCM-URM7	These buildings have burnt clay brick in cement mortar as the main lateral load resisting system. The walls are built in English bond pattern and are usually one brick thick with an average wall thickness of 250 mm.
2. HEIGHT RANGE	LR	These buildings are single storied. The story height is about 2 to 3 m.
3. SEISMIC DESIGN LEVEL	LD	Generally, these buildings were constructed by the communities without following any seismic design guidelines. The walls are properly interlocked in English bond pattern. No seismic enhancement measures such as roof level horizontal band beam, lintel band beam etc. are usually present. These buildings lack global behavior and thus, the seismic design level of these school buildings is low.
4. DIAPHRAGM TYPE	FD	Roof of these school buildings is mostly laid on timber or steel framed structures. Thus the diaphragm action is flexible.
5. IRREGULARITY	NI	These school buildings usually are rectangular and regular without any horizontal or vertical irregularities.
6. WALL PANEL LENGHT	LP	These of buildings usually have long unrestrained wall panels.
7. WALL OPENINGS	SO	These buildings in general have small (less than 50% of the restrained wall length) openings for doors and windows.
8. FOUNDATION TYPE	RF	The foundation are usually strip type foundation made up of stonework. The depth of the foundation is usually more than 0.5 m and compared to the superstructure, this type of foundation can be assumed to be rigid type.
9. SEISMIC POUNDING RISK	NP	These school buildings are usually isolated with a clear separation distance of more than 1 m.
10. EFFECTIVE SEISMIC RETROFITTING	OS	Effective seismic retrofitting is considered a process by which a building's seismic resistance and resilience is increased, which has not been implemented in these school buildings.
11. STRUCTURAL HEALTH CONDITION	PC	These school buildings usually have poor material (mortar) and poor structural health condition.
12. NON-STRUCTURAL COMPONENTS	VN	The roof coverings and their connections are usually in poor condition. Brick masonry gables are not secured. Thus, the non-structural components are vulnerable.

Author: UCL
Date: 25/09/2018 IB_Form_UCM-URM7_LR_LD 1

COUNTRIES WHERE THIS SCHOOL BUILDING TYPE EXISTS:

Nepal, El Salvador, Peru

REPRESENTATIVE PHOTOGRAPHS

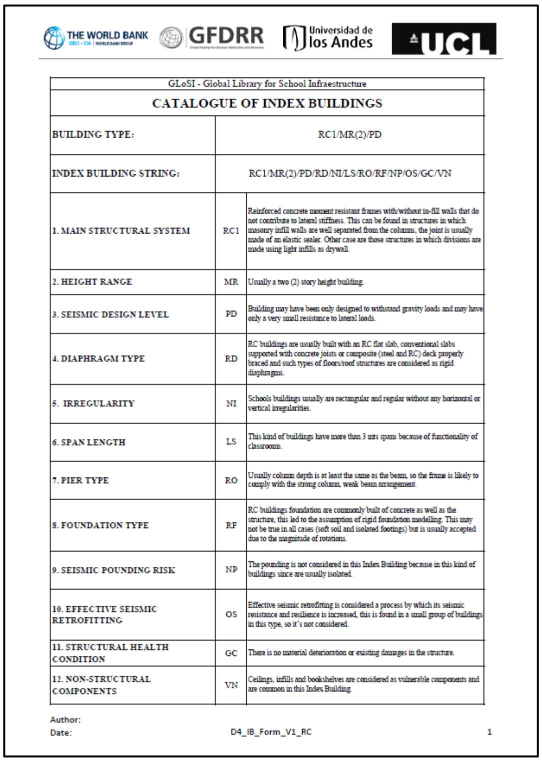
(a)

(b)

Figure 1. Photographs representative of an index building of the UCM-URM7/LR/LD school building type: (a) Longitudinal view, (b) Inside transverse view showing flexible steel roof structure. (Photos from Nepal, Copyright: The World Bank)

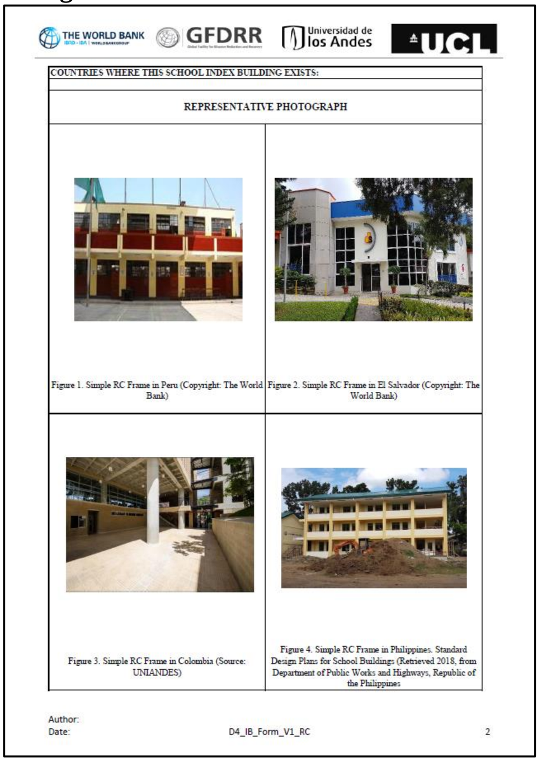
Author: UCL
Date: 25/09/2018 IB_Form_UCM-URM7_LR_LD 2

RC Index Building



GLoSI - Global Library for School Infraestructure		
CATALOGUE OF INDEX BUILDINGS		
BUILDING TYPE:	RC1/MR(2)/PD	
INDEX BUILDING STRING:	RC1/MR(2)/PD/RD/NI/LS/RO/RF/NP/OS/GC/VN	
1. MAIN STRUCTURAL SYSTEM	RC1	Reinforced concrete moment resistant frames with/without in-fill walls that do not contribute to lateral stiffness. This can be found in structures in which masonry infill walls are well separated from the columns, the joint is usually made of an elastic sealer. Other case are those structures in which divisions are made using light infills as drywall.
2. HEIGHT RANGE	MR	Usually a two (2) story height building.
3. SEISMIC DESIGN LEVEL	PD	Building may have been only designed to withstand gravity loads and may have only a very small resistance to lateral loads.
4. DIAPHRAGM TYPE	RD	RC buildings are usually built with an RC flat slab, conventional slabs supported with concrete joists or composite (steel and RC) deck properly braced and such types of floors/roof structures are considered as rigid diaphragms.
5. IRREGULARITY	NI	Schools buildings usually are rectangular and regular without any horizontal or vertical irregularities.
6. SPAN LENGTH	LS	This kind of buildings have more than 3 mts spans because of functionality of classrooms.
7. PIER TYPE	RO	Usually column depth is at least the same as the beam, so the frame is likely to comply with the strong column, weak beam arrangement.
8. FOUNDATION TYPE	RF	RC buildings foundation are commonly built of concrete as well as the structure, this led to the assumption of rigid foundation modelling. This may not be true in all cases (soft soil and isolated footings) but is usually accepted due to the magnitude of rotations.
9. SEISMIC POUNDING RISK	NP	The pounding is not considered in this Index Building because in this kind of buildings since are usually isolated.
10. EFFECTIVE SEISMIC RETROFITTING	OS	Effective seismic retrofitting is considered a process by which its seismic resistance and resilience is increased, this is found in a small group of buildings in this type, so it's not considered.
11. STRUCTURAL HEALTH CONDITION	GC	There is no material deterioration or existing damages in the structure.
12. NON-STRUCTURAL COMPONENTS	VN	Ceilings, infills and bookshelves are considered as vulnerable components and are common in this Index Building.

Author:
Date: D4_IB_Form_V1_RC 1



COUNTRIES WHERE THIS SCHOOL INDEX BUILDING EXISTS:

REPRESENTATIVE PHOTOGRAPH

Figure 1. Simple RC Frame in Peru (Copyright: The World Bank)

Figure 2. Simple RC Frame in El Salvador (Copyright: The World Bank)

Figure 3. Simple RC Frame in Colombia (Source: UNIANDES)

Figure 4. Simple RC Frame in Philippines. Standard Design Plans for School Buildings (Retrieved 2018, from Department of Public Works and Highways, Republic of the Philippines

Author:
Date: D4_IB_Form_V1_RC 2

Figure 3-4. Typical IB Catalogue for an LBM and an RC building.

4. FRAGILITY AND VULNERABILITY ASSESSMENT

4.1 Introduction and objectives

As part of the GLoSI initiative, a methodological approach is defined to derive both the seismic fragility and vulnerability (F/V) of selected school index buildings (IBs). Considering that each of those IBs represents a typology that can be found in several countries, a reliable analytical assessment of its expected seismic performance is an important contribution towards a robust seismic risk assessment process in any particular country or region worldwide.

Existing globally used risk assessment platforms (HAZUS (FEMA, 2012), CAPRA (ERN, 2009), OpenQuake (GEM, 2018), RISK-UE (Moroux et al, 2006)) typically provide, for each building typology, a quantitative probabilistic relationship between given seismic intensity and expected damage expressed in terms of either a fragility or a vulnerability function.

Fragility functions establish the probability of reaching or exceeding a particular damage state given a hazard intensity parameter. Damage states are usually defined in terms of global or local parameters, which identify the loss of physical integrity and structural capacity of the building. In analytical fragility assessment the damage states are defined with respect to damage thresholds, i.e. specific values of an Engineering Demand Parameter (EDP), such as roof or inter-story drift, which characterize the onset of a particular damage state (D'Ayala et al, 2015).

Vulnerability functions correlate the Mean Damage Ratio (MDR) and its variance with a hazard intensity parameter. The MDR is usually expressed in economic terms, as the ratio of the expected total repair cost to the total replacement cost of the building (Yamin, 2017). Within the GLoSI library, the total replacement cost of the building has been defined as the actual reconstruction cost of the building according to local price conditions in the country or zone under analysis.

The hazard intensity parameter used for the GLoSI library corresponds to PGA (g) for LBM structures and *Sa(T)* for RC structures. Figure 4-1 presents the general conception and representation of the fragility and vulnerability functions used in a risk assessment process (Yamin, 2017).

The main sources of uncertainty are considered in the vulnerability assessment. Both aleatory and epistemic uncertainties are estimated. Aleatory uncertainties are associated with the seismic input, the soil response, and the frequency content of seismic records used, and the variability in the materials and design of the building stock. The epistemic uncertainty is associated with lack of knowledge of some aspects of the problem and limitation of the numerical modelling methodology, the estimation of the damage states, the repair cost estimation and other analytical parameters used in the assessment (Yamin, 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).

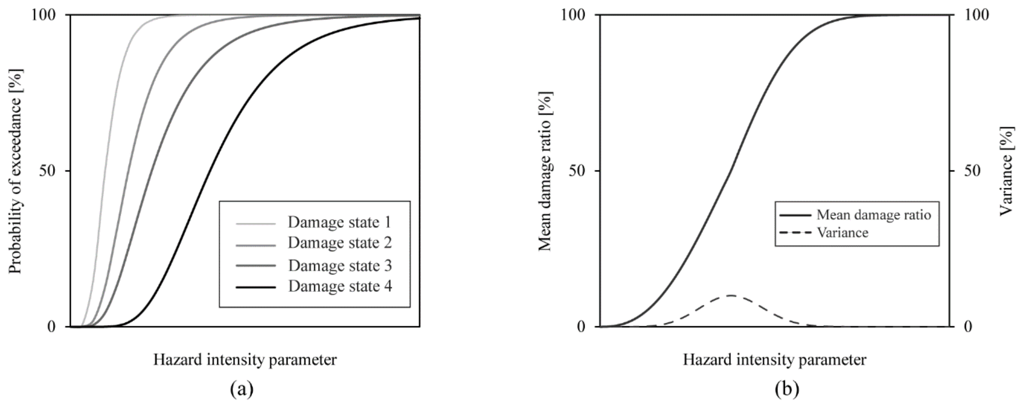


Figure 4-1. Typical representation of (a) fragility and (b) vulnerability (Yamin, 2017)

Finally, sensitivity analyses are performed in order to quantify the expected variations in the resulting F/V functions for a given IB, when particular critical taxonomy parameters take different values. The collection of F/V functions for all IB considered, its uncertainty and possible sensible variations of them with critical selected parameters, form one of the main contributions of the GLoSI library to future risk assessment processes.

The main objective of the present chapter is to explain the methodological approach used in the GLoSI to estimate both fragility and vulnerability functions for each IB and present illustrative examples demonstrating its applicability. Using this methodological approach, the GLoSI is populated with F/V functions for the group of IBs considered.

The specific objectives are the following:

e) Define a reliable methodology for the F/V assessment of RC and LBM school buildings.
f) Establish a simple procedure for seismic performance assessment of representative IBs which generates reliable EDP with respect to a range of seismic IM.
g) Select the criteria to build F/V functions using representative EDP and appropriate components and global damage models.
h) Assemble a database of seismic F/V functions for representative IBs of different school building types in the Global Library of School Infrastructure (GLoSI).

The proposed methodology shall be easily adopted by the structural engineering community to generate F/V functions for particular local conditions in developing countries worldwide.

4.2 Methodological Approach for Fragility/Vulnerability Assessment

A great diversity of methodologies has been proposed recently in the literature for the seismic F/V assessment of representative buildings (D´Ayala et al, 2015, Yamin, 2017). Approaches may consider empirical or expert opinion-based or analytical or hybrid methods for the derivation of F/V functions. For the GLoSI library, the analytical vulnerability approach is adopted, as it allows for an un-biased and consistent assessment applicable worldwide, independently of historic seismic damage data and local expertise on specific typological building performance (Rossetto et al., 2014). The analytical methods allow fragility and vulnerability functions to be easily updated, complemented and modified as more refined data on exposure or refined analytical approaches become available. Notwithstanding its generic essential quality, the analytical approach also allows taking into account in the structural modelling and hazard specification, the local geographical and seismic conditions as well as particular characteristics of each IB, generating more specific vulnerability curves region dependent. For each IB, pushover curves are derived for both principal directions (longitudinal and transverse) of the building in order to identify the weakest direction. F/V functions are then generated and documented for the weaker direction only. It is acknowledged that specific detailed strengthening strategies should be dependent on a full 3D analysis of vulnerability considering the reduction of fragility/vulnerability.

The general methodological approach proposed in the framework of GLoSI to generate representative and comprehensive F/V functions for an IB using the analytical approach are the following:

a) Seismic hazard definition: hazard is defined in terms of the acceleration spectra of a set of 22 earthquake ground motion records given by FEMA P-695 that represent the following typical seismic environment:
    a. High seismicity.
    b. Both subduction and shallow type of seismicity.
    c. Peak ground acceleration greater than 0.2g.
    d. Peak ground velocity greater than 15 cm/sec.
    e. Magnitude greater than Mw=6.5.
    f. Rock and soft soils sites.

b) Definition of index buildings: the taxonomy parameters and intrinsic characteristics (geometrical characteristics and material properties) for the IBs (refer to Chapter 3).
c) Numerical modelling and non-linear pushover analysis: reliable 3-D numerical models of index buildings are generated, and non-linear pushover analyses are performed in order to generate the pushover curves. Any acceptable methodology and/or software can be used for the pushover curve derivation. It should be noted that for flexible diaphragm type LBM structures, the pushover curves are generated with respect to global IP and global OOP behaviors separately (discussed in detail in section 4.2.4).
d) Seismic performance assessment (N2 method): the non-linear static approach is selected based on the latest version of the N2 method (Fajfar, 2000, D'Ayala et al., 2015). For each pushover curve, the thresholds of discretized damage states represented by the roof drift are determined in terms of specific element and global damage indicators. The definition of damage states and associated threshold limits can be code-based, from the available literature or IB specific. In GLoSI, the approach adopted is to identify building index- specific damage states, obtained through validation with experimental and field observations available in literature. This is preferred to the reliance on codal prescriptions, which are affected by expert opinion, which is not easily traceable. For each IB, the building or MDoF pushover curves are converted to bilinear idealized pushover curve of the equivalent single degree of freedom system (SDoF) following standard rules (detailed discussion presented in section 4.2.6.2). This is intersected to the demand spectrum of each of the different ground motions suite (scaled to different values of IM) to generate a number of seismic performance points (IM vs. EDP) ranging from slight damage to complete damage thresholds.
e) Derivation of fragility functions: building-based fragility assessment is conducted for the derivation of fragility functions for each damage state with the help of the cloud of performance points (IM vs. EDP). For the derivation of fragility functions, the least square regression method is used for each damage state (D'Ayala, 2015). It should be noted that for flexible diaphragm type LBM

structures, the fragility curves are generated with respect to global IP and global out of plane (OOP) behaviors separately (discussed in detail in section 4.2.4).

f) Derivation of vulnerability functions: For the derivation of vulnerability function for LBM IBs, either the building-based method (in case of rigid diaphragm type structures) or the component-based (in case of flexible diaphragm structures, more details provided in section 4.2.4) method is followed. The building-based method implies convolving building-level fragility curves with the cumulative distribution of the total cost (D'Ayala, 2015). Whereas, component-based methodology is followed to derive vulnerability functions for all cases for RC IBs using the approach proposed by Yamin et al. (2017).

Figure 4-2 summarizes the main steps of the methodology used in the present study to derive F/V functions. Each component of the proposed methodology is described in detail in the following sections.

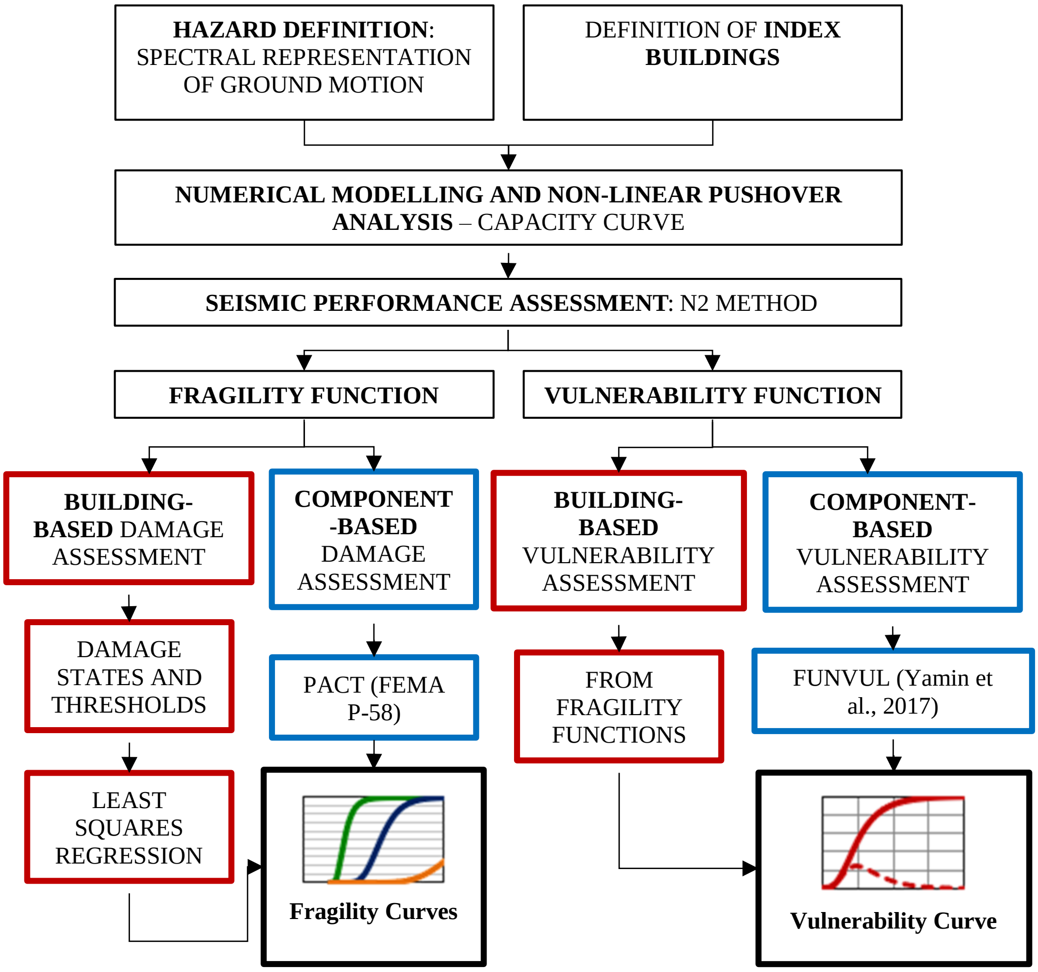


Figure 4-2. General fragility/vulnerability assessment methodology. Note that the red and blue colors represent steps for building-based and component-based fragility/vulnerability assessment methodology, respectively.

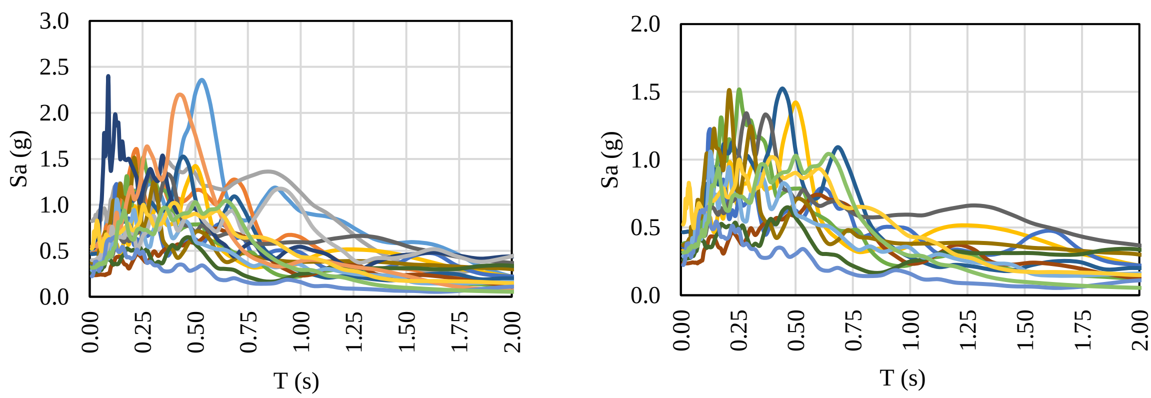
4.2.1 Hazard Definition

The proposed hazard definition is based on the FEMA P-695 (FEMA, 2009) approach, which considers a set of pre-selected seismic records for two different distance criteria: far-field and near field. In Table 4-1 the selection criteria for these two groups of records is presented.

Table 4-1. FEMA P695 ground motion selection criteria.

	Far Field	Near Field
PGA	>0.2 g	>0.22 g & <1.43 g
PGV	>15 cm/sec	>30 cm/sec & < 167 cm/sec
Distance	-	>1.7 km & <8.8 km
Minimum Mw	Mw>6.5	Mw>6.5
Soil Type	Soft rock and stiff soil sites (C&D)	Soft rock and stiff soil sites (C&D)

Figure 4-3 presents the collection of acceleration response spectra for the far field and nearfield records. The collection of seismic record is included in a separate digital **Annex G**. For this specific project, F/V assessment will use far field records only and a sensitivity analysis will include near field set. For country specific assessment, it is advisable to use ground motion sets obtained from local seismological networks or historic records, wherever available.



a) b)

Figure 4-3. a) Far-field ground motion response spectra and b) Near-field ground response spectra.

No specific additional consideration to the type of soil is made. Buildings located in particularly vulnerable soil conditions such as very soft soils (NHERP E or F type of soils) or with specific topographic conditions, such as steep slopes, for which the above selected records would not be applicable, would need a site-specific F/V assessment.

4.2.2 Definition of Index Building

Index buildings representative of different building types of common LBM and RC school buildings around the world (see Chapter 3) are selected for F/V assessment. A total of 14 LBM and 23 RC IBs are identified considering the main lateral load structural resisting system, the height range and the level of seismic design. Each IB is further characterized by:

a) The most applicable attribute's range of the secondary taxonomy parameters such as diaphragm type, structural irregularities, slenderness, structural health conditions, etc.
b) The intrinsic characteristics including the plan characteristics, geometrical details of the main structural components and the material properties.

4.2.3 Numerical Modelling

4.2.3.1 LBM Index Buildings

The following are the assumption and strategy adopted for the structural modelling of LBM School IBs:

a) Full 3-dimensional numerical models are developed for LBM IBs, with an element by element non-linear modelling approach resulting in a simplified micro-modelling technique, based on the applied element method (AEM) (Meguro and Tagel-Din, 2001). In the AEM, masonry is modelled using simplified micro-modelling technique (Figure 2 4), in which the applied elements are modelled as rigid elements whereas the joint and the mortar-unit interfaces are sandwiched into one element which is represented by the joint springs. If the units are expected to damage, the units can be divided into several elements (usually two) by having unit springs in between the applied elements of the units. All the stresses, deformation and non-linearities in the material behavior are thus represented in the joint springs (Meguro and Tagel-Din, 2001). Using the applied element method, several studies have been conducted in the past on masonry structures under static and dynamic analysis (Pandey and Meguro, 2004; Karbassi and Nollet, 2013; Guragain, 2015 etc.) as the complete response of structures from the initiation of cracking to the final collapse can be studied with reasonable accuracy (Meguro and Tagel-Din, 2001). The version of this approach codified in the Extreme Loading for Structures® (ELS) software (ASI, 2018) is used in the present work. However, it is possible to develop the numerical model and perform pushover analysis with any other software such as macro-element based approaches (e.g. TREMURI (Lagomarsino et al., 2013)) or FEM based software (e.g. ABAQUS (ABAQUS, 2013)).
b) As shown in Figure 4-5, the layout of the masonry units and the resulting bond (e.g. running bond or English bond) is accurately modelled. The construction details such as lintels, diaphragm structure (e.g. slab) etc. are also modelled appropriately. The lintels are modelled as elastic continuous elements. Further, in the numerical model of confined masonry IBs, the tie-beams and tie-columns are modelled explicitly. Similarly, in the numerical model of reinforced masonry IBs, the reinforcements along with the grout is also modelled explicitly inside the concrete blocks.
c) Foundation flexibility is not considered in the present work.
d) The mechanical characteristics and parameters needed for the numerical model of each IB are calibrated using validation against experimental results on tested masonry walls for the specific masonry fabric considered.
e) The elastic and non-linear material properties (modulus of elasticity, compressive strength, tensile strength, friction coefficients etc.) for units, mortar and the masonry are established from available literature based on experimental test results.
f) Dead loads and live loads are considered in the analysis. The total dead load consists of the self-weight all the structural elements as well as the weight of the non-structural elements such as

roof, ceiling etc. In multi-story buildings, 25% of the design live load is considered in the total seismic weight of the structure (ASCE, 2013).

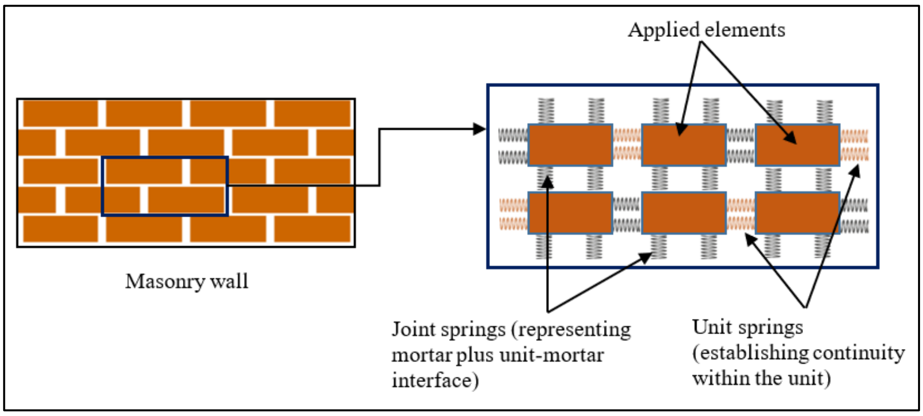


Figure 4-4. Simplified-micro modelling technique for masonry adopted in the numerical modelling.

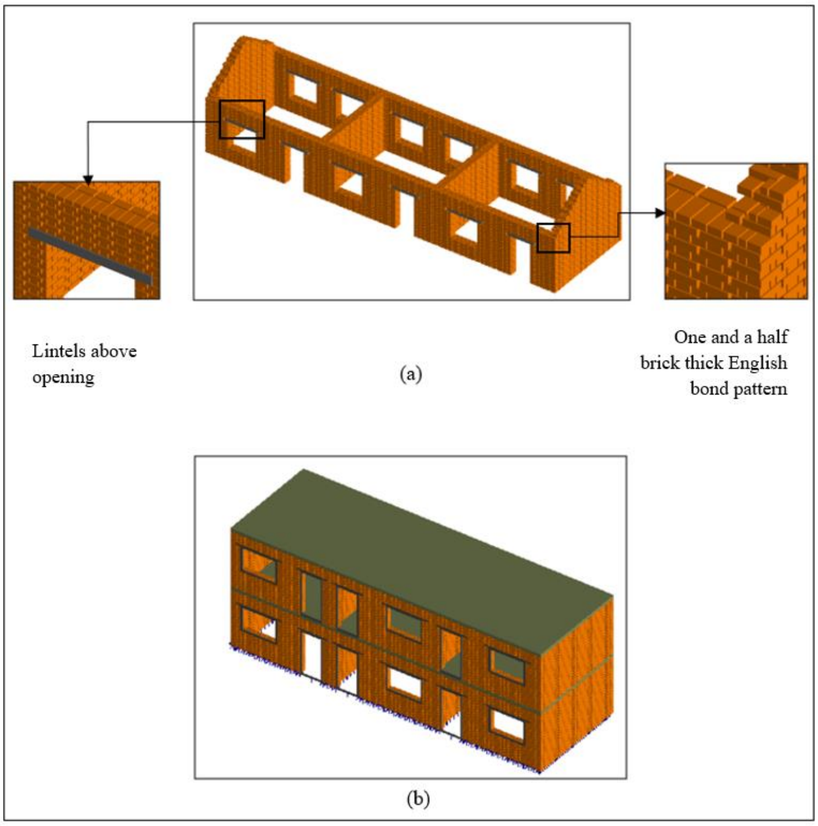


Figure 4-5. Example 3-dimensional numerical models of a) a single-story brick in mud mortar (UCM-URM4) IB and b) a two-story brick in cement mortar (UCM-URM7) IB.

4.2.3.2 RC Index Buildings

For the RC structures, the following are the modelling considerations:

a) General considerations:

- 3-dimensional model.
- Concentrated plasticity for beams and columns (hinges) and distributed plasticity for walls (fiber model).
- Consideration of P-Delta effects.
- Concrete beam and column and masonry infills modeled as FRAMES and concrete walls as SHELLS.
- Consideration of rigid zones for RC.
- Rigid diaphragms in floors and roofs when a concrete slab is present (for particular cases of very thin slabs or irregular plan shapes, diaphragm flexibility considerations may be required).
- Cracked sections for the main structural elements.

Figure 4-6 illustrate a typical model of a plastic hinge for a representative structural component model such as a beam, a column, a wall or any other.

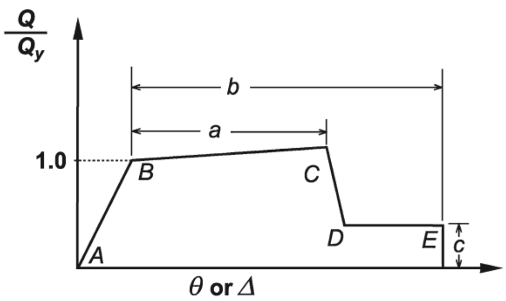


Figure 4-6. ASCE 41-17 general shear or flexural plastic hinge behavior (ASCE, 2017).

b) Loads considered in the analysis:
   - Self-weight of all elements.
   - Additional dead loads (slabs, nonstructural walls, roofs, ceilings, etc.,).
   - A permanent 25% of the design live load is considered for the non-linear analysis

c) Foundations flexibility consideration:
   - In general, a fixed based condition is adopted.
   - Flexible foundation conditions are considered in particular sensitivity analysis for one IB. For this, linear springs are included in the base of the main structural elements.

d) Proposed software: Perform3D (CSI, 2013), SAP2000 (CSI, 2017) or SeismoStruct (Seismosoft, 2016).

4.2.4 Non-Linear Pushover Analysis and Pushover Curve Derivation

The first phase of the assessment consists of determining the capacity curve for the IB under pushover analysis. A pushover curve relates the total horizontal base shear of the building with respect to the corresponding roof displacement. Figure 4-7 illustrates a typical pushover analysis procedure and the resulting curve for a building. For LBM and RC structures with rigid diaphragms, usually the fundamental mode shape is used for the application of pushover loading. The derivation of pushover curves for these types of structures is straightforward and is simply derived by summing up the total base shear and plotting it with respect to the roof master node displacement.

The pushover curve is used as an input for non-linear static seismic performance assessment procedures, in this case the N2 Method (Fajfar, 2000; D'Ayala et al., 2015), to predict the seismic performance of a building to a specific ground motion. During such assessments, the quantities in the building pushover curve are transformed into response measurements of an equivalent single-degree-of-freedom (SDoF) system (Figure 4-7). In addition, the pushover curve is also used for the determination of building specific damage state threshold definitions (section 4.2.5).

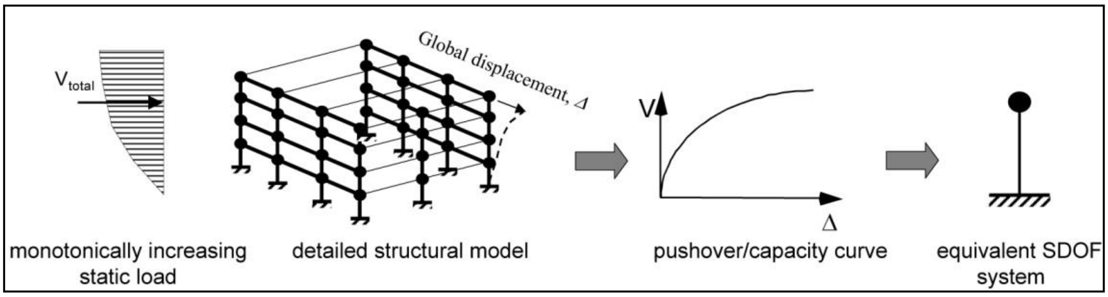


Figure 4-7. Schematic representation of static pushover analysis (excerpted from: FEMA 440 (FEMA, 2005))

Conventional pushover analysis of blocky masonry structures modelled using element-by-element modelling technique, with discontinuous joint represented by finite strength and stiffness springs, is a complex task as the application of pushover forces/displacements on the structure often causes strain concentration on a particular element or region thereby causing local failure without affecting the rest of the structure. Thus, in the present study, the numerical models of LBM IBs are subjected to a non-linear pushover analysis under linearly increasing ground acceleration (rather than a force pattern on the structure) until collapse. This causes an application of an increasing 'effective earthquake force' on the structure as illustrated in see Figure 4-8 (Chopra, 1995). Such analysis represents a force-based non-linear pushover analysis, as opposed to the displacement-based pushover analysis, usually implemented for frame structures.

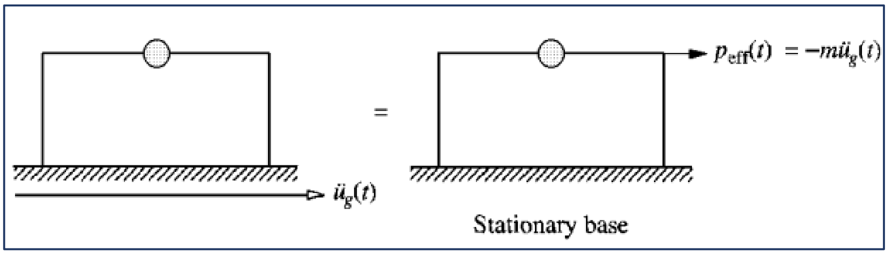


Figure 4-8. Illustration of an effective earthquake force application to a structure under the application of a ground acceleration (Adapted from Chopra, 1995).

Observations from post-earthquake damage surveys (e.g. D'Ayala and Paganoni, 2011; Moon et al., 2014; Penna et al., 2014) show that the out of plane (OOP) walls sustain heavy damage before the in plane (IP) walls suffer any significant damage when the diaphragm is flexible type. This is mainly because of the weaker stiffness of the OOP walls compared to that of the IP walls and the absence of diaphragm action at the floor/roof level to control the global displacement. At a given instant of seismic loading, the OOP walls are subjected to more displacements that IP walls, and structural damage is directly related to the drift (Figure 4-9 and Figure 4-10).

Moreover, because of the substantial difference in stiffness, IP and OOP walls tend to have different natural frequency of vibration, and it can hence be inaccurate to represent the whole building with one SDoF, as this would necessarily have characteristics averaged among the ones of the different walls. Therefore, the procedure presented here for the application of N2 method (Fajfar, 2000, D'Ayala et al., 2015) is also conducted with respect to OOP and IP walls separately.

For the reasons discussed above, pushover curves and fragility functions are derived separately with respect to OOP behavior and IP behavior. Nonetheless, although the pushover curves and hence the fragility functions are generated with respect to IP and OOP walls separately, the interaction the interaction among walls in the two orthogonal direction is correctly simulated by the three-dimensional numerical models developed, depending on the level of connection among the two sets of walls observed in the IB under consideration. Hence, such interaction affects the pushover curves and the identification of damage thresholds, and ultimately both fragility functions and vulnerability.

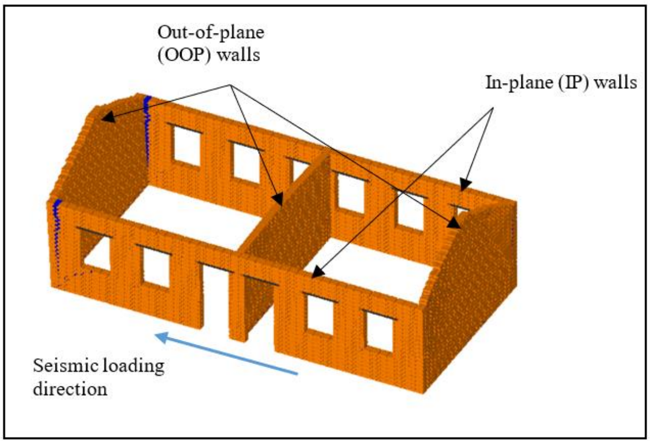


Figure 4-9. Damage due to seismic loading in an unreinforced masonry building with flexible diaphragm. The OOP walls have been already damaged heavily while the IP walls haven't suffered any serious damage yet.

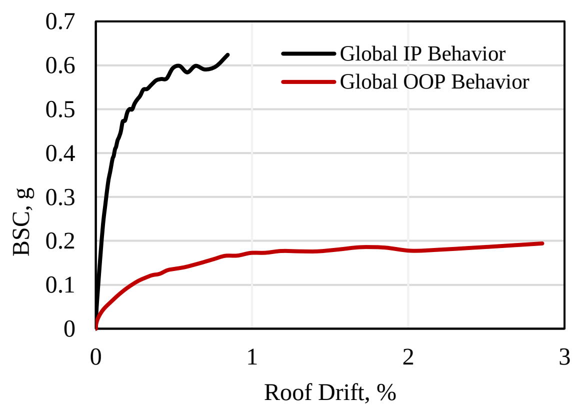


Figure 4-10. Typical capacity curves for an unreinforced masonry building when loaded in longitudinal direction.

In line with the above discussion, the following procedure is applied for the generation of pushover curves for LBM IBs with flexible type diaphragms:

a) The pushover curve for individual walls (in-plane and out-of-plane) are extracted by recording the base shear at the base of the walls vs the roof displacement of the corresponding wall.
b) Threshold for different damage state (explained in Section 4.2.5) for individual walls are identified along the respective pushover curve based on the progressive damage associated with the wall response. Corresponding drift limits for different damage state thresholds are identified.
c) All the pushover curves of the walls acting in IP behavior are integrated by summing up the base shear and averaging the roof displacement of each IP walls at each instant of loading to generate the global pushover curve for IP behavior in a particular loading direction. Similarly, all the pushover curves in the walls acting in OOP behavior are integrated by summing up the base shear and averaging the roof displacement of each OOP walls at each instant of loading to generate the global pushover curve for OOP behavior in the same loading direction.
d) Finally, the fragility curves are also developed with respect to global IP and OOP behavior, respectively.

4.2.5 Damage States and Thresholds

Five different discretized damage states are considered for each structural component (column or wall) or at building level: no damage (ND), slight damage (SD), moderate damage (MD), extensive damage (ED) and complete damage (CD) or collapse state. Table 4-2 presents the general definition of the damage states (ND, SD etc.) and their corresponding damage thresholds or performance points (DT1, DT2 etc.), i.e. the particular event which can be identified in the push over analysis which determines a change in structural response and hence a new damage phase. The threshold of each damage state for global building behavior is defined as the point when the first structural component (wall or column) starts to enter the corresponding damage state. For example, the threshold for slight damage state is the point when one of the structural components (e.g. masonry pier/wall or an RC column) enters the slight damage state (e.g. hairline cracks have started to appear). Also shown in Table 4-2 are the seismic performance levels according to ASCE 14-13 (ASCE, 2013) equivalent to the different damage states considered. These definitions are illustrated also in Figure 4-11.

Table 4-2. Definition of Damage States and Damage Thresholds

Damage Threshold or Performance Point	Definition of Threshold of Damage State	Damage State	Equivalent Seismic Performance Level (ASCE 41-13)
-	-	No Damage (ND): up to DT1	Operational (OP): up to DT1
Slight Damage Threshold (DT1)	Elastic (cracking limit), slight reduction in initial stiffness starts	Slight Damage (SD): DT1 to DT2	Immediate Occupancy (IO): DT1 to DT2
Moderate Damage Threshold (DT2)	Strength is increasing, stiffness starts to reduce noticeably as all the structural components have achieved slight damage state	Moderate Damage (MD): DT2 to DT3	Life Safety (LS): DT2 to DT3

Extensive Damage Threshold (DT3)	Peak strength is achieved as all the components have attained moderate damage state, stiffness changes from positive to zero. The structure now enters plastic deformation state i.e. it will withstand certain deformation at a constant capacity.	Extensive Damage (ED): DT3 to DT4	Collapse Prevention (CP): DT3 to DT4
Complete Damage Threshold (DT4)	Some structural components start to fail (loosing load resisting capacity), stiffness and strength start to degrade considerably. Further lateral deformation will cause the structure to collapse.	Complete Damage or Collapse (CD): after DT4	Collapse after DT4

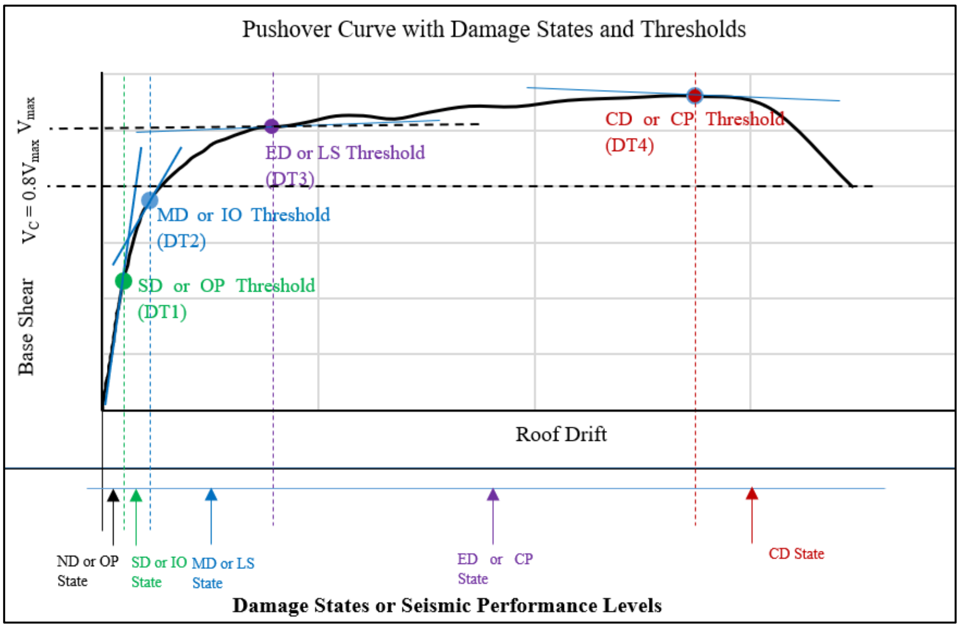


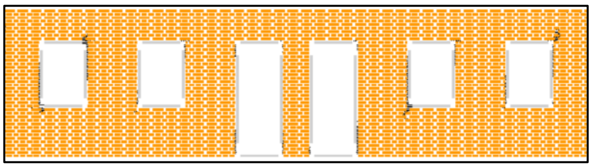
Figure 4-11. Definition of damage states (or seismic performance levels) and damage state thresholds along the pushover curve.

For masonry buildings, the damage state thresholds are identified based on the crack pattern, extent and maximum width of cracks occurring on each wall, depending on the prevalent seismic response of the wall (in shear or bending depending on in-plane or out-of-plane prevalent loading). These element's damage thresholds, obtained from literature, experiments and standards, are marked on each elements capacity curve and correlated to the drifts and changes in strength and stiffness as obtained from the 3D analysis. In the global pushover curves, each global damage state threshold (expressed in terms of roof drift) is reached and overcome when the first wall enters the respective damage state. For each global IP or OOP behavior, the global collapse is defined when one of the walls reaches the collapse damage state.

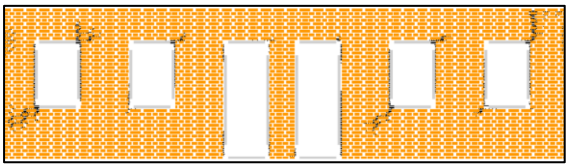
Table 4-3 and figure 2-12 illustrate the physical definition of four different damage state thresholds for an unreinforced masonry wall under IP behavior. It should be noted that the crack pattern development as well as width at different damage state threshold is dependent on the masonry fabric and connections among walls for each IB.

Table 4-3. Example physical definition of damage states for an unreinforced masonry wall under IP behavior.

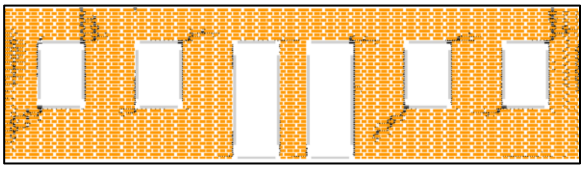
Damage Threshold Definition	Physical Damage Definition
Slight Damage Threshold (DT1)	Hairline cracks (about 0.1 - 1 mm width) on few corners around the openings.
Moderate Damage (MD) Limit (DT2)	Hairline to minor cracks appear on all the corners around opening, minor flexural cracks of about 1 mm - 5 mm width appeared in few spandrels, diagonal shear cracks (about 1 mm - 5 mm maximum width) start to appear in some piers.
Extensive Damage (ED) Limit (DT3)	Most of the piers and spandrels have developed minor flexural/diagonal shear cracks (about 5 mm in width). Few spandrels and piers start to develop major flexural/shear cracks of 10 mm maximum width.
Complete Damage (CD) or Collapse Limit (DT4)	Most of the spandrels and piers have already developed a major crack of about 10 mm width. Few spandrels damaged with an extensive crack width of 10 mm to 15 mm and few piers start to develop extensive cracks in shear or combined shear-flexure mechanism with a maximum crack width of about 15 mm.



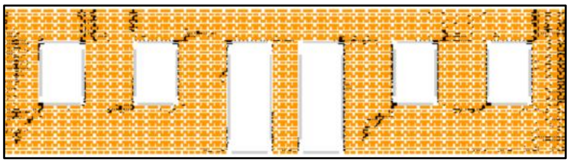
a) Slight Damage Threshold (DT1) (maximum crack width - 1 mm)



b) Moderate Damage Threshold (DT2) (maximum crack width - 5 mm).



c) Extensive Damage Threshold (DT3) (maximum crack width - 10 mm).



d) Complete Damage or Collapse Threshold (DT4) (Maximum crack width - 15 mm)

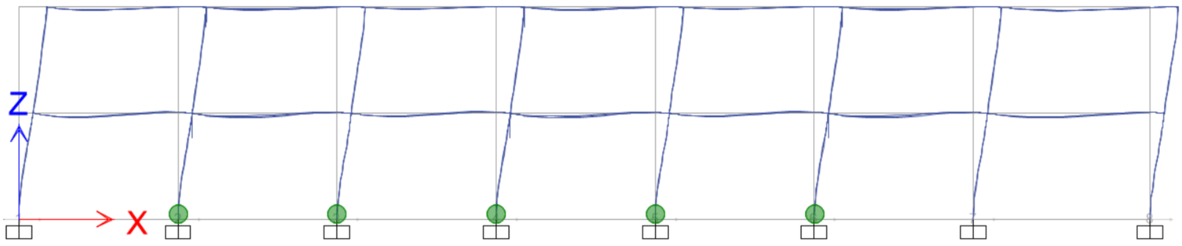
Figure 4-12. Illustration of different damage state thresholds for an unreinforced masonry wall under IP behavior (black lines represents the cracks, figures shown in reduced scale to better represent the cracks).

For RC structural systems, the precise definition of the limits of each damage threshold is rather difficult if there is no detailed model developed. In literature different approaches have been adopted, one of this consists in establishing different ranges in the non-linear part of the pushover curve. For instance, document FEMA-356 (FEMA, 2000) recommend some fixed limits for reinforced concrete frames such as the following: Immediate occupancy: 1% Roof drift, Life safety: 2% Roof Drift and Collapse prevention 4% Roof Drift. However, those limits will only be applicable for frames with some specific characteristics. Another way to define damage states is based on recommendations from SEAOC (1999) whereby the thresholds are defined by identifying the first yield and the collapse point first, and then the intermediate points as percentages of the plastic displacement range.

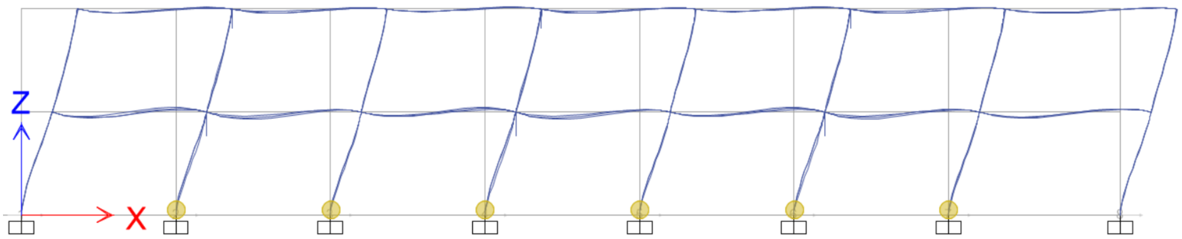
In the present study, for RC school buildings, the definition of damage limit thresholds, corresponding drift limits for each of the damage states, are extracted from the damage progression analysis under increasing seismic action. Each damage state is defined by the limit state of the plastic hinges developing in the model's elements. For instance, when the first hinge in the first column reaches the yielding threshold, as per Figure 2.6, the whole building is considered in slight damage threshold. In this way the approach for RC structures and for LBM structures is consistent. The description of each damage threshold for RC buildings is shown in Table 4-4 and is illustrated in Figure 4-13.

Table 4-4 Example physical definition of damage states for an RC school building.

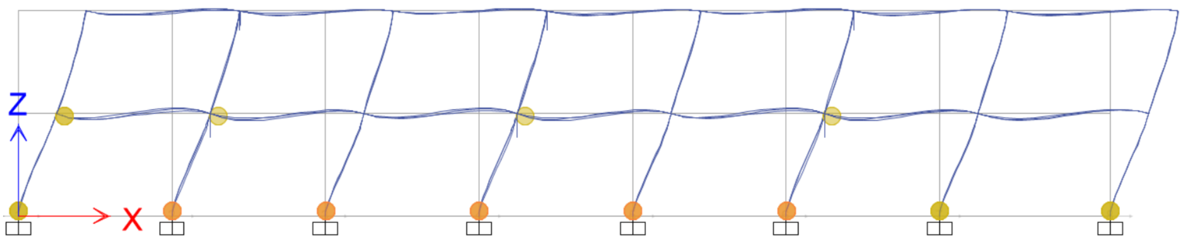
Damage Threshold Definition	Physical Damage Definition
Slight Damage Threshold (DT1)	When the first plastic hinges exceed Immediate Occupancy (IO) performance point defined by ASCE 41-17.
Moderate Damage (MD) Limit (DT2)	When the first plastic hinges exceed Life Safety (LS) performance point defined by ASCE 41-17.
Extensive Damage (ED) Limit (DT3)	When the first plastic hinges exceed Collapse Prevention (CP) performance point defined by ASCE 41-17.
Complete Damage (CD) or Collapse Limit (DT4)	When collapse mechanism is developed and structure loses its capacity (negative stiffness).



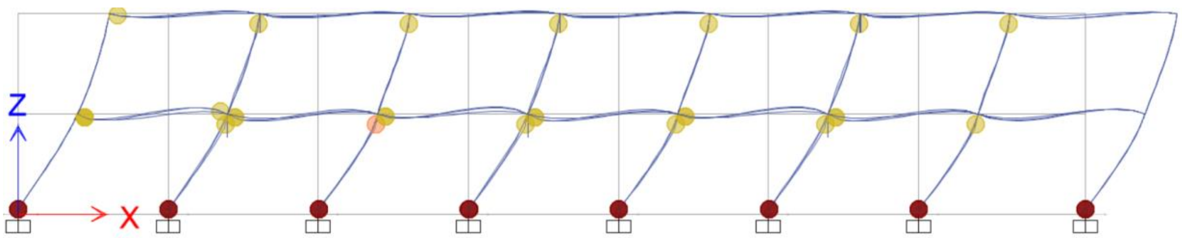
Slight Damage Threshold (DT1): First columns plastic hinge exceeds IO limit (green).



Moderate Damage Threshold (DT2): First columns plastic hinge exceeds LS limit (yellow).



Extensive Damage Threshold (DT3): First columns plastic hinge exceeds CP limit (orange).



Complete Damage Threshold (DT4): Collapse mechanism developed.

Figure 4-13. Example illustration of different damage states for RC IB.

The roof drift is considered as the engineering demand parameter (EDP) in this study. The seismic performance assessment as well as fragility derivation is carried out with respect to the roof drifts and each limit of the damage state is represented by the corresponding roof drift. As mentioned before, some judgement is required in order to define the limits for each damage state and therefore the final fragility functions shall be used and interpreted with this clear limitation. Also, specific IB types may have slightly different definition of damage limit states, which will be specified in the analysis of the respective IBs.

4.2.6 Seismic Performance Assessment: N2 Methodology

A simplified non-linear static seismic performance assessment methodology is adopted based on the N2 method first proposed by Fajfar (2000). For the GLoSI library, the proposed approach is adapted mainly from D'Ayala et al. (2015). The seismic performance assessment procedure using N2 method is articulated in the following steps, each of which is thoroughly described in the following sections of this document:

- Convert the MDoF pushover curve (obtained with the procedure described in Section 4.2.4) to SDoF pushover curve and then transform the resultant capacity curve to ADRS (acceleration displacement response spectra) format.
- Idealize the capacity curve into a bilinear curve based on the principle of equivalent energy (Eurocode 8).
- Compute the elastic and inelastic spectrum for the ground motion record. Obtain the expected performance point using the N2 methodology, which will correspond to the maximum spectral displacement of the structure.
- Calculate the corresponding horizontal roof displacement and then the roof drift (EDP) which are back-calculated from the maximum spectral displacement at the performance point.
- Repeat the procedure to generate the EDPs (i.e. roof drift) for each IM with a number of scaled ground motion spectra.

For the seismic performance assessment and the generation of IM vs EDP results for a number of ground motions, a Microsoft Excel® based program named "N2_Bilinear_Capacity_Curve.xlsx" is developed and available in the GLoSI library (see **Annex H**), at CAPRA website (www.ecapra.org) and at the UCL EPICentre website (https://www.ucl.ac.uk/epicentre).

4.2.6.1 Derivation of Equivalent SDoF Capacity Curves

The SDoF system is a virtual oscillator, which has the same natural frequency and elastic properties (e.g. stiffness) as that of the MDoF system building. More precisely, the applied load is translated into spectral acceleration, and the lateral deformation is translated into spectral displacement. The pushover curve represented by these two parameters is called the capacity curve. A building's capacity curve reflects various seismic characteristics of the building, such as its stiffness, its material brittleness or ductility, and its strength. This curve correlates the lateral deformation of the building (in terms of spectral displacement) to a specific level of dynamic demand (expressed in terms of spectral acceleration). The transformation of the Force-Displacement (F-D) curve to Acceleration-Displacement Response Spectra (ADRS) format is done using the modal participation factors and effective modal weight ratios, determined from the fundamental mode of the structure. The procedure is summarized below:

- Run Eigen value analysis and extract the fundamental mode shapes, of multi-degree-of-freedom (MDoF) system.
- Obtain the F-D relationship (pushover curve) as result of non-linear static pushover analysis of MDoF system (Section 4.2.4).
- Derive the equivalent SDoF-based capacity curve by dividing the base shear and displacement of the MDoF-based capacity curve by the transformation factor.

The conversion of a MDoF system to an equivalent SdoF system is an established engineering procedure and the readers are referred to well established literature such as FEMA 440 (FEMA, 2005) and Fajfar (2000) for more details.

4.2.6.2 Idealization of SDoF Capacity Curve

The application of nonlinear static-based procedure (N2 method) depends on the determination of an idealized capacity curve of the equivalent SDoF system. This curve is derived by using the equal energy principle, imposing that the area under the SDoF capacity and idealized curve is equal. Several forms of capacity curve idealization models exist in the literature, including the simple bilinear elastic-perfectly plastic model and the multilinear elastic-plastic amongst others. In the present study, it is assumed that the idealized curve follows a simple bilinear elastic-perfectly-plastic form (EPP hereafter). In EPP idealization of the capacity curve, the elastic segment is defined from ordinate zero to the yielding point, and the plastic segment is a plateau from the yielding point to ultimate deformation at the collapse (refer to Figure 4-14).

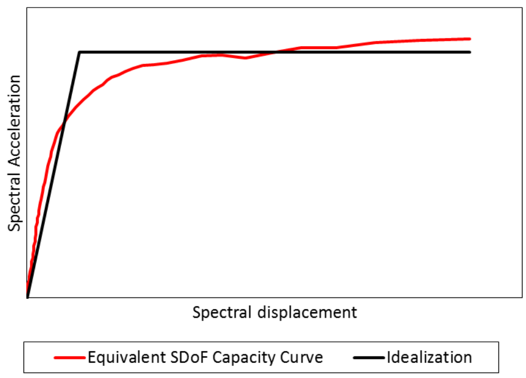


Figure 4-14. Example bilinear idealization of a capacity curve.

The choice of idealization approach is of high importance for determination of the seismic response of a structure. Although several variations of multilinear idealization models exist in literature, most design codes/guidelines (e.g. FEMA-440, EC8 etc.) recommend simple bilinear idealization model fitting (elastic-plastic or elastic strain-hardening). The simplicity of the bilinear shape means that one only needs to estimate the position of the nominal 'yield point' and the 'ultimate point'.

A summary of the some of the most commonly used fitting capacity idealization approaches found in design codes/guidelines are discussed below. Eurocode 8, following the original N2 method, suggests an elastic perfectly plastic idealized capacity curve based on the balancing of the area discrepancy above and below the fit (equal energy rule), optionally using an iterative procedure. In this case, the capacity corresponding to the idealized yield point i.e. F_y^* is taken as $1*F_u$, where F_u is the maximum capacity of the actual pushover curve (Figure 4-15).

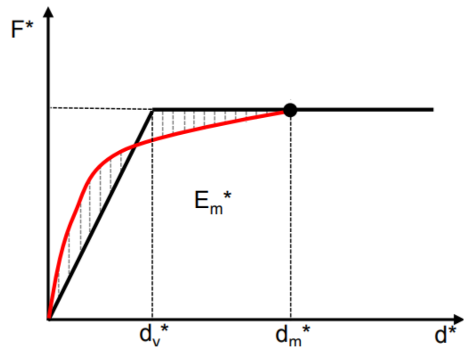


Figure 4-15. Bilinear fitting procedure according to EC8 (excerpted from De Luca et al 2013a).

Federal Emergency Management Agency (FEMA) documents (e.g. FEMA 440 and ASCE/SEI 41-06) generally employ a bilinear model with an initial slope and a post-yield slope (either positive or negative) up to the target point. The initial effective slope is calculated at a capacity equal to 60% of the nominal yield strength. In all cases, the idealized elastic-hardening shape is fitted through an iterative procedure approximately balancing the area above and below the fitted curve (Figure 4-16).

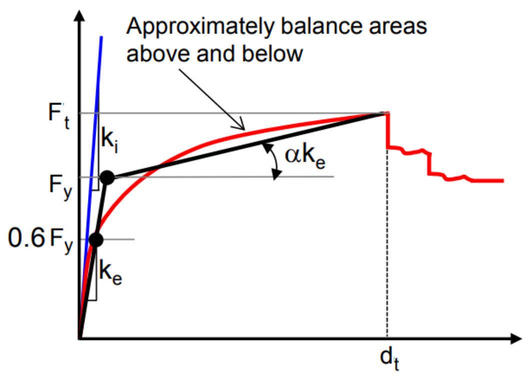


Figure 4-16. Bilinear fitting procedure according to FEMA 440 (excerpted from De Luca et al 2013a).

Italian guidelines (Decreto Ministeriale del 14/01/2008) suggest an elastic-plastic fit that may also account for a limited softening behavior up to a point of a 15% degradation of maximum capacity in the capacity curve. The initial stiffness fit is also based on the 60% rule[2], as in all FEMA documents. An equal energy criterion is then applied to derive the plateau of the bilinear fit. In the cases that structural model does not reach a negative stiffness, this model becomes equivalent to the Eurocode 8 fitting model. In this case, the capacity corresponding to the idealized yield point i.e. F_y^* is taken as $0.85*F_u$. (Figure 4-17).

[2] The ratio of the base shear at the intersection of idealized and exact capacity curve over the maximum (ultimate) base shear of the exact capacity is equal to 60%.

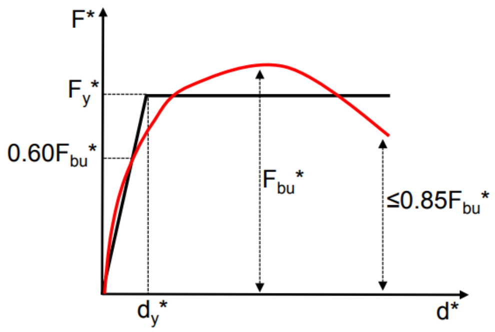


Figure 4-17. Bilinear fitting procedure according to Italian guidelines (excerpted from De Luca et al 2013a).

De Luca et al. (2013a, b) studied a comparison of the abovementioned fitting procedures and evaluated the results against incremental dynamic analysis (IDA) results. These comparisons showed that the bilinear fitting proposed in FEMA and the Italian guidelines provided the most acceptable errors to the IDA results, for the cases of capacity curves with strain hardening and strain softening respectively.

The index buildings analyzed in the present work showed a great variety of capacities. Therefore, an alternative strategy for capacity idealization (based on the principles of some of the fitting approaches discussed above) is used. Specifically, a bilinear elastic-perfectly plastic model using the equal energy rule and the 60% capacity rule for initial stiffness principle is employed. To this aim, a sensitivity test for the determination of the optimal value of F_y was carried out and it is found that a value of F_y^* equal to 0.95*F_u satisfies the 60% capacity rule for initial stiffness principle for all index buildings in average. Thus, the authors suggest idealizing the capacity curves into a bilinear elastic perfectly plastic curve with equal energy principle and the recommended value of F_y^* is 0.95*F_u.

4.2.6.3 Determination of Seismic Performance Point

Once the idealized capacity curve is determined, one needs to select a seismic record, or a suite of seismic records to represent the seismic demand. For each selected record and the associated 5% damped elastic response spectrum (S_{ae}(T), S_{de}(T)), the inelastic response spectrum (S_a(T), S_d(T)) is derived by means of an R − μ − T relationship; where R is the reduction factor, μ is the ductility and T is the natural period of vibration of the SDoF system:

$$S_a(T) = \frac{S_{ae}(T)}{R} \tag{5}$$

$$S_d(T) = \frac{\mu}{R} S_{de}(T) = \frac{\mu}{R}\frac{T^2}{4\pi^2} S_{ae}(T) = \mu \frac{T^2}{4\pi^2} S_e(T) \tag{6}$$

The seismic performance can be obtained graphically by extending the elastic branch of the idealized capacity curve up to the intersection with the elastic demand spectrum (see Figure 4-18). By performing a number of iterations, the intersection of the capacity curve with the inelastic demand spectrum for the correct value of ductility is then identified. This point is known as the performance point and links the seismic performance of the building, expressed in terms of EDPs, with the seismic demand, expressed in terms of ground motion intensity measures (IMs).

For a given earthquake ground record, the performance point of the equivalent SDoF system can be calculated with respect to the following two conditions:

- For medium and long period range: $T^* \geq T_C$
- For short period range: $T^* < T_C$

Where T_C (also known as the corner period) is the characteristic period of the ground motion, which identifies the transition from constant acceleration (corresponding to the short-period range) to constant velocity (the medium-period range) section of the elastic spectrum. A detailed description of the performance point calculation for each of the above conditions can be found in D'Ayala et al. (2015). Using this process, a set of spectral displacement and spectral acceleration values corresponding to the performance point can be obtained.

The corresponding horizontal roof displacement and then the roof drift (EDP) can be obtained through the back-calculation from the maximum spectral displacement at the performance point. In order to determine the structure's performance under increasing ground motion intensity, the analysis described above should be repeated for multiple accelerograms scaled up until all the limit states are reached. The selected number of accelerograms/ground motions should be sufficient to provide stable estimates of the median capacities. The resulting cloud of performance points is then used to determine the median EDP for each damage state threshold and its dispersion, and then create a fragility curve by fitting a statistical model, as described in the following section.

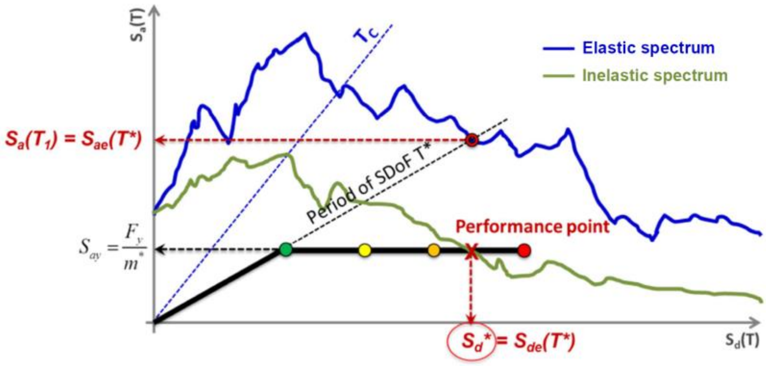


Figure 4-18. N2 graphical procedure (D'Ayala et al, 2015).

4.2.7 Derivation of Fragility Functions

The proposed methodology considers the assessment of fragility functions using a building-based damage assessment methodology. In order to generate fragility functions for the different damage levels defined, the following procedure is proposed:

a) Select all resulting performance points (IM vs EDP) as obtained from N2 analysis in the corresponding range of values to each particular damage state.
b) Using a least square method calculate the mean and variance of the resulting collection of seismic intensity values.
c) Performing piece-wise regression over these different IM intervals. Assign a log normal probability distribution function for each particular damage state.
d) Conform the collection of fragility functions for the building under consideration.

A MATLAB® based software package (**Annex I**) has been developed for the calculation of fragility functions, given a collection of EDP resulting from the seismic performance assessment at different intensity levels and will be made available in GLoSI library, at CAPRA website (www.ecapra.org) and at the UCL EPICentre website (https://www.ucl.ac.uk/epicentre). Figure 4-19 shows the cloud of IM versus EDP points (expressed in terms PGA and roof drift ratio (RDR) in this example), divided into five bins based on the four damage state thresholds corresponding to SD, MD, ED and CD damage states. Figure 4-20 illustrate an example of the resultant LSM fragility curves for the given the collection of EDP values at different intensity levels.

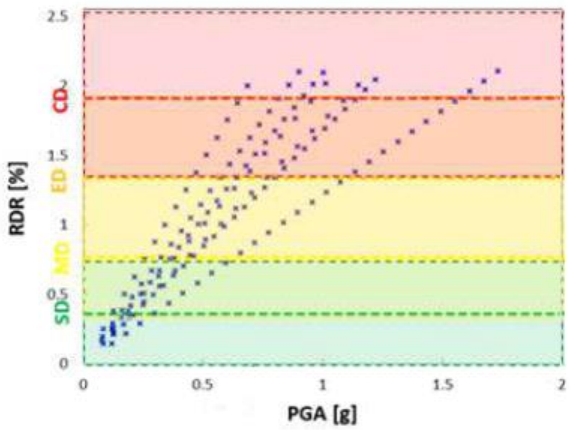


Figure 4-19. LSM Methodology (excerpted from D'Ayala et al, 2015).

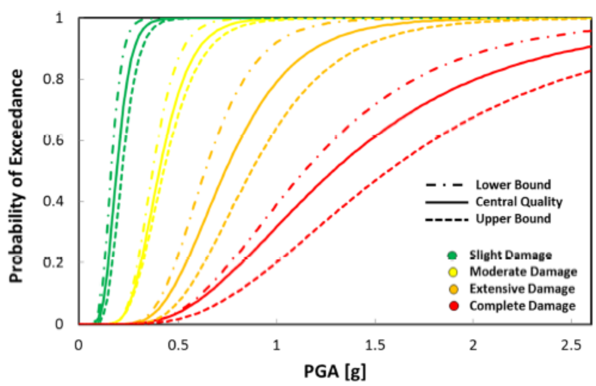


Figure 4-20. Example of a fragility function obtained by LSM Methodology (excerpted from D'Ayala et al, 2015).

Least Squares regression is a widely used technique to estimate, for each damage threshold, the probabilistic relation between EDPs and IMs (e.g. Shome and Cornell 1999). Assuming a lognormal distribution between EDP and DS, the predicted median demand is represented by a normal cumulative distribution:

$$\Phi\left[\frac{\ln IM - \ln \alpha}{\beta}\right] \tag{7}$$

Where Φ represents the standard normal cumulative distribution function, β is the global standard deviation for the predicted median demand α. For an assumed probabilistic damage threshold, IMs are chosen in a way that roughly half the points are below that damage threshold and half above, determining an interval of IMs values, which are assumed to be lognormally distributed within each interval.

Performing piece-wise regression over these different IM intervals, the fragility parameters are computed using the corresponding relation (Figure 4-21):

$$\ln(EDP) = a\ln(IM) + \ln(b) \tag{8}$$

The median demand α_{ds_i} and its dispersion β_{ds_i}, for each assumed threshold ds_i, can be written as:

$$\alpha_{ds_i} = \exp\left(\frac{\ln\left(\frac{ds_i}{b}\right)}{a}\right) \text{ and } \beta_{ds_i} = \frac{STDEV(\ln IM_i)}{a} \tag{9}$$

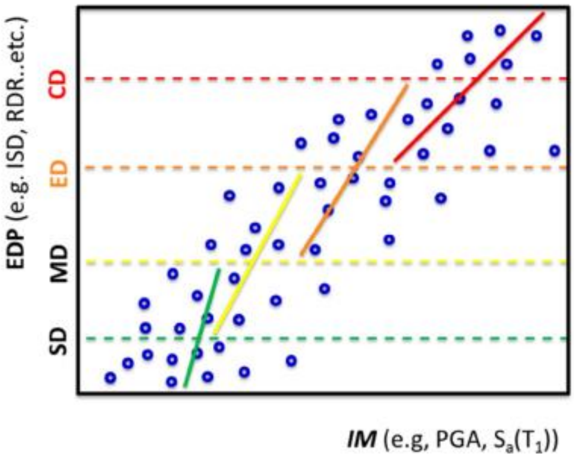


Figure 4-21. Derivation of fragility functions (median demand and dispersion) using Least Squares regression technique (excerpted from D'Ayala et al, 2015).

The proposed fragility assessment methodology is associated with a number of advantages and limitations. The main advantages of the methodology are:

- It is a simple and rapid approach which does not require a significant level of detailed information
- It is an established approach which has been used in several scenarios and applications worldwide and appears to perform well in vulnerability assessment application of similar scope.

However, this approach is also associated with the following limitations:

- It does not allow to include directly the damage levels of the non-structural components.
- The definition of the damage levels is rather subjective and therefore high uncertainty shall be associated to the final qualification in a particular damage state.
- Only one parameter is used as reference for damage assessment and it correspond to the roof horizontal displacement (either the maximum value or some kind of mean value when flexible diaphragms are present).

4.2.8 Derivation of Vulnerability Functions

There are two approaches for the derivation of vulnerability functions: building-based or component-based. As mentioned previously for LBM IBs, the component vulnerability models are not available/well established, thus the building-based vulnerability function derivation is employed, while for RC IBs, component-based methodology is followed.

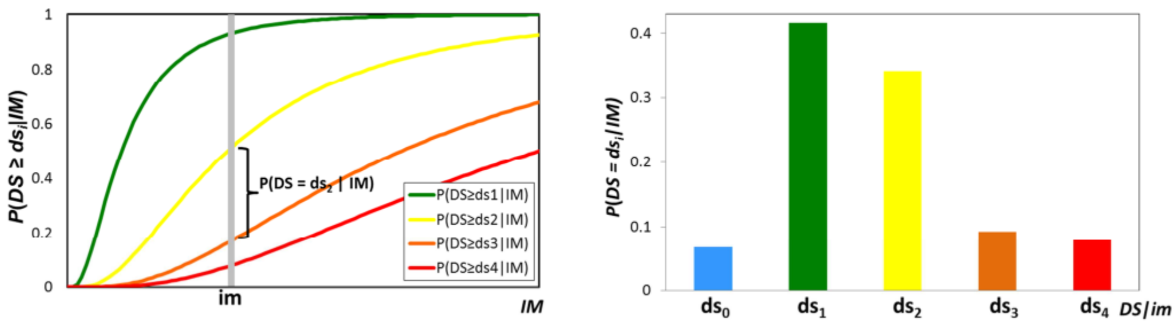
4.2.8.1 Building-Based Vulnerability Assessment Approach

for the generation of building-based vulnerability curves, the procedure suggested in the GEM analytical vulnerability guideline is employed With the building-based fragility curves for different damage states obtained in Section 4.2.7, the transformation of these curves into vulnerability curves is conducted with the following total probability relation:

$$E(C > c \mid im) = \sum_{i=1}^{n} E(C > c \mid ds_i).\mathrm{P}(ds_i \mid im) \tag{10}$$

Where, n (= 4) is the number of damage states considered; $\mathrm{P}(ds_i \mid im)$ is the probability of a building sustaining a damage state ds_i given an intensity level im; $E(C > c \mid ds_i)$ is the complementary cumulative distribution of the cost (or loss) given ds_i; $E(C > c \mid im)$ is the complementary cumulative distribution of cost (or loss) given an intensity level im.

The probability of a building sustaining a particular damage state requires the calculation of damage probabilities from the fragility curves for specific intensity levels.



(a) (b)

Figure 4-22. Calculation of damage probabilities from the fragility curves for a specific level of intensity measurement, im: a) Fragility curves corresponding to n=4 damage limit states and b) Column of the damage probabilities for different damage states given an intensity (adapted from D'Ayala et al. 2015). ds_0 = *No Damage;* ds_1 = *Slight Damage State;* ds_2 = *Moderate Damage State;* ds_3 = *Extensive Damage State;* ds_4 = *Collapse State*

Each element (or bar) in the damage probabilities is defined as the distance between two successive fragility curves for a given intensity im, as shown in Figure 4-22. The mean, $E(C \mid im)$, and the variance, $\mathrm{var}(C \mid im)$ of the vulnerability, given an im can then be obtained by the following expressions (where n = is the number of damage states considered):

$$E(C \mid im) = \sum_{i=1}^{n} E(C \mid ds_i).\mathrm{P}(ds_i \mid im) \tag{11}$$

$$\mathrm{var}(C \mid im) = \sum_{i=1}^{n} \left[[E(C \mid ds_i) - E(C \mid im)]^2 . \mathrm{P}(ds_i \mid im) \right] \tag{12}$$

Repeating the application of these two equations (2) and (3) for different levels of im (0.01g, 0.02g, 0.03g,…) will result in the vulnerability curve for the IB, similar to the one shown in Figure 4-23.

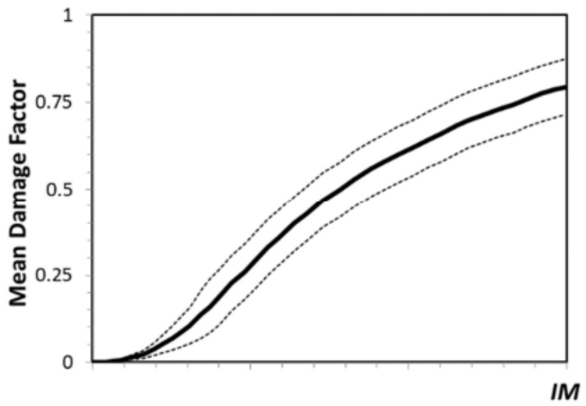


Figure 4-23. Example illustration of transformation of the fragility curves into vulnerability curve, with confidence boundaries (excerpted from D'Ayala et al. 2015).

For the cases of flexible diaphragm LBM IBs where the capacity curves and fragility functions are derived with respect to global OOP and global IP behavior, vulnerability curves are also computed separately with respect to global OOP and global IP behavior and the global building vulnerability curve is computed by adding these two vulnerability curves with appropriate vulnerability factors depending on the mass of the masonry (and the roof portion supported) by OOP and IP walls respectively.

4.2.8.2 Component-Based Vulnerability Assessment Approach

The proposed methodology considers the assessment of vulnerability functions using a component-based damage assessment. The methodology is partially based on the component-based fragility assessment method proposed in document FEMA P-58 (FEMA, 2016) and it is explained in detail in Yamin (2017). The proposed methodology is more suited for RC buildings as compared to LBM, where the interaction between structural and non-structural components define the level of damage at different intensities. It includes the following steps:

a) Definition of a model of structural and non-structural components at each story of the building.
b) Each component to be assigned a particular fragility function in terms of different damage levels and the EDP that best correspond to the damage qualification. Each damage level is associated a repair cost and time for calculating the vulnerability functions.
c) For each seismic intensity level, estimate the total repair cost and time of repair for all the collection of seismic records and all possible variations of damage states and costs of all individual structural and non-structural components.

4.2.8.2.1 Component model of the building

A component model, with both structural and non-structural elements, is to be assembled for each building under consideration. It shall include all structural and non-structural components at each story. For each type of component, the unit of measure, the quantity of elements, the fragility in terms or repair cost and time at different damage states, the controlling EDP and the correlation of damage between all the same components at the same story, have to be defined. Table 4-5 illustrates a typical component model for a two-story building.

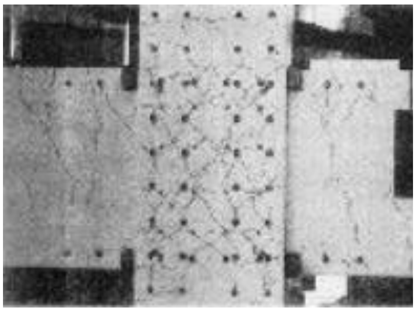
Table 4-5. Typical component model for a two-story building.

Group	Subgroup	Unit	Quantity	Fragility specification code	*EDP*	DS correlation between components
Structural	Columns and beam end nodes	Node	8	B1041.001a	Drift	No
Structural	Column and beam central nodes	Node	8	B1041.001b	Drift	No
Non-structural	Confined masonry facade	5mx3m	3	C1011.006b	Drift	Yes
Non-structural	Confined masonry partition wall (veneer)	5mx3m	1	C1011.005b	Drift	Yes
Non-structural	Confined masonry partition wall	5mx3m	2	C1011.004b	Drift	Yes
Non-structural	Plastered ceiling	5mx5m	9	C3032.005a	Acceleration	No
Non-structural	Gas piping	22ml	1	D2022.025a	Acceleration	Yes
Non-structural	Electrical piping	110ml	1	D2021.011a	Acceleration	Yes
Non-structural	Water piping	62ml	1	D2022.011a	Acceleration	Yes
Contents	Contents (acceleration controlled)	5mx5m	8	E2022.010	Acceleration	No
Contents	Contents (drift controlled)	5mx5m	8	E2022.010a	Drift	No

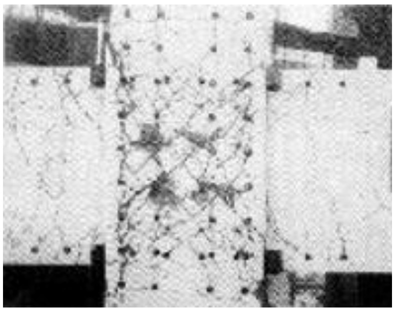
Note: Confined masonry refers to infill walls built with additional confinement elements in a framed RC building.

4.2.8.2.2 Component fragility functions

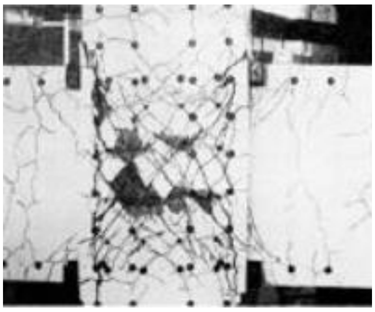
Fragility functions are to be assigned to each component type in the building. They represent the probability of being in a given damage state (usually slight, moderate or extensive) as a function of the corresponding EDP (as defined previously). Each damage state is assigned a probability density function of repair cost and time. Figure 4-24 illustrates a typical fragility definition for a beam-column connection and Table 4-6 presents the fragility function parameters for a confined masonry partition.



Damage state 1: Beams or joints exhibit residual crack widths > 0.06 in. No significant spalling. No fracture or buckling of reinforcing (FEMA, 2013).



Damage state 2: Beams or joints exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes beam and joint transverse reinforcement but not longitudinal reinforcement. No fracture or buckling of reinforcing (FEMA, 2013).



Damage state 3: Beams or joints exhibit residual crack widths > 0.06 in. Spalling of cover concrete exposes a significant length of beam longitudinal reinforcement. Crushing of core concrete may occur. Fracture or buckling of reinf. Requiring replacement may occur (FEMA, 2013).

Figure 4-24. Beam-column joint damage states (FEMA, 2013)

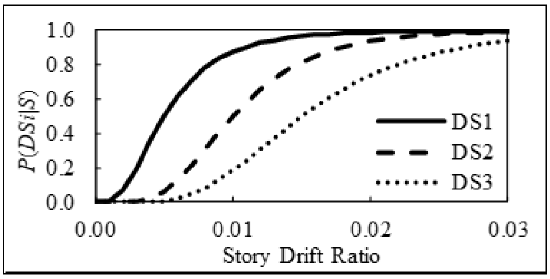
Table 4-6. Typical fragility functions parameters. Adapted from Yamin (2017).

FRAGILITY FUNCTIONS PARAMETERS	Date
GENERAL DATA	5/10/2015

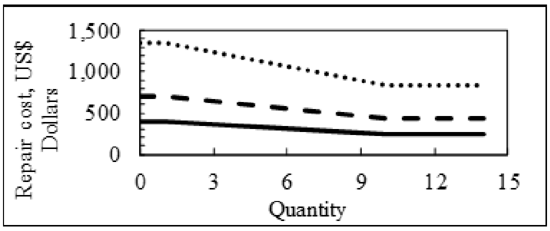
Code	C1011.004b
Description	Confined masonry partition wall isolated from the structure
# Damage States	3
Demand Parameter	Story Drift Ratio
Standard Unit	5m x 3m

Fragility Function Parameters			
Damage State	DS1	DS2	DS3
Median Demand, θ	0.005	0.01	0.015
Total Dispersion, β	0.60	0.45	0.45

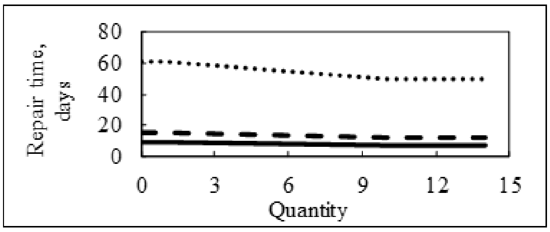
Damage Associated	
DS1	Minor cracking
DS2	Crack at joints and plaster
DS3	Partial collapse



COST MODEL			
Damage State	DS1	DS2	DS3
Lower Quantity (LQ)	1	1	1
Upper Quantity (UQ)	10	10	10
Cost LQ, US$ Dollars	404	694	1,350
Cost UQ, US$ Dollars	249	427	830
Best Fit	Normal	Lognormal	Lognormal



TIME MODEL			
Damage State	DS1	DS2	DS3
Lower Quantity (LQ)	1	1	1
Upper Quantity (UQ)	10	10	10
Time LQ, days	9.5	15.5	60.8
Time UQ, days	7.8	12.7	49.8
Best Fit	Normal	Lognormal	Lognormal



4.2.8.2.3 Repair Cost Integration

Using Monte Carlo simulation techniques, the integration of repair costs from all components in the model is performed considering all possible sources of uncertainty (for details, see Yamin et al. 2017). For each specific building typology, a sufficient number of realizations are used to obtain the total expected repair costs and its variance at each intensity level. Figure 4-25 summarizes the proposed procedure.

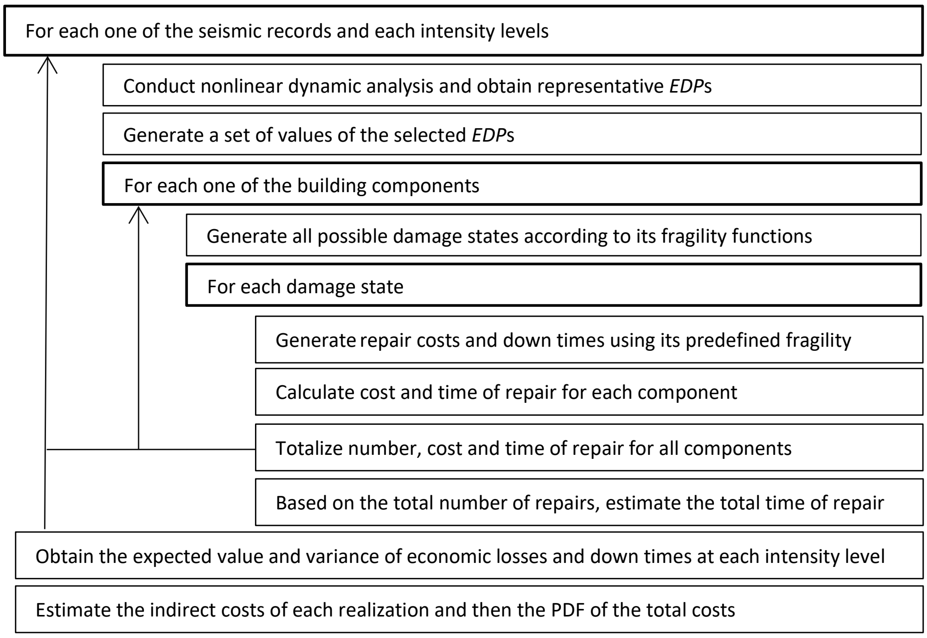


Figure 4-25. Monte Carlo simulation procedure to obtain total repair costs. Adapted from (Yamin et al., 2017)

Once the results are available for a sufficient number of realizations, the following considerations are included in the vulnerability and loss assessment procedure:

- Residual drift in order to consider the building irreparable.
- Excessive repair costs to consider a complete replacement.
- Minimum seismic intensity level for initial damages.
- Specific considerations for estimation of indirect costs such as business interruption, consideration of efficiency and scaled economy to estimate repair costs and times, maximum time frame for repairs; time spans required to initiate the repair works and to re-occupy the building once the repair works are finished and number of simultaneous laborer teams for structural and non-structural repair works; amongst others.

A software package has been developed to facilitate the calculation of vulnerability functions, given a collection of EDP resulting from the analysis at different intensity levels (**Annex J**: Software **IT-Funvul**

V2.0 (Yamin et al., 2017), available at www.ecapra.org). Figure 4-26 illustrates a typical vulnerability function obtained with the proposed methodological approach.

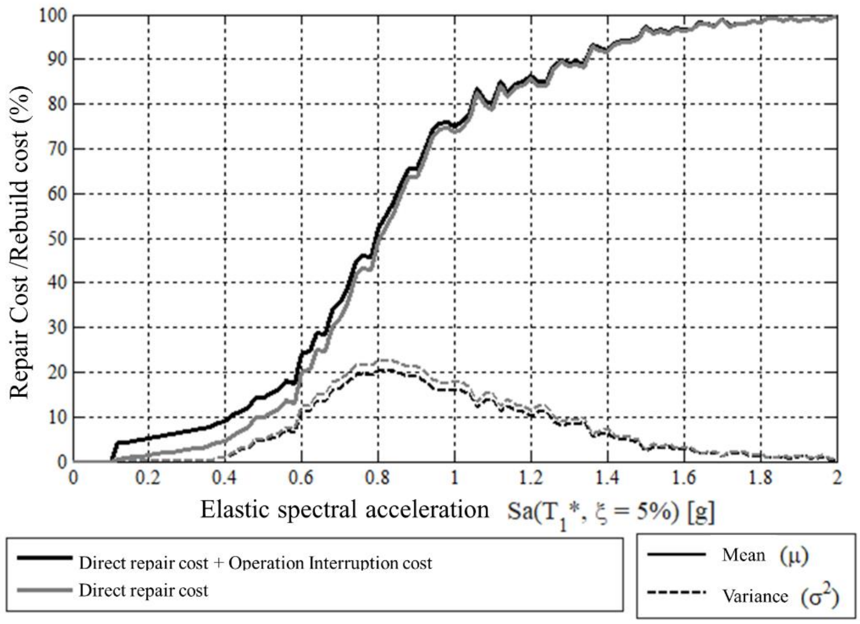


Figure 4-26. Example of vulnerability curve computed using IT-FUNVUL V2.0 (www.ecapra.org)

4.2.8.2.4 Advantages and limitations of the component-based vulnerability assessment method

The following are the advantages of the proposed vulnerability assessment method:

- It is a component-based damage assessment method and therefore it can consider simultaneous damages occurring in different structural and non-structural components.
- No subjective assessment is directly demanded by the method.
- Multiple EDPs can be used to define damage states for different component types.

The main limitations are:

- It requires the definition of information that may not be readily available.

4.3 Illustrative examples

This section presents the illustrative example for the application of the proposed F/V assessment methodology for one IB of LBM school construction type and one IB of RC school construction type, respectively. Results for all the IBs are documented in Chapter 3.

4.3.1 Example Analysis for an LBM Index Building

In this section an example application of the discussed methodology to an LBM IB is presented. To this aim, an UCM-URM7 IB is considered in this example for carrying out the fragility and vulnerability analysis following the steps described in the previous sections.

4.3.1.1 *Hazard Definition*

The group of far-field records discussed in Section 4.2.1 are selected for the analysis. Figure 4-27 presents the response spectra of the 22 ground motions.

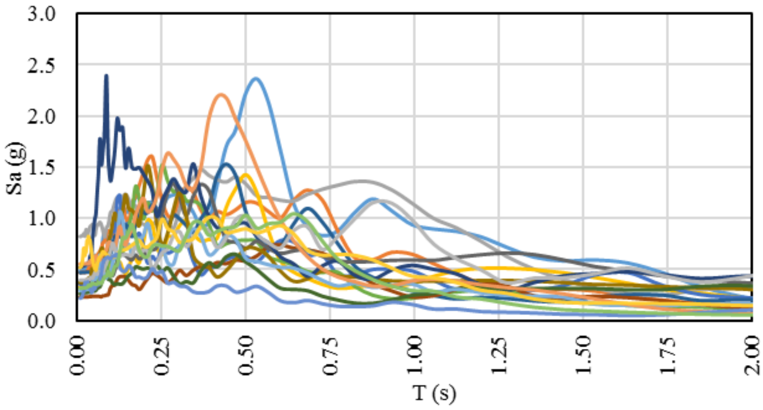


Figure 4-27. Response spectra of 22 far field ground motion suite.

4.3.1.2 *Index Building Definition*

Figure 4-28 shows representative photographs of the IB considered. A single story two-classroom rectangular plan school building is chosen for the analysis of an UCM-URM7 IB.

(a) (b)

Figure 4-28. Photographs representative of an IB of the UCM-URM7/LR(1)/LD school building type: (a) outside front view and (b) Inside view showing the flexible roof diaphragm. (Photo from Nepal, Copyright: The World Bank)

Table 4-7 presents the GLoSI taxonomy string for the selected UCM-URM7 IB.

Table 4-7. IB Taxonomy parameters.

Building Type	GLoSI Taxonomy String
UCM-URM7/LR/LD	**UCM-URM7/LR(1)/LD**/FD/NI/LP/LO/RF/NP/OS/PC/VN

4.3.1.3 *Numerical Modelling, Pushover Analysis and Seismic Behavior*

Figure 4-29 shows the element by element 3-dimensional numerical model developed in ELS using applied element method. Table 4-8 presents the geometrical characteristics of the building and Table 4-9 presents the average material properties for the UCM-URM7 construction in Nepal (Guragain, 2015). This model is subjected to an equivalent pushover analysis as explained previously in Section 4.2.4.

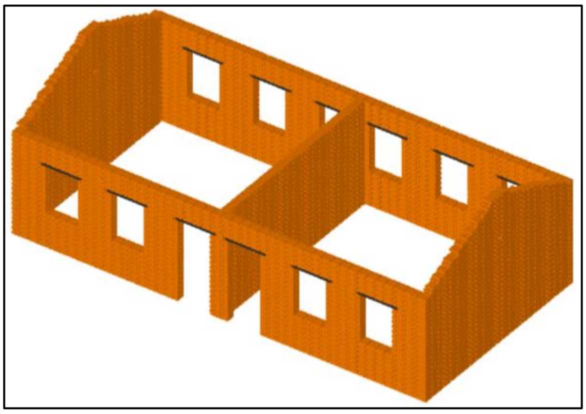


Figure 4-29. Numerical model of the UCM-URM7 IB in ELS using simplified micro-modelling technique.

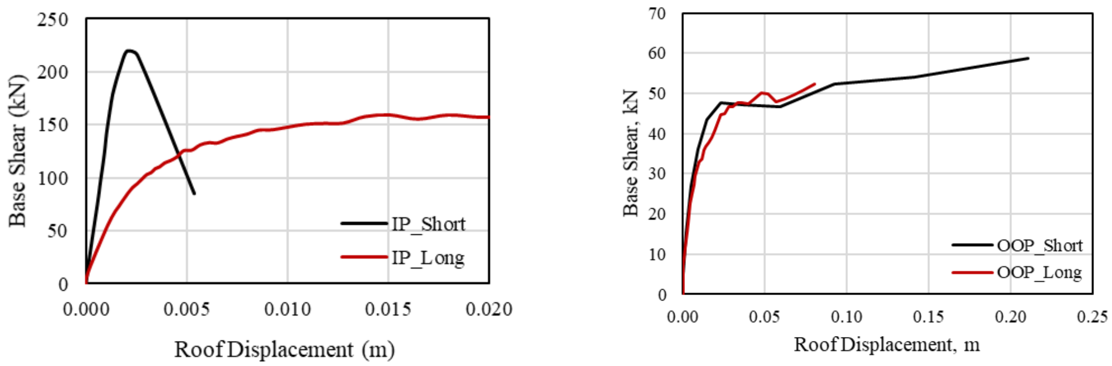
Table 4-8. Geometrical characteristics of the UCM-URM7 IB.

Characteristic	Value
Building plane area (m^2):	60
Building total area (m^2):	60
Number of stories:	1
Story height (m):	2.8
Number of spans in long direction:	2
Typical span length in long direction (m):	5.7
Number of spans in short direction (m):	1
Typical span length in short direction (m):	5.3
Wall Thickness (mm):	250
Wall Construction:	English Bond
Thickness:	One brick

Table 4-9. Elastic and non-linear material properties of masonry.

Masonry Material Properties	Average Value (unit)
Unit Wight	1920 kg/m^3
Modulus of Elasticity	263 MPa
Shear Modulus	158 MPa
Compressive Strength	4.14 MPa
Cohesion	0.17 MPa
Flexural Tensile Strength	0.069 MPa
Friction Coefficient	0.6

Figure 4-30 shows the pushover curves for the selected IB in two principal directions with respect to global IP and OOP behavior, respectively.

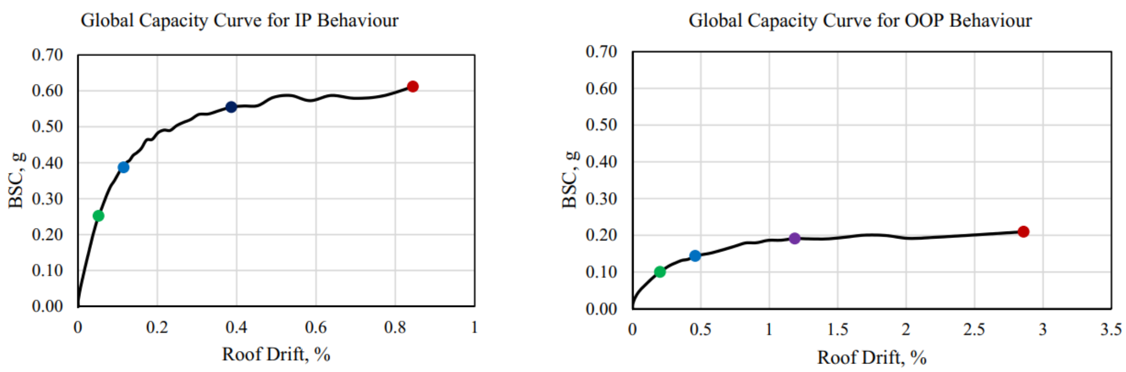


(a) (b)

Figure 4-30. Pushover curve comparison of the UCM-URM7 IB in long and short direction with respect to (a) global IP and (b) global OOP behavior.

As shown in Figure 4-30, it is clear that the building is weaker in the longitudinal direction in comparison to the transverse direction (under IP behavior, initial stiffness and peak strength both are higher in transverse direction; while under OOP behavior, although the initial stiffness and peak strength are comparable, the ductility is lower in the longitudinal direction). Thus, the F/V analysis is conducted in the longitudinal direction only.

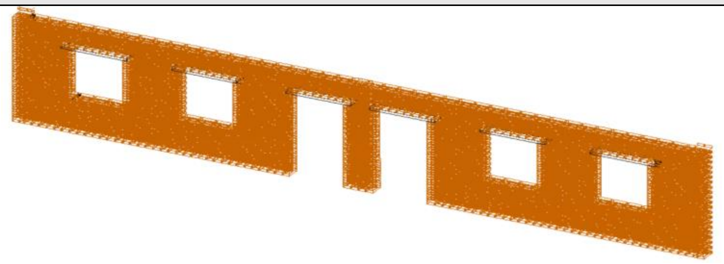
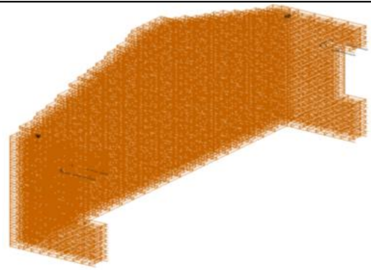
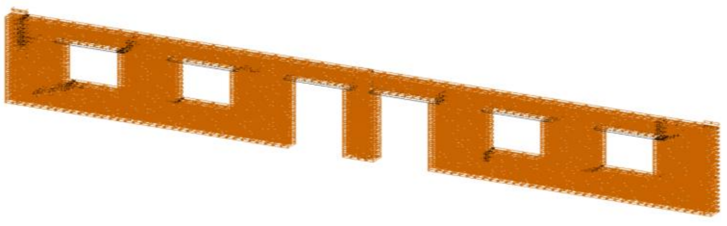
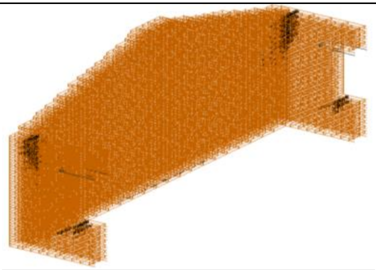
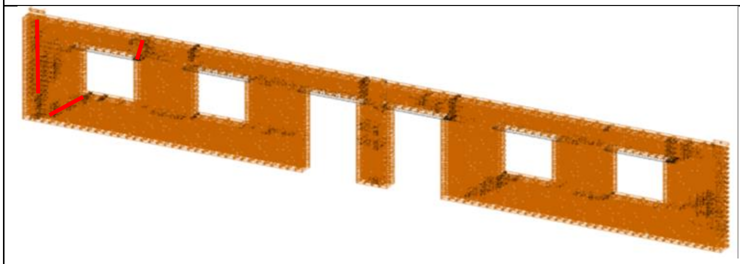
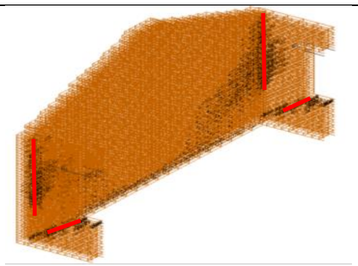
Figure 4-31 presents the global capacity curves with respect to global IP and global OOP behavior respectively for the selected IB in longitudinal direction. Also shown in the figures are the thresholds for different damage states. As explained in the methodology, for this flexible diaphragm type structure lacking global behavior, analysis with respect to IP and OOP behavior will be conducted.

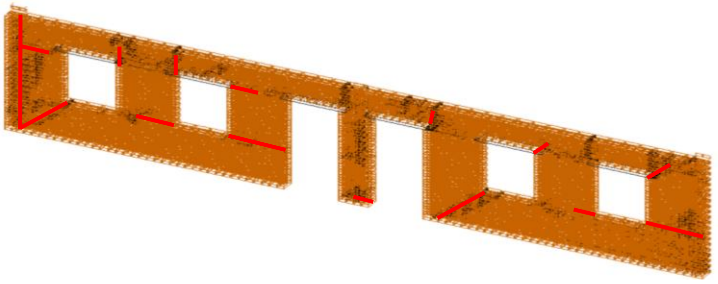
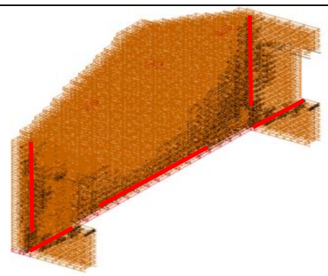


(a) (b)

Figure 4-31. Capacity curves and associated damage state thresholds for the UCM-URM7 IB: (a) global IP behavior and (b) global OOP behavior. (Damage state thresholds: green – DT1, blue – DT2, indigo – DT3 and red – DT4)

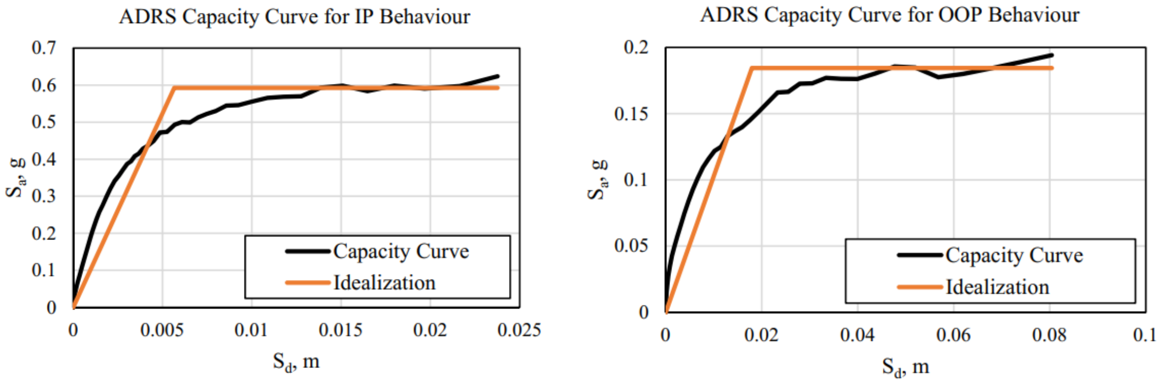
Table 4-10. Damage (Crack Pattern, Width and Extent) progression during seismic loading.

IP Behavior	OOP Behavior
	
OP Threshold: Hairline cracks (black) of maximum width 0.35 mm appeared at few corners of openings.	**OP Threshold:** Minor cracks (black) of 0.5 mm maximum width appeared at the connection with the in-plane wall.
	
IO Threshold: Hairline to minor cracks (black) of maximum width 1 mm developed at most of the corners of the openings, left most pier and spandrel start to develop shear and flexural cracks, respectively.	**IO Threshold:** Minor cracks (black) with maximum width of 3 mm started to extend downwards at the connection between IP walls, minor shear cracks (black) of 1 mm started in the IP walls.
	
LS Threshold: Left most pier has developed extensive shear crack (red) of 12.5 mm maximum width. The left most spandrel also develop an extensive flexural crack (red). Major shear cracks (red) of maximum width 10 mm as well as horizontal (flexural) cracks (red) with a maximum opening of 2 mm appear through most of piers.	**LS Threshold**: Full combined mechanism started with major cracks (red) of 12.5 mm maximum width at the IP walls connections through half of the wall height and shear cracks (red) of 12.5 mm width developed in IP walls. A minor horizontal crack at the bottom layer extended to full length, with maximum crack opening of 1 mm.

	
CP Threshold: Most piers and spandrels developed extensive shear cracks (of more than 12.5 mm maximum width) and flexural cracks (of 4 mm maximum crack opening) (red). The left most pier and spandrel are on the verge of collapse.	**CP Threshold**: The cracks (vertical, red) at the IP wall connection becomes extensive with maximum width of more than 12.5 mm and extends through full wall height. Extensive shear crack (diagonal, red) with a width of more than 12.5 mm developed in the IP walls. A horizontal crack extended through the wall with a maximum crack opening of 4 mm.

4.3.1.4 N2 Analysis

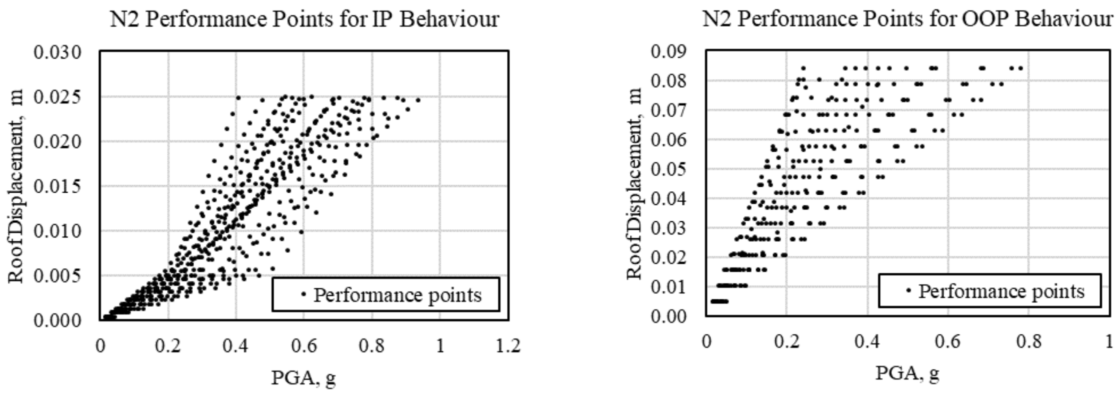
Figure 4-32 shows the bilinear idealization of the capacity curves for global IP and OOP behavior, respectively.



(a) (b)

Figure 4-32. Bilinear idealization of the capacity curves with respect to (a) IP behavior and (b) OOP behavior.

The performance point cloud (IM vs EDP) obtained for the IP and OOP behavior using the 22 set (each scaled) of ground motions are shown in Figure 4-33.

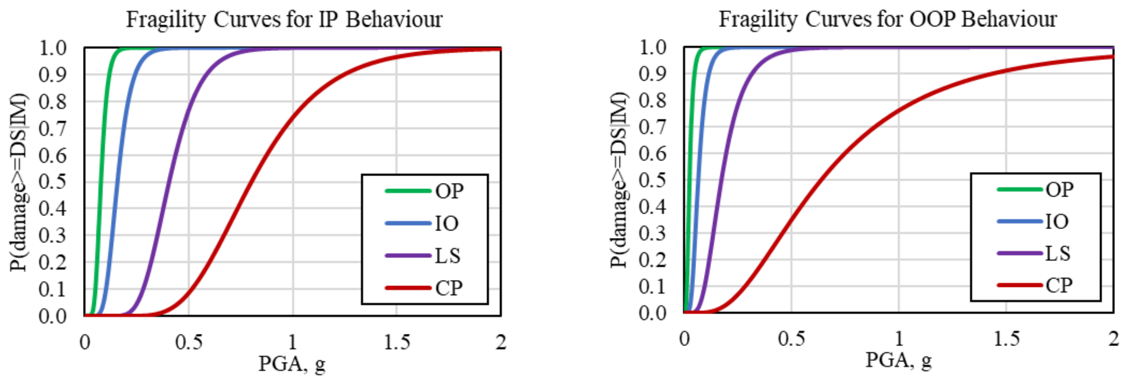


(a) (b)

Figure 4-33. Performance points (IM vs EDP) for OOP behavior (left) and IP behavior (right),

4.3.1.5 Fragility Analysis

Figure 4-34 present the fragility curves for each damage states computed using least squares methodology. A PGA of 2g is considered as the upper limit of the IM.



(a) (b)

Figure 4-34. Fragility curves for UCM-URM7 IB for a) global IP behavior and b) global OOP behavior.

4.3.1.6 Vulnerability Analysis

Figure 4-35 a) and b) shows the vulnerability functions with respect to global IP and global OOP behavior. Finally, Figure 4-35 c) presents the building total vulnerability curve obtained by combining the vulnerability curves with respect to global IP and global OOP behavior based on the contribution factor of walls under IP behavior and walls under OOP behavior (50% each in this case) to the total building vulnerability.

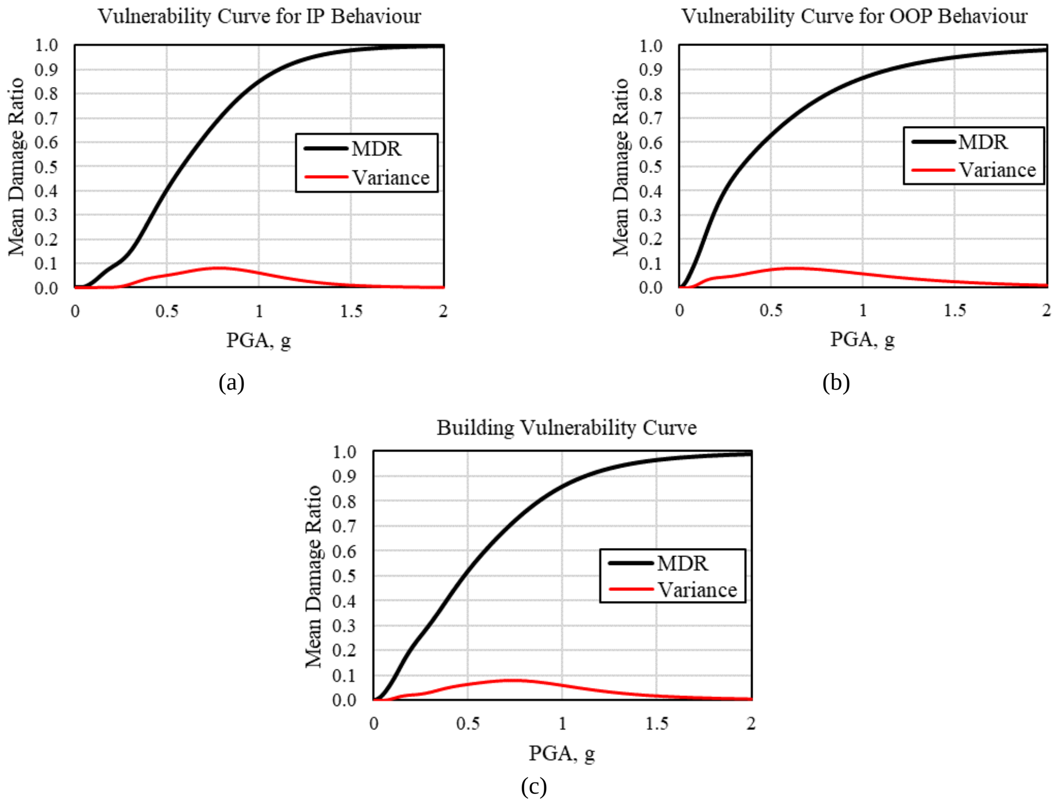


Figure 4-35. Vulnerability curves with respect to a) IP behavior, b) OOP behavior, and (c) building total vulnerability curve for the UCM-URM7 IB.

4.3.2 Example Analysis of an RC Index Building

4.3.2.1 Hazard Definition

The group of far-field records indicated in section 2.1.2 are selected for the analysis. Figure 4-36 presents seismic record response spectra from the proposed group.

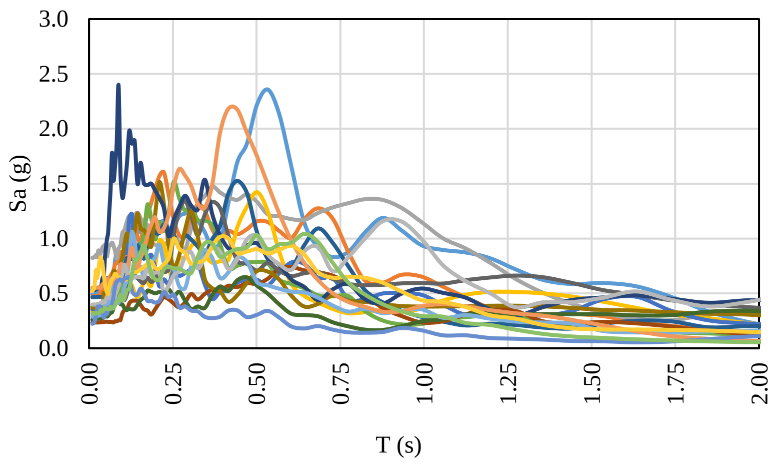


Figure 4-36. Far field ground motion response spectrum.

4.3.2.2 Index Building Definition

Table 4-11 presents and summarizes the main parameters of the IB selected for illustration purposes.

Table 4-11. IB taxonomy parameter

Building Type	GLoSI Taxonomy String
RC1/MR/PD	**RC3/MR/LD/RD/NI/SS/SW/RF/NP/OS/GC/VN**

Table 4-12 shows geometry intrinsic characteristics for the Index Building in consideration.

Table 4-12. Geometry intrinsic characteristics

Characteristic	Value
Building plane area (m^2):	299.25
Building total area (m^2):	598.5
Number of stories:	2
Story height (m):	3
Number of spans in X direction:	7
Typical span length in X direction (m):	4.5
Number of spans in Y direction (m):	3
Typical span length in Y direction (m):	3.5
Foundation system:	CISF
Typical column dimensions (cm x cm):	25X25
Typical beam dimensions (cm x cm):	20X30
Typical shear wall dimensions (cm x cm):	-
Typical bracing member section (cm x cm):	-

Table 4-13 presents material properties used in modelling. It includes the concrete, the reinforcement steel and the masonry in the infills.

Table 4-13. Material properties

Concrete	f'c (MPa):	17	Ec (GPa):	19
Reinforcement	fy (Mpa):	420	Es (GPa):	200
Masonry	f'm (MPa):	8	γ:………..	-

4.3.2.3 Numerical Modelling and Pushover Analysis

The modelling was made following ASCE 41-17 recommendations for bare frame structures, infill walls are not considered in this IB. Table 4-14 shows the modelling considerations, the loads assigned and the analysis considerations for this specific example.

Table 4-14. Modelling considerations

Modelling considerations:				
Plasticity model:	Lumped			
Infill walls modelling approach:	Equivalent frame			
Roof Diaphragm:	Rigid			
Foundation:	Rigid			
Loads:				
Over imposed design dead load (D) (kN/m2):	1.2			
Design Live load (L) (kN/m2):	2.0			
Load combination in non-linear analysis:	D+0.25L			
Average load per square meter (kN/m2):	8.7			
Analysis considerations:				
Global P-Delta effects:	Yes			
Rigid zones:	Yes			
Initial effective stiffness:	Beams	0.35	Columns	0.30
Analysis direction:	X			
Analysis orientation:	(+)X			

Figure 4-37 shows the mathematical model developed for the structural analysis.

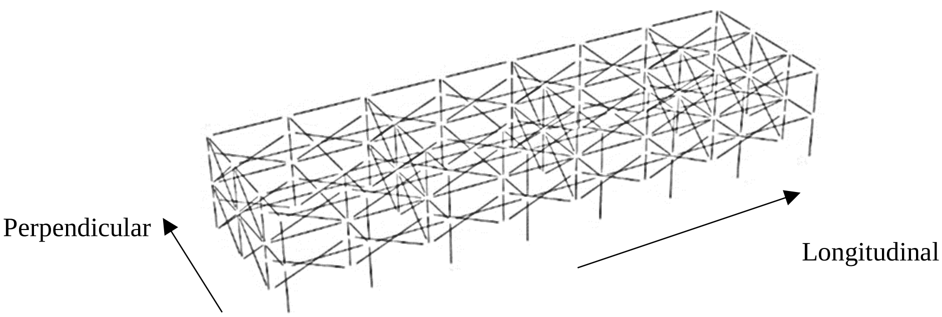


Figure 4-37. RC IB structural model

Figure 4-38 presents the pushover curve for the selected IB.

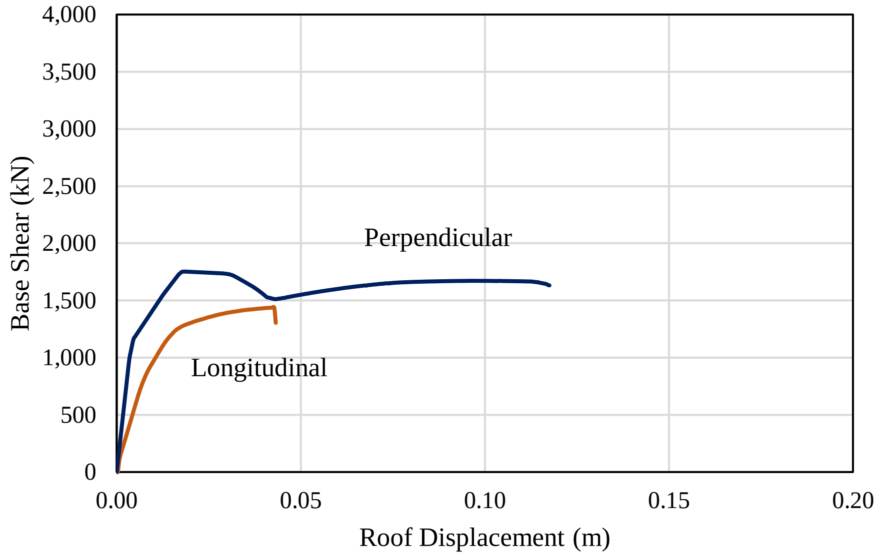


Figure 4-38. RC IB pushover curve

4.3.2.4 N2 Analysis

Figure 4-39 present the EDPs obtained using N2 non-linear static methodology.

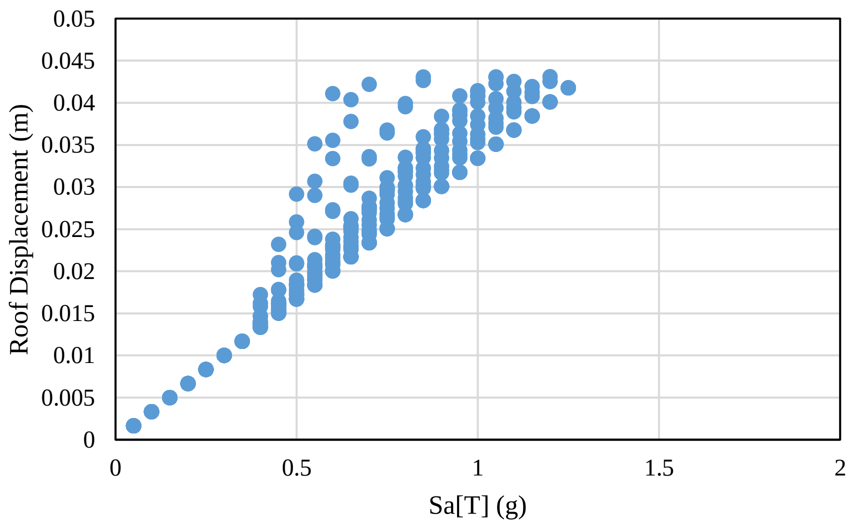


Figure 4-39. RC IB EDPs

4.3.2.5 Fragility Analysis

Figure 4-40 present the fragility curves for each damage states computed using least squares methodology as explained in 2.9.

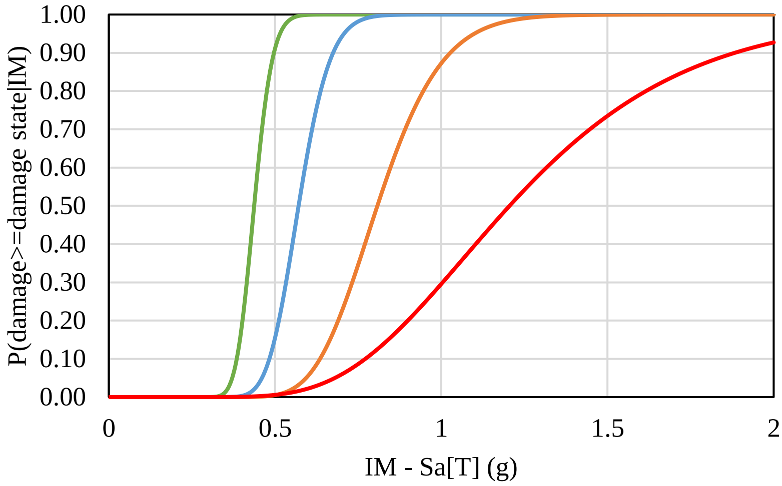


Figure 4-40. RC IB fragility function

4.3.2.6 Vulnerability Analysis

The component model used is shown in Table 4-15.

Table 4-15. Component model

Story	Group	Subgroup	Description	Unit	Quantity	Fragility curve	EDP	Correlation
1	E	C1	Column-one beam	Node	8	B1041.091a	Drift	0
1	E	C2	Column-two beams	Node	21	B1041.091b	Drift	0
1	A	F2	Masonry facade	5m x 3m	14	C1011.006a	Drift	1
1	A	M4	Masonry wall	5m x 3m	6	C1011.006b	Drift	1
1	C	S2	Contents	5m x 5m	13	E2022.010a	Drift	0
2	E	C1	Column-one beam	Node	8	B1041.091a	Drift	0
2	E	C2	Column-two beams	Node	21	B1041.091b	Drift	0
2	A	F2	Masonry facade	5m x 3m	14	C1011.006a	Drift	1
2	A	M4	Masonry wall	5m x 3m	6	C1011.006b	Drift	1
2	C	S2	Contents	5m x 5m	13	E2022.010a	Drift	0

Table 4-16 shows FUNVUL phase's parameters used in this example.

Table 4-16. FUNVUL calculation parameters

Phase I:

Beta model uncertainty:			0.3	
Number of iterations for model uncertainty:			15	
Number of iterations for damage states uncertainty:			15	
Number of iterations for cost and time uncertainty:			15	
Scale factor for cost:	Yes		No	X

Phase II:

Lower intensity to no damage (g/g):	0.1
Maximum allowable residual drift for demolition (%):	1.5
Percentage of building replacement value (%):	100
Bidirectional factor for total cost model:	1
Intensity level for building evacuation (g/g):	2

Figure 4-41 shows the vulnerability function obtained using FUNVUL methodology as explained above.

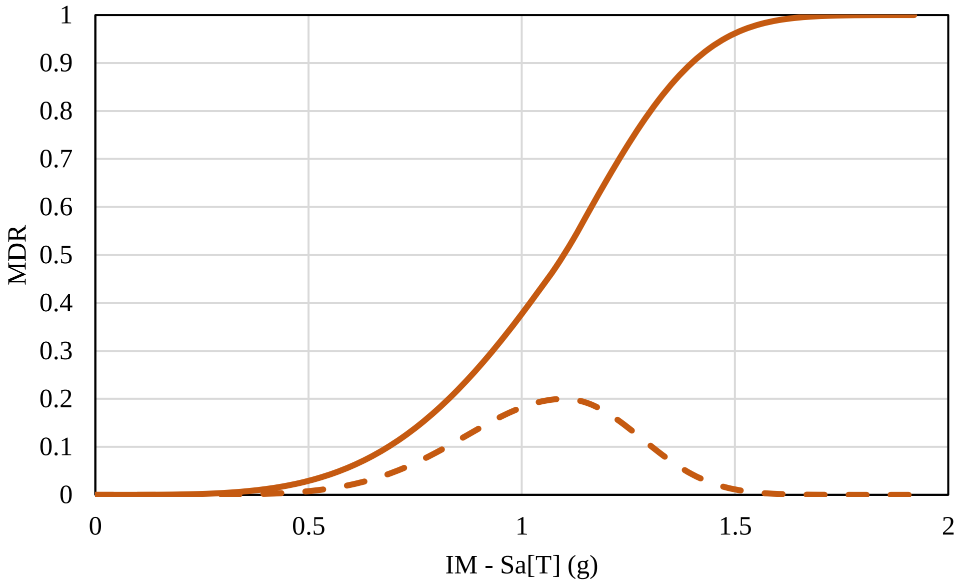


Figure 4-41. RC IB Vulnerability function

4.4 Catalog of Fragility/Vulnerability Assessment Results

In order to organize, use and disseminate the final information obtained in relation to the F/V assessment, a special form has been designed. Figure 4-42 and Figure 4-43 presents two different illustrative forms, one for LBM and one for a RC building. **Annex K** summarizes the F/V forms for all IBs analyzed.

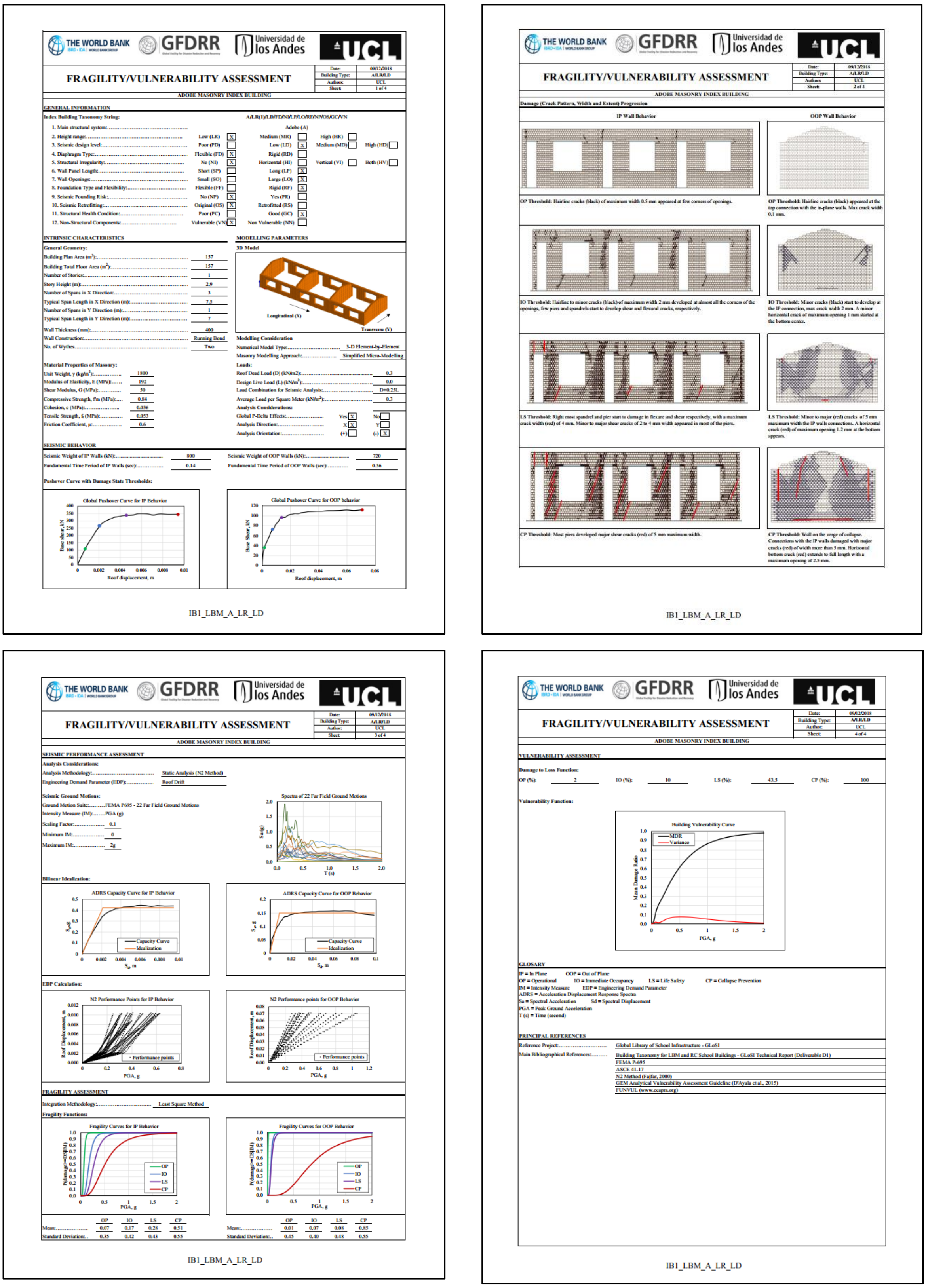


Figure 4-42. Example F/V Assessment Form for an LBM IB.

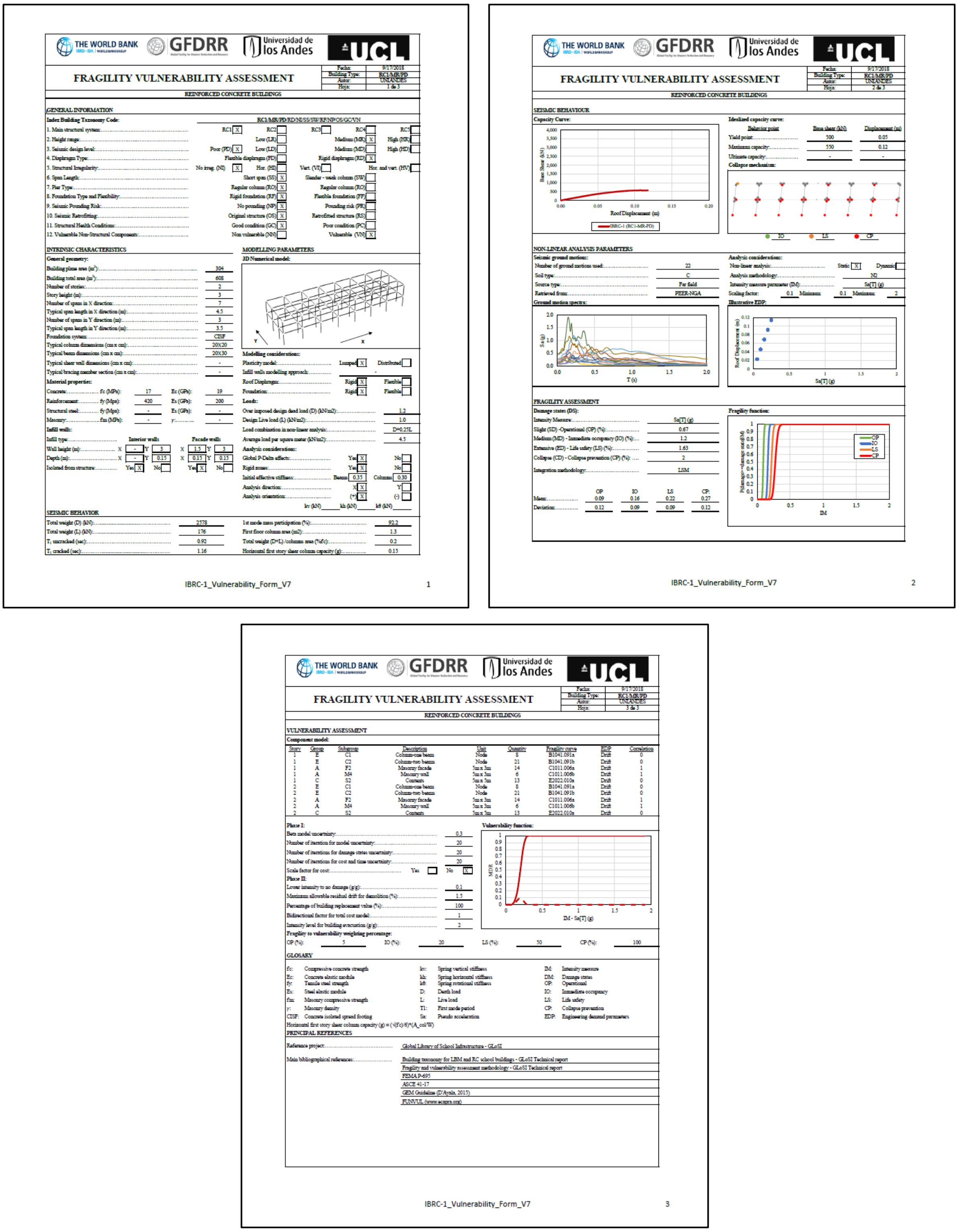


Figure 4-43. Example F/V Assessment Form for an RC IB.

4.5 Sensitivity Analysis

The different construction characteristics of a building, i.e. the vulnerability parameters, pose a considerable amount of uncertainties, from actual material properties of masonry components, to variation in geometry and layout. As the fragility and vulnerability functions derived in this document are expected to be of international and global applicability, it is of great importance to quantify the variability of these parameters in practice, and hence the uncertainty associated with any of the functions derived, and its remit of applicability to a given taxonomy class.
To this end, sensitivity analyses are conducted for one LBM and one RC building type to understand and quantify the effect of the different vulnerability parameters and their associated attributes, on the seismic performance and seismic vulnerability of the corresponding construction type.

4.5.1 Load Bearing Masonry

A sensitivity analysis is conducted for the UCM-URM7/LR building type to understand and quantify the effect of the different vulnerability parameters and their associated attributes, on the seismic performance and seismic vulnerability. It should be noted that in most of the cases, only a single relevant parameter is changed at a time, keeping all the others constant (this method of sensitivity analysis is known as one-at-a-time (OAT) method (e.g. Pannell, 1997).
The detailed information on the baseline model (UCM-URM7/LR/LD) is provided in the illustrative example section (section 4.3.1) and the full taxonomy string is repeated in Table 4-17. This building type represents an UCM-URM7 school construction typical to Nepal. As the building is weaker in its longer direction, the seismic analysis for all different models is carried out in this direction unless otherwise specified.

Table 4-17. Baseline model taxonomy parameters.

Building Type	GLoSI Taxonomy String
UCM-URM7/LR/LD	**UCM-URM7/LR(1)/LD**/FD/NI/LP/LO/RF/NP/OS/PC/VN

Table 4-18 gives the details of the parameters used in the sensitivity analysis. The attributes highlighted in red represent the attributes of the sensitivity parameter being considered.

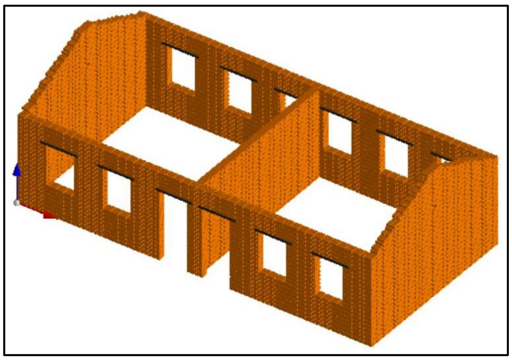
Table 4-18. Sensitivity parameters, attributes and associated taxonomy strings.

S.N.	Parameters	Range (Attributes)	Taxonomy String	Expected Range Covered
1	Seismic Design Level	Poor Design (PD)	**UCM-URM7/LR(1)/PD**/FD/NI/LP/LO/RF/NP/OS/PC/VN	Yes
		Low Design (LD)	**UCM-URM7/LR(1)/LD**/FD/NI/LP/LO/RF/NP/OS/PC/VN	
		Medium Design (MD)	**UCM-URM7/LR(1)/MD**/FD/NI/LP/LO/RF/NP/OS/PC/VN	
		High Design (HD)	**UCM-URM7/LR(1)/HD**/FD/NI/LP/LO/RF/NP/OS/PC/NN	
2	Diaphragm Type	Flexible Diaphragm (FD)	**UCM-URM7/LR(1)/LD**/FD/NI/LP/LO/RF/NP/OS/PC/VN	Yes
		Rigid Diaphragm (RD)	**UCM-URM7/LR(1)/LD**/RD/NI/LP/LO/RF/NP/OS/PC/NN	
3	Irregularity	No Irregularity (NI)	**UCM-URM7/LR(1)/LD**/FD/NI/LP/LO/RF/NP/OS/PC/VN	No (Vertical and Combined Irregularities not covered)
		Horizontal Irregularity (HI)	**UCM-URM7/LR(1)/LD**/FD/HI/LP/LO/RF/NP/OS/PC/VN	
4	Wall Panel Length	Long Panel (LP)	**UCM-URM7/LR(1)/LD**/FD/HI/LP/LO/RF/NP/OS/PC/VN	Yes
		Short Panel (SP)	**UCM-URM7/LR(1)/LD**/FD/HI/SP/LO/RF/NP/OS/PC/VN	
5	Wall Opening	Large Openings (LO)	**UCM-URM7/LR(1)/LD**/FD/HI/SP/LO/RF/NP/OS/PC/VN	Yes
		Small Openings (SO)	**UCM-URM7/LR(1)/LD**/FD/HI/SP/SO/RF/NP/OS/PC/VN	

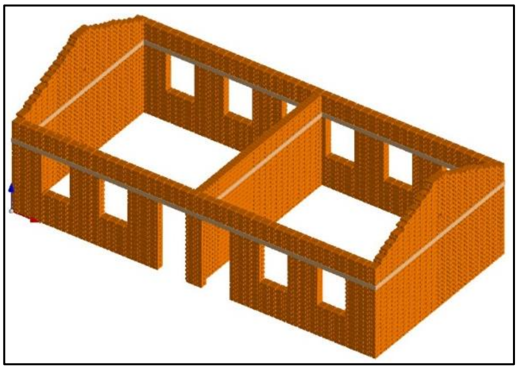
6	Effective Seismic Retrofitting	Original Structure (OS)	**UCM-URM7/LR(1)/LD**/FD/HI/SP/SO/RF/NP/OS/PC/VN	Yes
		Retrofitted Structure (RS)	**UCM-URM7/LR(1)/LD**/FD/HI/SP/SO/RF/NP/RS/PC/NN	
7	Structural Health Condition	Poor Condition (PC)	**UCM-URM7/LR(1)/LD**/FD/HI/SP/SO/RF/NP/OS/PC/VN	Yes
		Good Condition (GC)	**UCM-URM7/LR(1)/LD**/FD/HI/SP/SO/RF/NP/OS/GC/VN	

4.5.1.1 Seismic Design Level

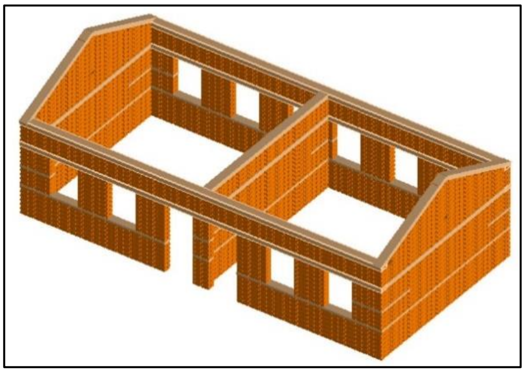
Seismic design level is an important parameter that highly influences the seismic behavior of a building (see section 2.5.3 for more details on the seismic design level). Four different seismic design levels are considered viz. Poor Design (PD), Low Design (LD), Medium Design (MD) and High Design (HD). The PD model represents a masonry building that has poor material quality and poor connection between the orthogonal walls. The LD model represents a building that has poor material qualities, but the orthogonal walls are well connected in English bond pattern. The MD model represents the buildings that have good material quality and a lintel level band beam to improve the global building behavior. Finally, the HD model represents the buildings that are relatively new (e.g. built in the school reconstruction program after 2015 Nepal earthquake) with good material quality and have sufficient number of seismic enhancement measures such as the sill level band, lintel level band, roof level band and intermediate ties in the walls. Figure 4-44 shows the numerical models for UCM-URM7 typology with different seismic design levels.



UCM-URM7/LR/PD and UCM-URM7/LR/LD



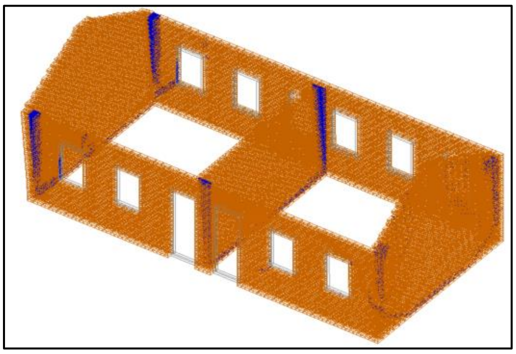
UCM-URM7/LR/MD



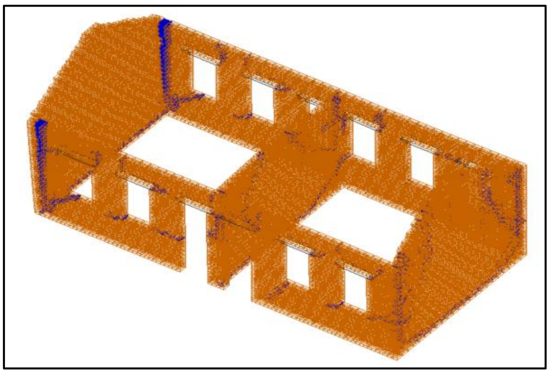
UCM-URM7/LR/MD

Figure 4-44. Numerical models of UCM-URM7 index buildings with different seismic design levels.

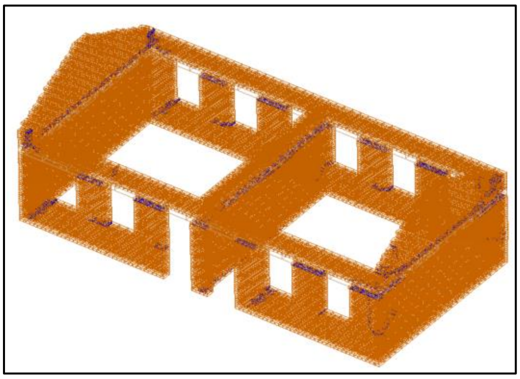
Figure 4-45 shows the final collapse mechanisms of each different index buildings. The OOP walls are highly vulnerable and tend to detach (from IP walls) and overturn in case of PD and LD models. On the other hand, in case of MD, the global seismic behavior is improved due to the box-like behavior provided by the lintel band beam (which binds all the walls together). However, in the MD model, the gables are not confined and hence are highly vulnerable as can overturn easily. In the case of HD, the behavior is highly improved mitigating the local failure modes of OOP walls. In both the MD and HD model, the global collapse is due to the shear failure of piers in the IP walls.



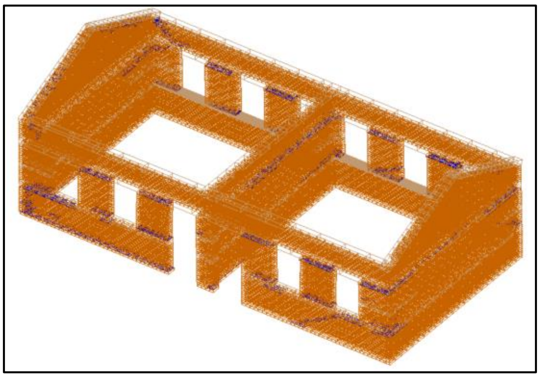
UCM-URM7/LR/PD



UCM-URM7/LR/LD



UCM-URM7/LR/MD



UCM-URM7/LR/HD

Figure 4-45. Collapse mechanisms of UCM-URM7 index buildings with different seismic design levels. The blue lines represent the extensive cracks of width more than 12.5 mm (only extensive cracks are shown).

Figure 4-46 shows the capacity curves along with the different damage state thresholds marked along the capacity curves. For the models which do not have global behavior (PD and LD), the capacity curves under IP and OOP behavior are plotted separately. It can be seen from the capacity curves that both the strength and displacement capacity is increased with the increase in seismic design level.

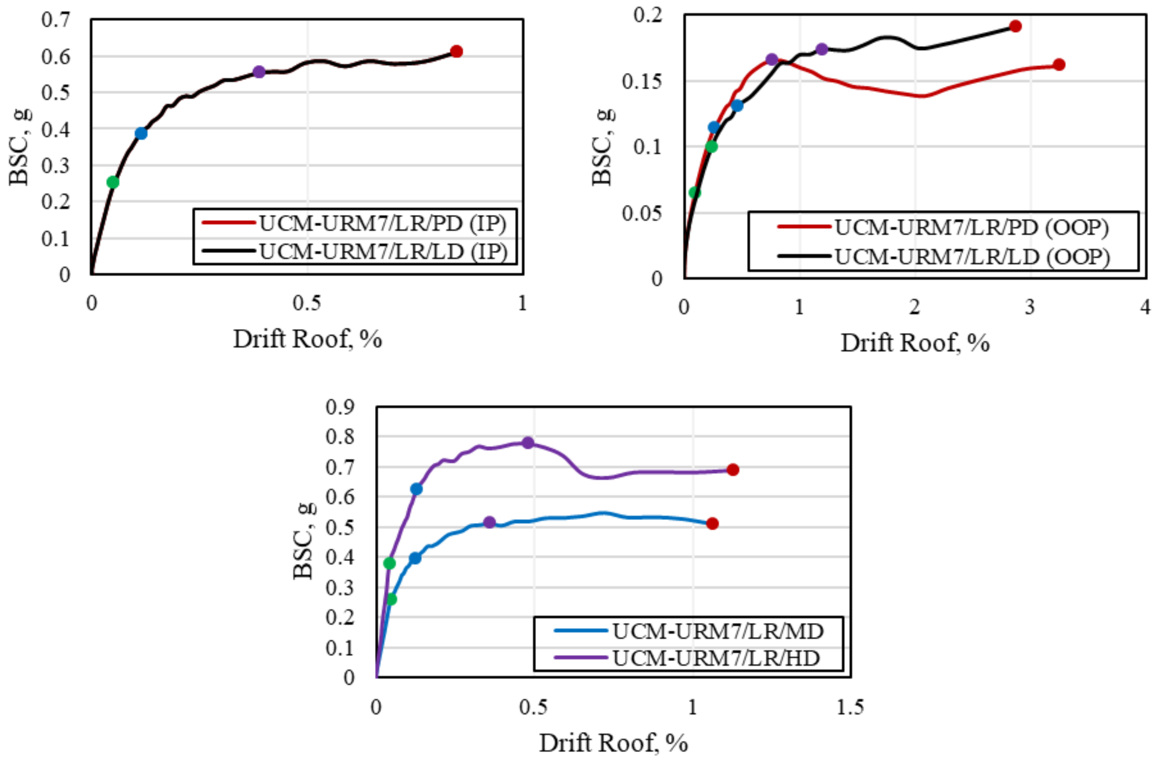


Figure 4-46. Comparison of capacity curves for the index buildings with different seismic design levels. The colored dots represent the threshold of different damage states: Green = Slight Damage, Blue = Moderate Damage, Purple = Extensive Damage and Red = Collapse.

Figure 4-47 shows the vulnerability curves for the index buildings with different seismic design levels. It can be seen that even the introduction of the lintel band beam only (MD) highly reduces the seismic vulnerability of the UCM-URM7/LR buildings. This is due to the restriction of OOP walls failure and the improvement of the global behavior. Vulnerability is further reduced when more seismic enhancements are introduced (in case of HD).

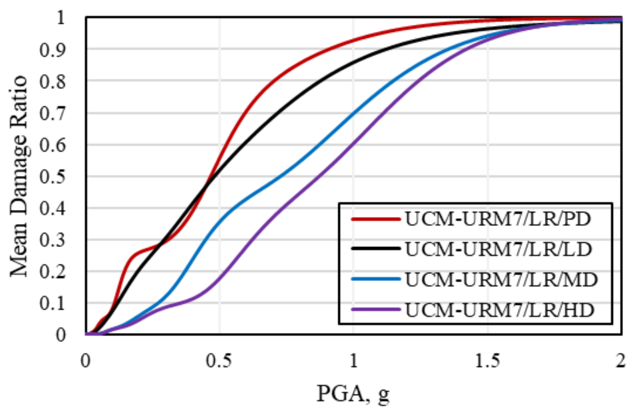
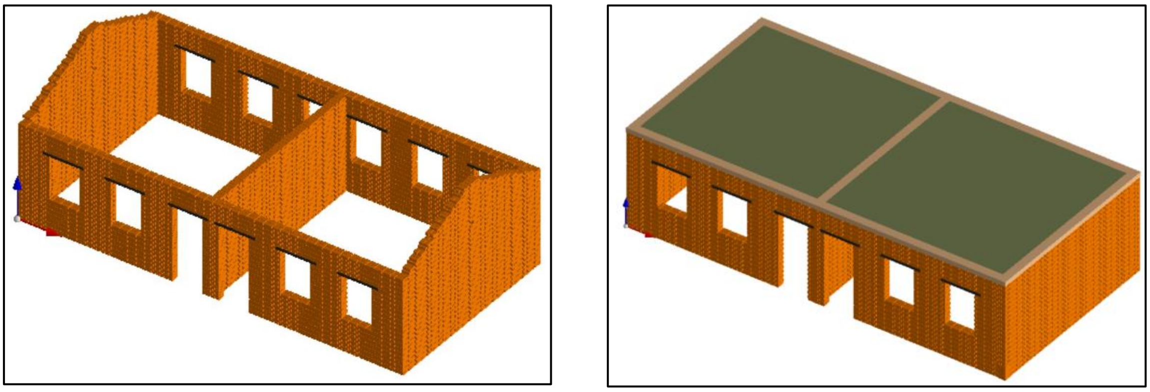


Figure 4-47. Comparison of vulnerability curves for the index buildings with different seismic design levels.

For a single-story UCM-URM7 building with same geometrical characteristics, the results show that the seismic behavior and failure mode as well as the vulnerability curves are unique to each different seismic design levels which supports the appropriateness of having four different attributes (i.e. poor, low, medium and high design levels) for the seismic design level.

4.5.1.2 Diaphragm Type

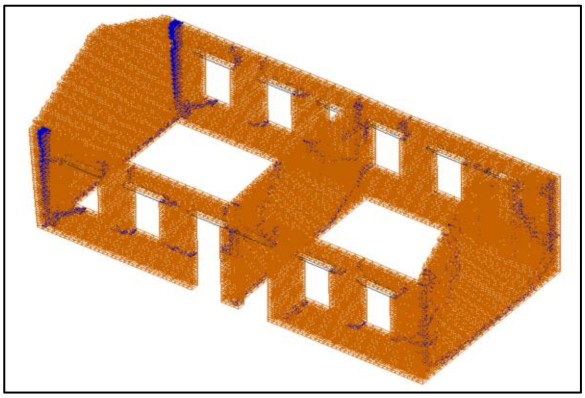
Roof and floor diaphragm action is also a critically important parameter that influences the seismic performance. When all the walls are well connected at the roof/floor level by a stiff structure such as an RC slab, along with a ring beam properly connected to the masonry walls, box-like behavior is obtained thereby improving the global seismic performance, controlling in particular early OOP wall failure.
Figure 4-48 shows the numerical models of the index buildings with flexible diaphragm and rigid diaphragm. The flexible diaphragm (FD) model consists of a light roof frame structure (not modelled i.e. the stiffness of the roof structure is neglected) while the rigid diaphragm (RD) type model consists of a ring beam and an RC slab. In this study, the depth of both the slab and the ring beams is 150 mm and the slab is provided as a flat structure without gables as this is the usual construction practice in many developing countries (e.g. Nepal) for a building structure with RC slab.



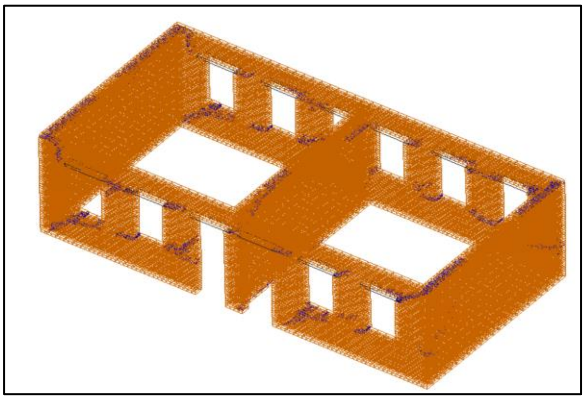
UCM-URM7/LR/FD UCM-URM7/LR/RD

Figure 4-48. Numerical models of UCM-URM7 index buildings with flexible and rigid diaphragm.

Figure 4-49 shows the collapse mechanisms of the two index buildings, it can be seen that the OOP failure mechanisms are prevented in the RD model and the final collapse mechanism is formed due to the shear failure of IP wall piers.



UCM-URM7/LR/LD/FD



UCM-URM7/LR/LD/RD

Figure 4-49. Collapse mechanisms of UCM-URM7 index buildings with different diaphragm types. The blue lines represent the extensive cracks of width more than 12.5 mm (only extensive cracks are shown).

Figure 4-50 shows the capacity curves along with the different damage state thresholds marked along the capacity curves. It can be seen from the capacity curves that both the strength (with respect to the OOP wall of FD model) and displacement capacity (with respect to the IP wall of FD model) is improved when the diaphragm action is rigid.

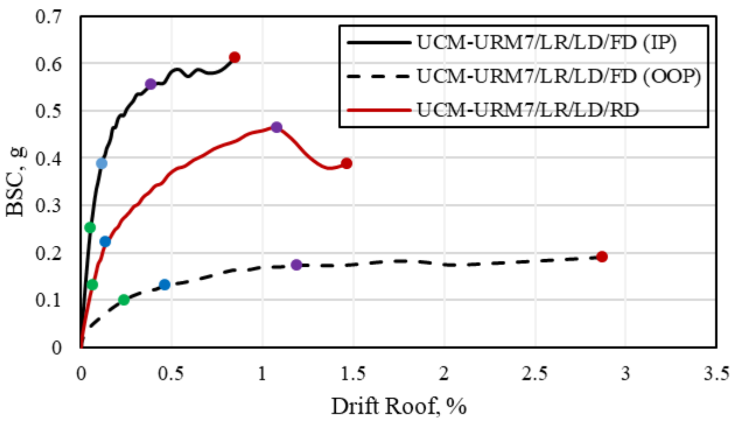


Figure 4-50. Comparison of capacity curves for the index buildings with different diaphragm types. The colored dots represent the threshold of different damage states: Green = Slight Damage, Blue = Moderate Damage, Purple = Extensive Damage and Red = Collapse.

Figure 4-51 shows the vulnerability curves for the index buildings with different diaphragm types. Although the vulnerability reduction at lower IM is not significant, there is considerable reduction in higher IM range. This is due to the prevention. of OOP walls failure and the improvement of the global behavior.

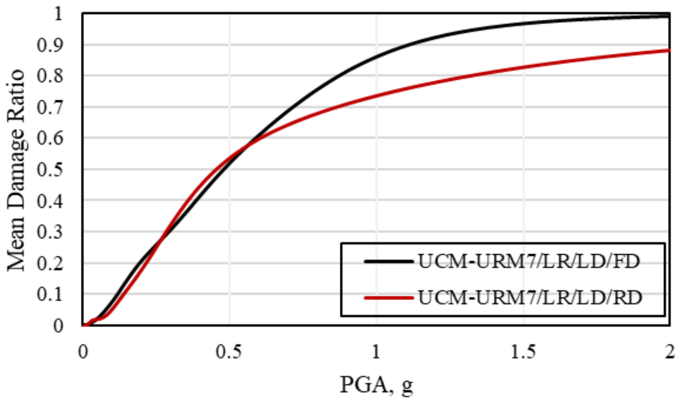
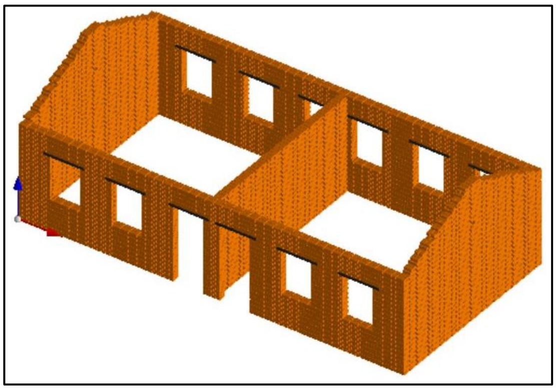


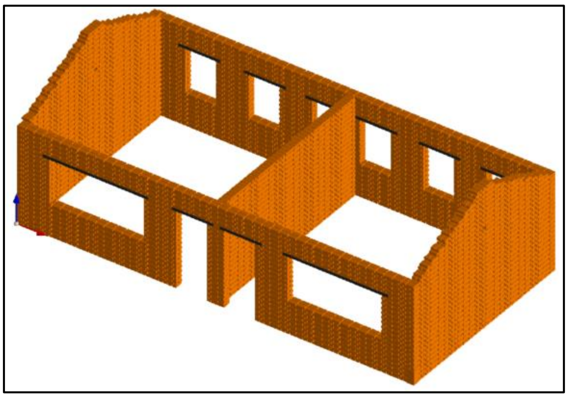
Figure 4-51. Comparison of vulnerability curves for the index buildings with different diaphragm types.

4.5.1.3 Irregularities

Different types of horizontal and vertical irregularities can be present in a school building. It is difficult and unrealistic to introduce much irregularity in the case study building without changing the plan dimensions. Here a horizontal irregularity imposed by the openings (size, location and distribution) is considered. The % opening in the front wall is increased (to 65%) compared to the 46% in the back wall. The opening irregularity can also be introduced when there are no openings at all in the back wall (solid wall) or when window openings are introduced in the shorter walls. But these later cases are not very common in real school buildings and hence are not studied in the present study. Figure 4-52 shows the numerical models of UCM-URM7/LR building with no irregularity and horizontal irregularity.



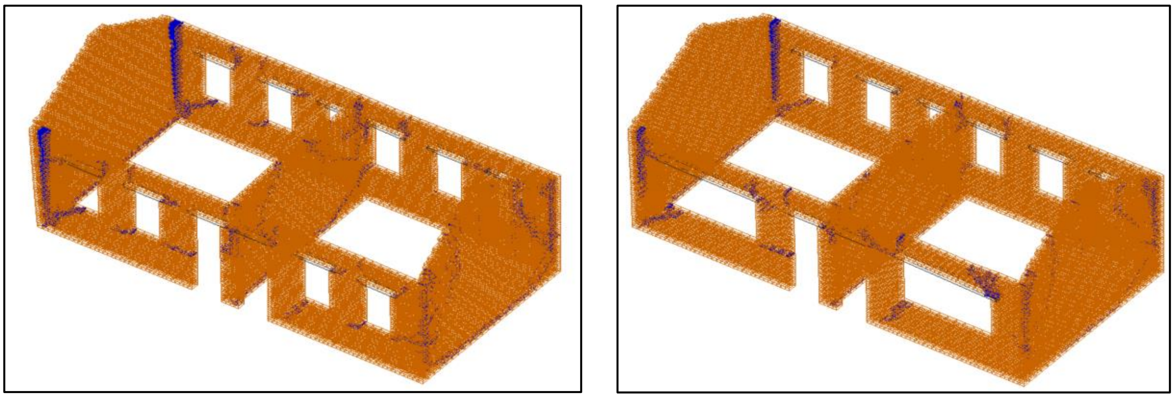
UCM-URM7/LR/LD/-/NI



UCM-URM7/LR/LD/-/HI

Figure 4-52. Numerical models of UCM-URM7 index buildings with no irregularity and horizontal irregularity (imposed by the openings). /-/ in the taxonomy string represents that some parameters are truncated in the string.

Figure 4-53 shows the collapse mechanisms of the two (NI and HI) index buildings. It is noticeable that in case of HI model, the weaker wall (front IP wall) is subjected to more damage than the stronger back IP wall.



UCM-URM7/LR/LD/-/NI UCM-URM7/LR/LD/-/HI

Figure 4-53. Collapse mechanisms of UCM-URM7 index buildings with no irregularity and horizontal irregularity. The blue lines represent the extensive cracks of width more than 12.5 mm (only extensive cracks are shown).

Figure 4-54 shows the capacity curves along with the different damage state thresholds marked along the capacity curves. In the case of HI model, there is noticeable reduction in both the strength and the displacement capacity.

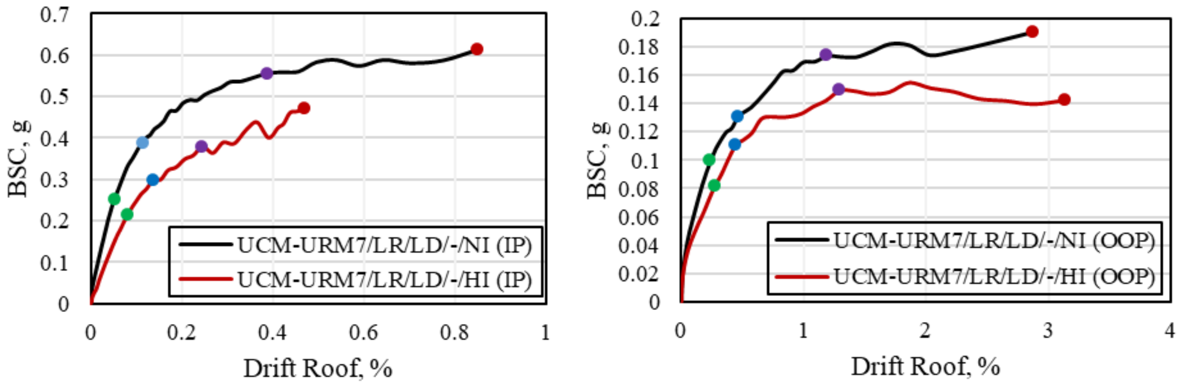


Figure 4-54. Comparison of capacity curves for the index buildings with no irregularity and horizontal irregularity. The colored dots represent the threshold of different damage states: Green = Slight Damage, Blue = Moderate Damage, Purple = Extensive Damage and Red = Collapse.

Figure 4-55 shows the vulnerability curves for the index buildings with no irregularity and horizontal irregularity. There is a modest increase in the seismic vulnerability due to the presence of horizontal irregularity in this case. However, other types of horizontal irregularities (such as plan shape irregularities) can significantly increase the vulnerability.

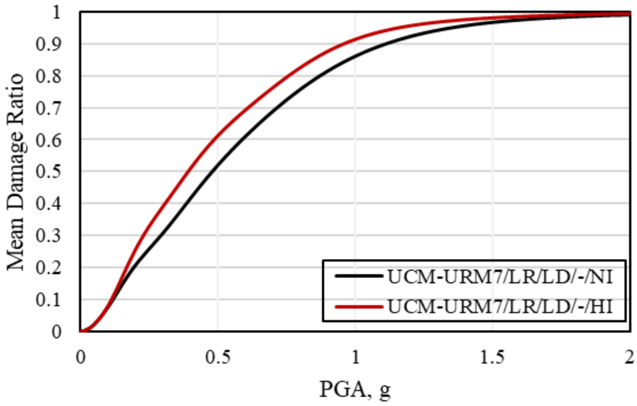


Figure 4-55. Comparison of vulnerability curves for the index buildings with no irregularity and horizontal irregularity.

4.5.1.4 Wall Panel Length

The vulnerability of a building increases with the increase in unrestrained length of a wall panel, mainly under OOP seismic loading. Here three different models are considered with different unrestrained panel lengths of the long walls, i.e. short panel length (SP) model, long panel length (LP) model and very long panel length (VLP) model. SP model has an unrestrained wall panel length of 3 m which is less than 12 times the wall thickness while the LP and VLP models have unrestrained wall panel lengths of 5.7 m and 10 m, respectively, both of which are larger than 12 times the wall thickness. Figure 4-56 shows the numerical models of the index buildings with different wall panel lengths.

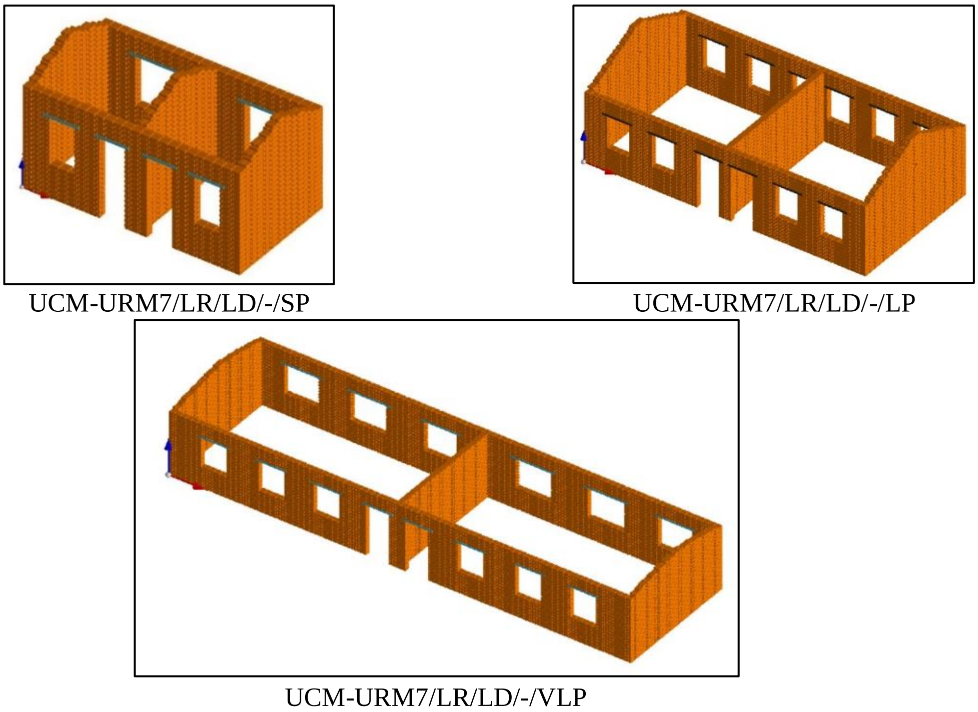


Figure 4-56. Numerical models of UCM-URM7 index buildings with different lengths of unrestrained wall panel.

When the wall panels are very large (in case of VLP model), the analysis should be carried out in both directions, as the longitudinal walls might become more vulnerable to OOP failure than the gable walls (short walls). Figure 4-57 shows the comparison of capacity curves for OOP walls when loaded in longitudinal and transverse direction. It can be seen that although the initial stiffness of the OOP capacity curve associated to the transverse (short) direction loading is higher, the ultimate strength and ultimate drift are both lower in comparison to the OOP capacity curve associated with the longitudinal loading. Hence the building is weaker in the OOP failure when loaded in shorter direction.

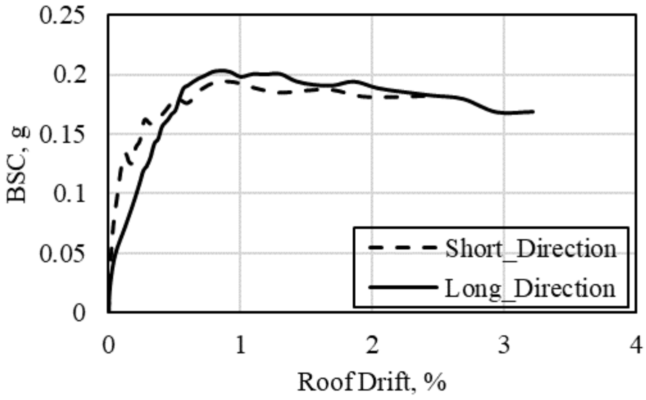
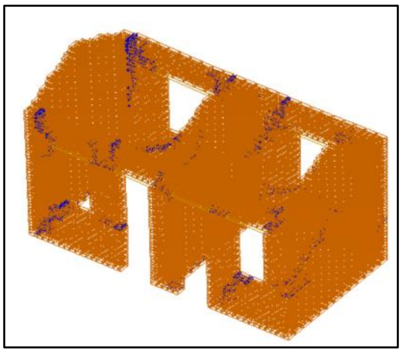
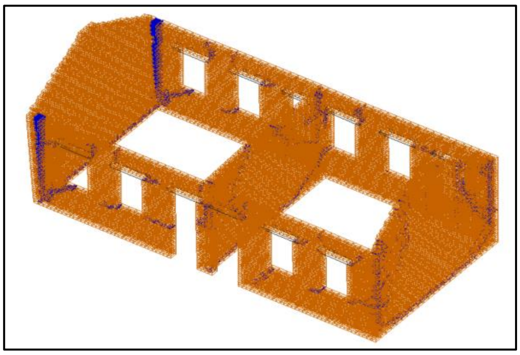


Figure 4-57. Capacity curves for the UCM-URM7/LR/VLP building under OOP behavior when loaded in the two principal directions.

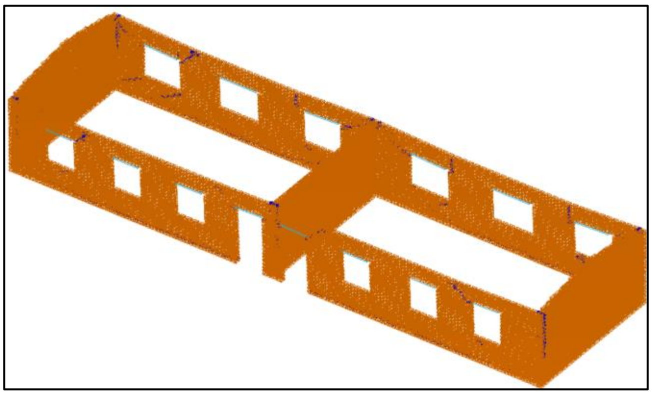
Figure 4-58Figure 4-53 shows the collapse mechanisms of the index buildings with different unrestrained wall panel lengths. The SP model has a box-like global behavior thus improved seismic performance while the VLP model has very weak long unrestrained walls under OOP direction when loaded in the shorter direction.



UCM-URM7/LR/LD/-/SP



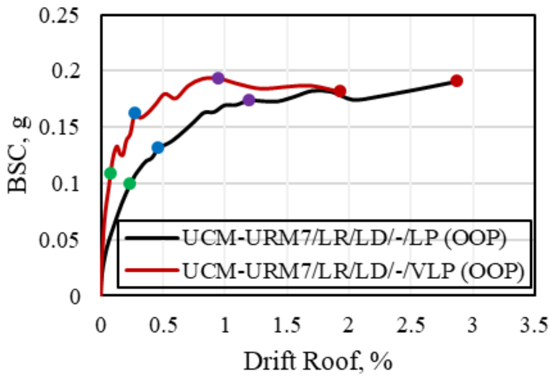
UCM-URM7/LR/LD/-/LP



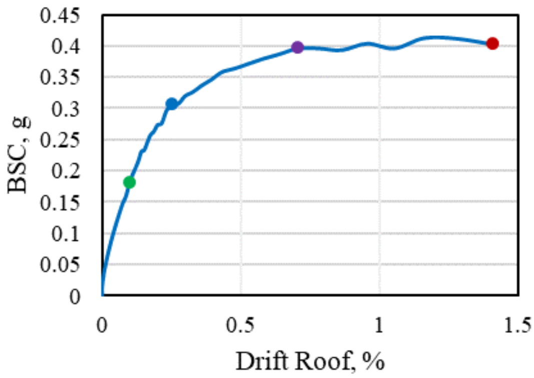
UCM-URM7/LR/LD/-/VLP

Figure 4-58. Collapse mechanisms of UCM-URM7 index buildings with different unrestrained wall panel lengths. The blue lines represent the extensive cracks of width more than 12.5 mm (only extensive cracks are shown). Note that the VLP model is loaded in the transverse direction.

Figure 4-59 shows the capacity curves along with the different damage state thresholds marked along the capacity curves. As explained before, the building total capacity curve (instead of separate IP and OOP capacity curves) is plotted for the SP model because of the controlled displacement at the roof level due to the box-like behavior while for the LP and VLP models, the capacity curves under OOP behaviors are plotted as the OOP behavior controls the collapse. The OOP capacity curve for VLP has higher initial stiffness and strength but the displacement capacity is reduced compared to the LP model because of the large unrestrained wall panel.



UCM-URM7/LR/LD/-/LP and UCM-URM7/LR/LD/-/VLP



UCM-URM7/LR/LD/-/SP

Figure 4-59. Comparison of capacity curves for the index buildings with different unrestrained wall panel lengths. The colored dots represent the threshold of different damage states: Green = Slight Damage, Blue = Moderate Damage, Purple = Extensive Damage and Red = Collapse.

Figure 4-60 shows the vulnerability curves for different index buildings with different wall panel lengths. For VLP index building, the vulnerability is considerably higher while the same reduces noticeably when the wall panels are short (SP model).

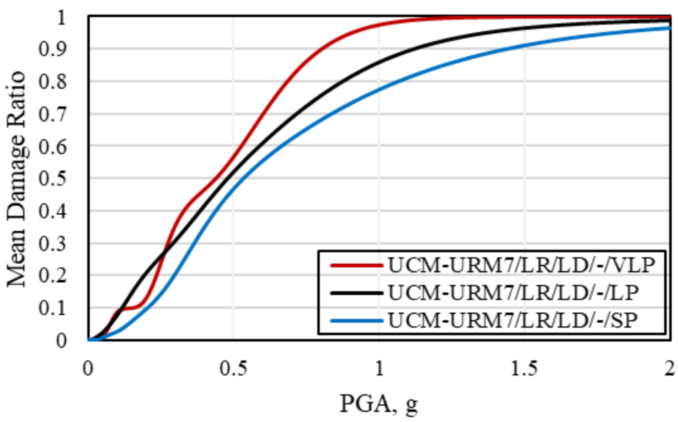
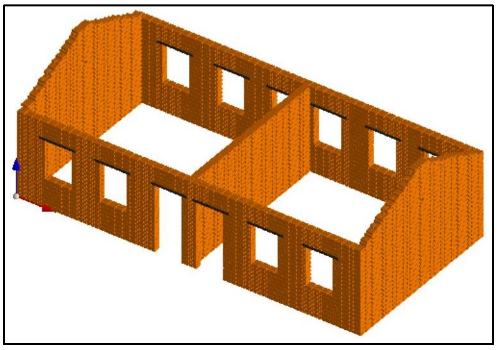


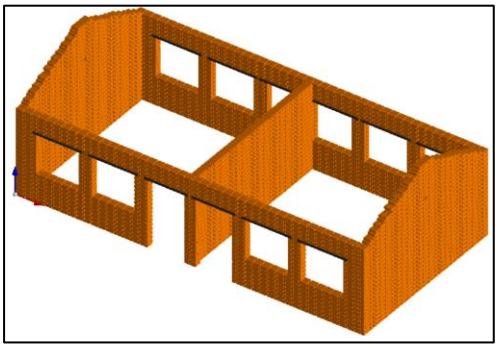
Figure 4-60. Comparison of vulnerability curves for the index buildings with different wall panel lengths.

4.5.1.5 Wall Opening

The openings (number, size and layout) can greatly reduce the shear/flexural capacity of masonry walls. The opening is considered small when the total width of opening in an unrestrained wall panel is less than 50% of the wall length. Two different percentages of wall openings are considered in the present study: one with 46% opening (small opening, SO model) and another with 65% opening (large opening, LO model). Figure 4-61 presents the numerical models of UCM-URM7 index buildings with different opening configurations.



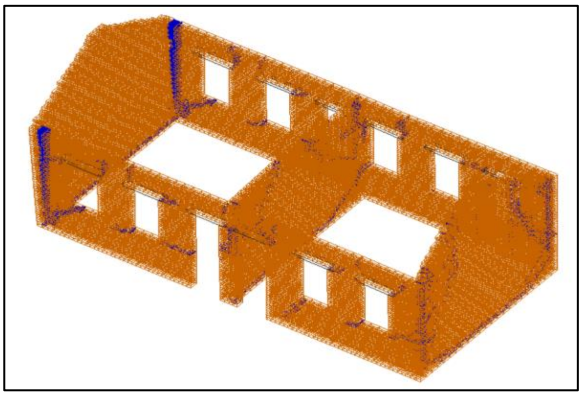
UCM-URM7/LR/LD/-/SO



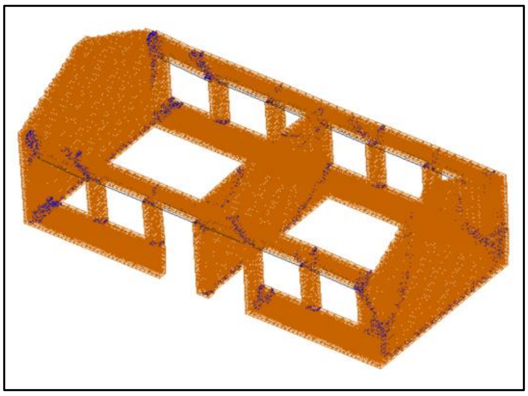
UCM-URM7/LR/LD/-/LO

Figure 4-61. Numerical models of UCM-URM7 index buildings with different wall opening configurations.

Figure 4-62 shows the collapse mechanisms of the index buildings with different opening configurations. The building with large openings has very weak and slender piers in the IP walls and the OOP walls easily develop the combined mechanism by detaching the portion of weaker connections with IP walls.



UCM-URM7/LR/LD/-/SO



UCM-URM7/LR/LD/-/LO

Figure 4-62. Collapse mechanisms of UCM-URM7 index buildings with different opening configurations. The blue lines represent the extensive cracks of width more than 12.5 mm (only extensive cracks are shown).

Figure 4-63 shows the capacity curves along with the different damage state thresholds marked along the capacity curves. For the LO model, the initial stiffness, strength and the displacement capacity are considerably reduced compared to the SO model.

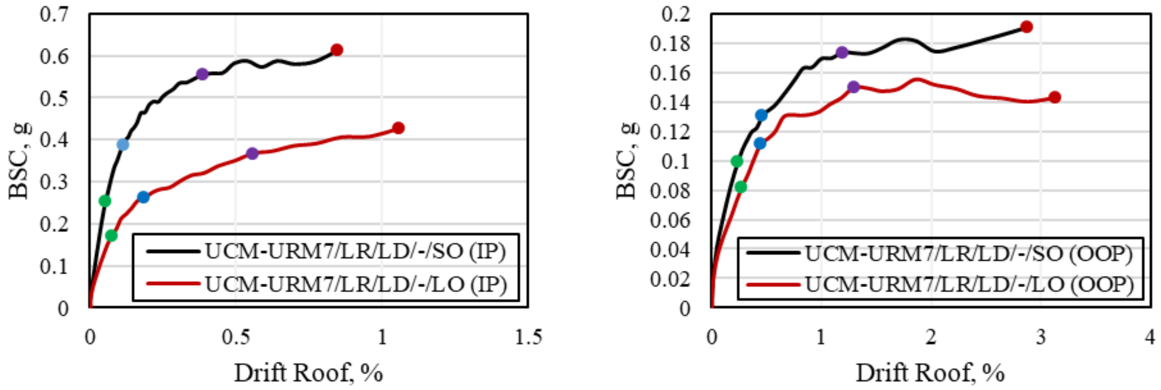


Figure 4-63. Comparison of capacity curves for the index buildings with different wall opening configurations. The colored dots represent the threshold of different damage states: Green = Slight Damage, Blue = Moderate Damage, Purple = Extensive Damage and Red = Collapse.

Figure 4-64 shows the vulnerability curves for different index buildings with different opening configurations. For LO index building, the vulnerability noticeably increases compared to the SO index building.

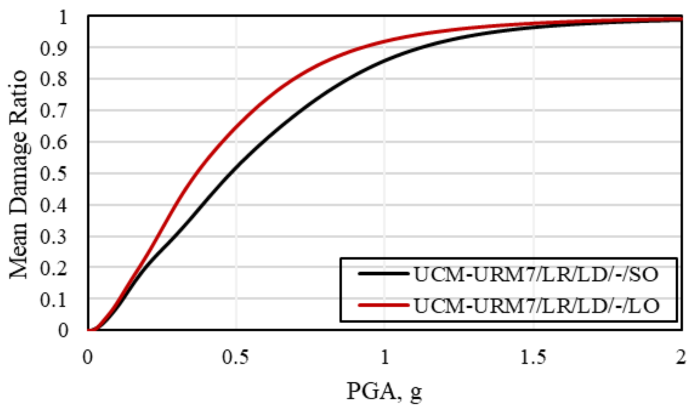
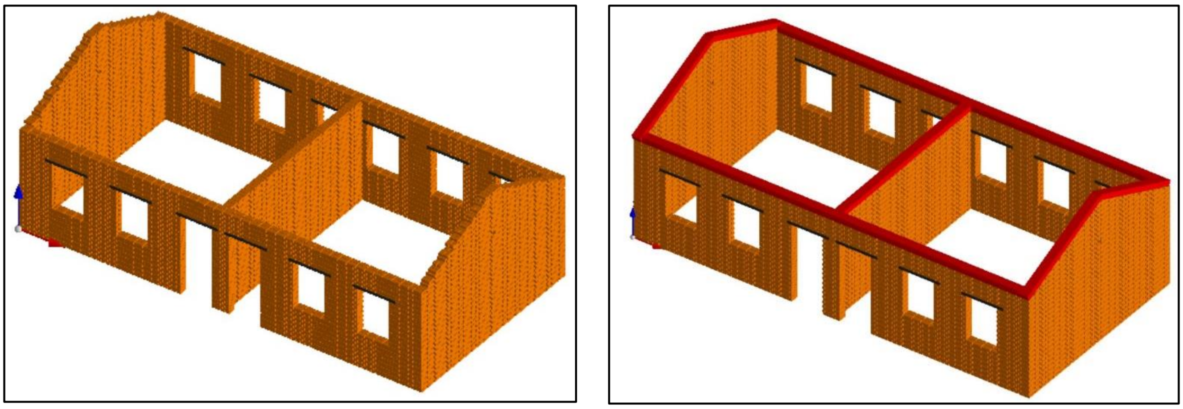


Figure 4-64. Comparison of vulnerability curves for the index buildings with different wall opening configurations.

4.5.1.6 Effective Seismic Retrofitting

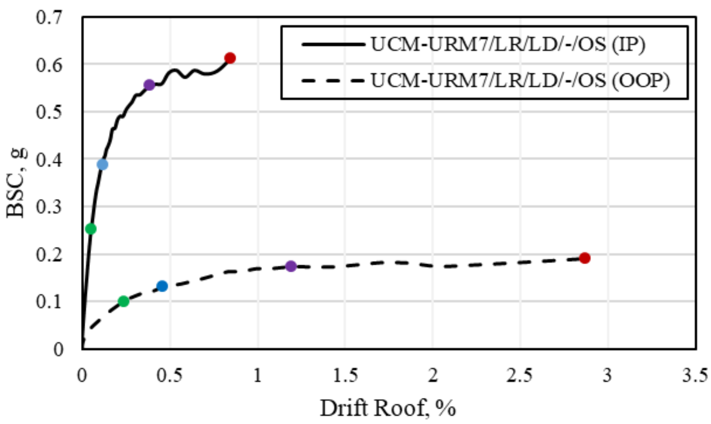
The seismic performance of the poorly designed older masonry buildings can be improved by applying effective seismic retrofitting. In this study, one of the most common strengthening methods, i.e. addition of roof level RC band beam is employed to determine the improvement in the building’s seismic behavior. As shown previously in case of MD and HD models, the roof level band beam will control the OOP wall failures thus improving the global seismic behavior. However, care should be taken when applying such retrofitting intervention as the high difference in the stiffness of the original structure and applied retrofitting measures can degrade the seismic performance. For example, if the units or mortar quality in the existing building is poor (or deteriorated), the structure cannot take the overburden due to the addition of retrofitted elements or the shear resistance at the boundary can be insufficient.



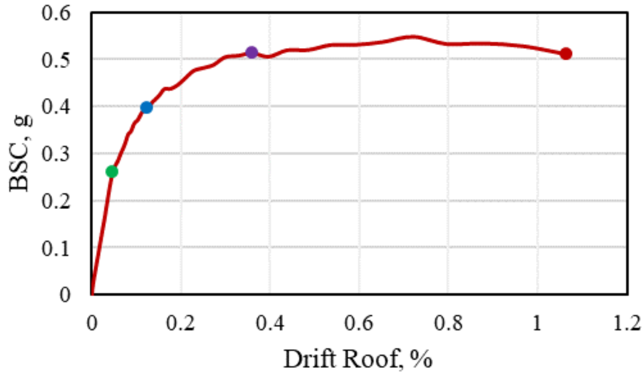
UCM-URM7/LR/LD/-/OS   UCM-URM7/LR/LD/-/RS

Figure 4-65. Numerical models of UCM-URM7 index buildings: original building and retrofitted building.

Figure 4-66 shows the capacity curves along with the different damage state thresholds marked along the capacity curves. It can be seen from the capacity curves that the both the strength (with respect to the OOP wall of FD model) and displacement capacity (with respect to the IP wall of FD model) is improved in the case of retrofitted structure (RS).



UCM-URM7/LR/LD/-/OS



UCM-URM7/LR/LD/-/RS

Figure 4-66. Comparison of capacity curves for the index buildings: original structure and retrofitted structure. The colored dots represent the threshold of different damage states: Green = Slight Damage, Blue = Moderate Damage, Purple = Extensive Damage and Red = Collapse.

Figure 4-64 shows the vulnerability curves for different index buildings with different opening configurations. In case of RS index building, the vulnerability greatly reduces compared to the OS index building.

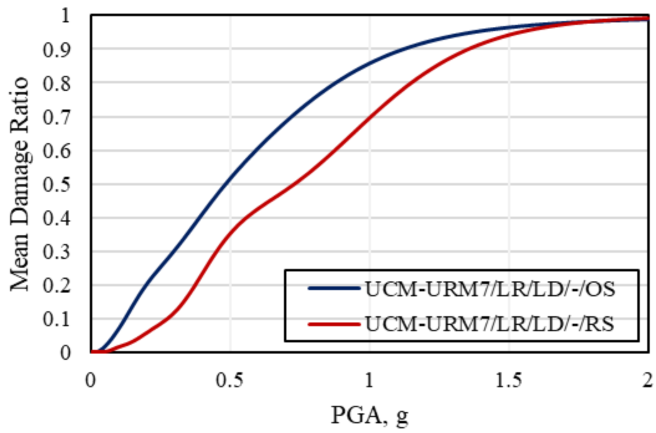


Figure 4-67. Comparison of vulnerability curves for the original and retrofitted index buildings.

4.5.1.7 Structural Health Condition

Structural health condition is the current condition of a building with respect to its material quality, existing damages etc. The quality of construction materials, and present deterioration condition, highly influences the seismic capacity and performance of a masonry building and has a high variability from one building to another. Thus, a comparison of the analysis results for three different index buildings with different material qualities are presented. The very poor condition (VPC) building model has 40% lower values of material properties than that of the baseline model (poor condition, PC whose material properties are presented in [refer to the table]) while the good condition (GC) model has 100% better values of material properties than that of the baseline model. However, it should be noted that in reality, the material properties in the same building typology can vary drastically from building-to-building within a country or from one country to another.

Figure 4-68 shows the capacity curves along with the different damage state thresholds marked along the capacity curves. As the material quality increases, the initial stiffness, strength and the displacement capacity also increase. The effect is more pronounced in the IP seismic behavior.

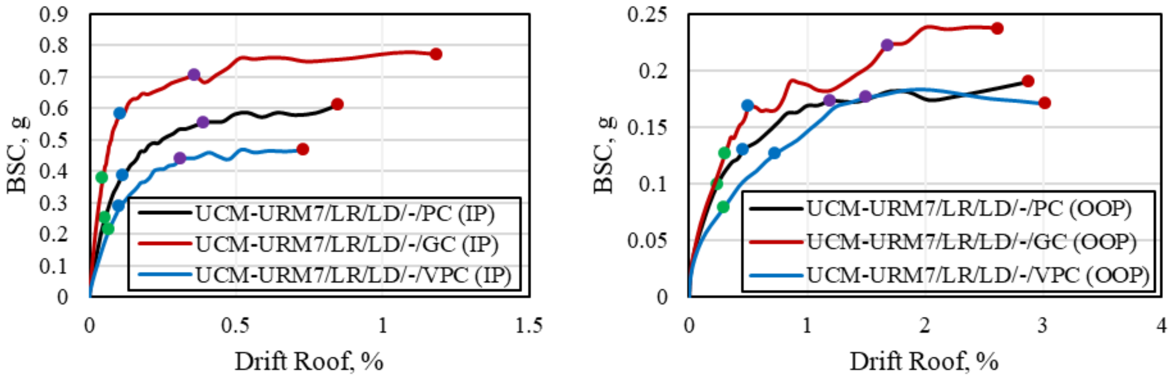


Figure 4-68. Comparison of capacity curves for the index buildings with different material quality. The colored dots represent the threshold of different damage states: Green = Slight Damage, Blue = Moderate Damage, Purple = Extensive Damage and Red = Collapse.

Figure 4-69 shows the vulnerability curves for different index buildings with different material quality. Buildings with poor quality materials (in the original construction or deteriorated) are highly vulnerable while the vulnerability can be greatly reduced if good quality materials are used in the building construction.

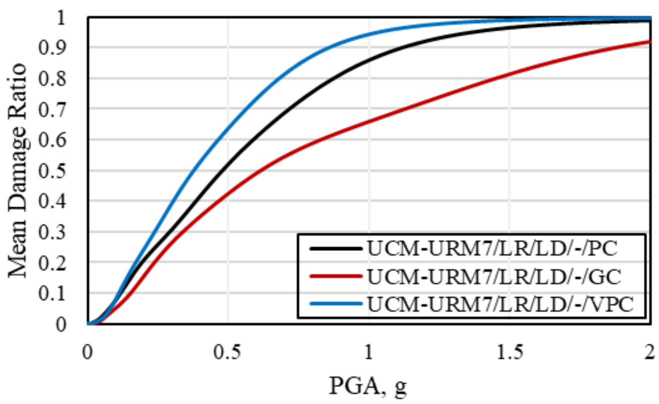


Figure 4-69. Comparison of vulnerability curves for the index buildings with different material quality.

The sensitivity analysis results show that the seismic performance and vulnerability is highly sensitive to the different vulnerability parameters and their attributes (range). Figure 4-69 compares the vulnerability curves for different index buildings of the UCM-URM7 building type. It is obvious that the vulnerability varies greatly with considerable dependence on all different sensitivity parameters. As expected, the model representing the high design (HD) case shows lowest vulnerability while the models with large openings (LO) and poor material qualities (PC) show highest vulnerability in realistic PGA range.

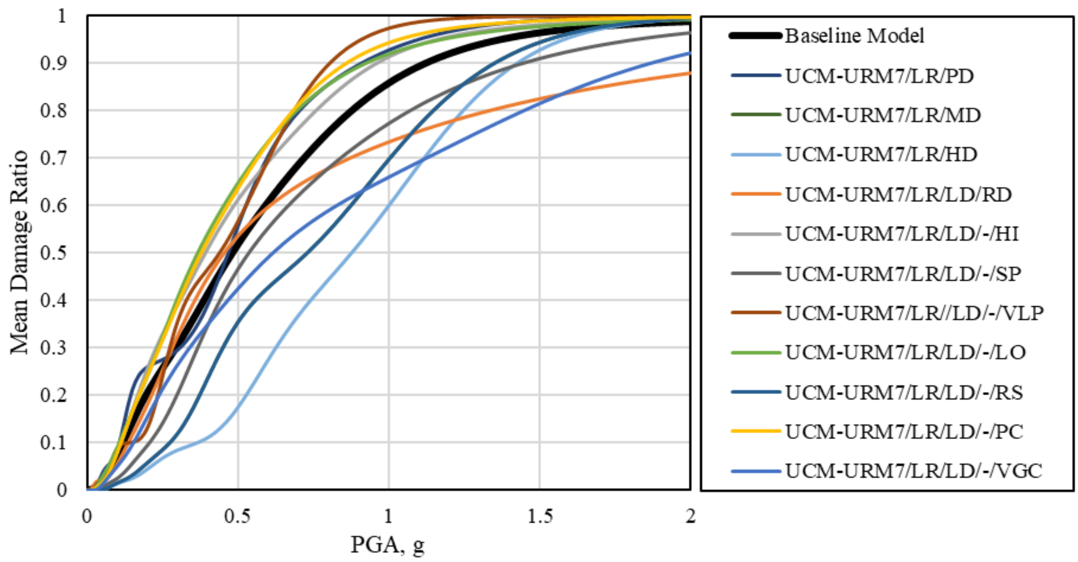


Figure 4-70. Comparison of all vulnerability curves for different index buildings considered in the sensitivity analysis.

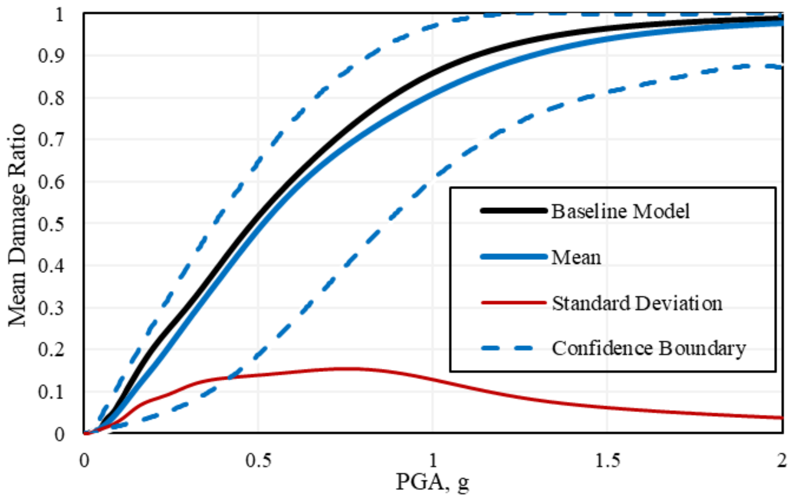


Figure 4-71. Mean, confidence boundary and the standard deviation of all different vulnerability functions for the UCM-URM7 school building class.

Figure 4-71 presents a more concise and meaningful plot of the vulneability curves in which the vulnerability curve of the baseline model, mean (of all different vulnearbility curves) and the confidence boundary as well as the standard deviation are depicted. It is interesting to note that the mean vulnearbility curve and the vulnerability curve for the baseline model are similar which proves that the baseline model (UCM-URM7/LR/LD) represents a building with average construction characteristics and was a good choice for the reference model. The confidence boundaries shown represent the variability of the vulneability values at a given IM (i.e. PGA) which is very useful in decesion making.

Table 4-19 presents the summary of the results with respect to the changes in seismic behavior, damage indicators and vulnerability.

Table 4-19. Summary of sensitivity analysis results for UCM-URM7 IB.

Parameter	Attributes	Global box-like behavior	Collapse Mechanism	% change with respect to the baseline case			
				Initial Stiffness	Ultimate Capacity	Ultimate Roof Drift	PGA at 50% MDR
Seismic Design Level	Poor Design (PD)	No	Collapse of OOP walls	0%	-15%	0%	-11%
	Low Design (LD) **Reference**	No	Collapse of OOP walls	-	-	-	-
	Medium Design (MD)	Yes	Shear failure of IP piers	1174%	168%	25%	58%
	High Design (HD)	Yes	Shear failure of IP piers	1893%	311%	33%	96%
Diaphragm Type	Flexible Diaphragm (FD) **Reference**	No	Collapse of OOP walls	-	-	-	-
	Rigid Diaphragm (RD)	Yes	Shear failure of IP piers	363%	167%	73%	0%
Irregularity	No Irregularity (NI) **Reference**	No	Collapse of OOP walls	-	-	-	-
	Horizontal Irregularity (HI)	No	Collapse of OOP walls	-46%	-23%	-45%	-22%
Wall Panel Length	Very Long Panel (VLP)	No	Collapse of OOP walls	212%	11%	-33%	-10%
	Long Panel (LP) **Reference**	No	Collapse of OOP walls	-	-	-	-
	Short Panel (SP)	Yes	Shear failure of IP piers	310%	130%	66%	15%
Wall Opening	Large Opening (LO)	No	Collapse of OOP walls	-52%	-34%	9%	-25%
	Small Opening (SO) **Reference**	No	Collapse of OOP walls	-	-	-	
Effective Seismic Retrofitting	Original Structure (OS) **Reference**	No	Collapse of OOP walls	-	-	-	-
	Retrofitted Structure (RS)	Yes	Shear failure of IP piers	1174%	168%	25%	58%
Structural Health Condition	Very Poor Condition (VPC)	No	Collapse of OOP walls	-31%	-33%	-14%	-24%
	Poor Condition (PC) **Reference**	No	Collapse of OOP walls	-	-	-	-
	Good Condition (GC)	No	Collapse of OOP walls	74%	17%	39%	27%

With respect to the seismic design level, the initial stiffness, ultimate capacity, ultimate drift is greatly improved (most significant improvement) when the seismic design level is medium and high (MD and HD) in comparison to poor and low design (PD and LD) cases. Similarly, the PGA level for 50% MDR is also significantly improved. Similarly, the introduction of rigid diaphragm (RD) or seismic strengthening (RS) also improve the seismic behavior noticeably. Global box-like behavior is obtained when the seismic design level is MD and HD, the diaphragm is RD type and when the structure is effectively retrofitted (RS).

When the openings are large (LO) or there is a horizontal irregularity (HI) introduced due to the difference in opening, the seismic capacity as well as the PGA for 50% MDR are noticeably reduced. In case of very long unrestrained panels (VLP), although the initial stiffness and ultimate capacity are higher, the ultimate drift capacity is reduced and hence is more vulnerable. However, when the unrestrained wall panels are short (SP), the building develops a box-like global behavior and the vulnerability reduces. With respect to the structural health condition, the seismic capacity improves when the material quality is good (GC) in comparison to the case when the material quality is poor (VPC and PC). The vulnerability can be further reduced by using better quality construction materials.

4.5.2 Reinforced Concrete

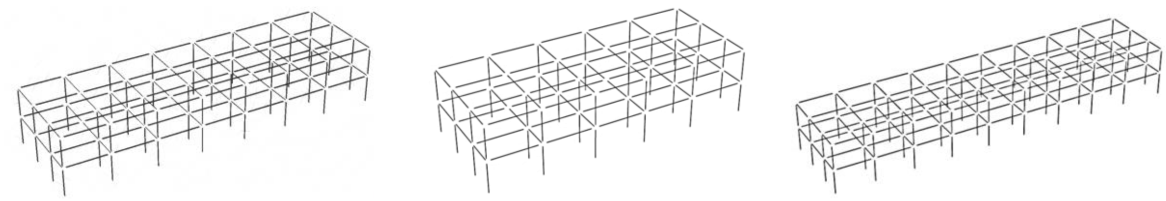
This section presents the sensitivity analysis for Reinforced Concrete school buildings. The main objective of these analysis is to understand the impact on the final vulnerability assessment with the expected variation in critical parameters that usually control the final results of the assessment. The following variables were considered:

- Geometrical variations: three different geometries considering 2, 3 and 5 classrooms.
- Ground motion records for different soil types: hard, medium and soft.
- Foundation-soil flexibility: combinations of different soil types and type of foundations.
- Masonry infills quality: high, medium and poor.
- Non-structural vulnerable elements: ductile and fragile behavior.
- Analysis type: N2 method vs. incremental dynamic analysis (IDA).

These analyses where performed using same methodological approach described above and the results are illustrated in the following sections.

4.5.2.1 Geometrical variations

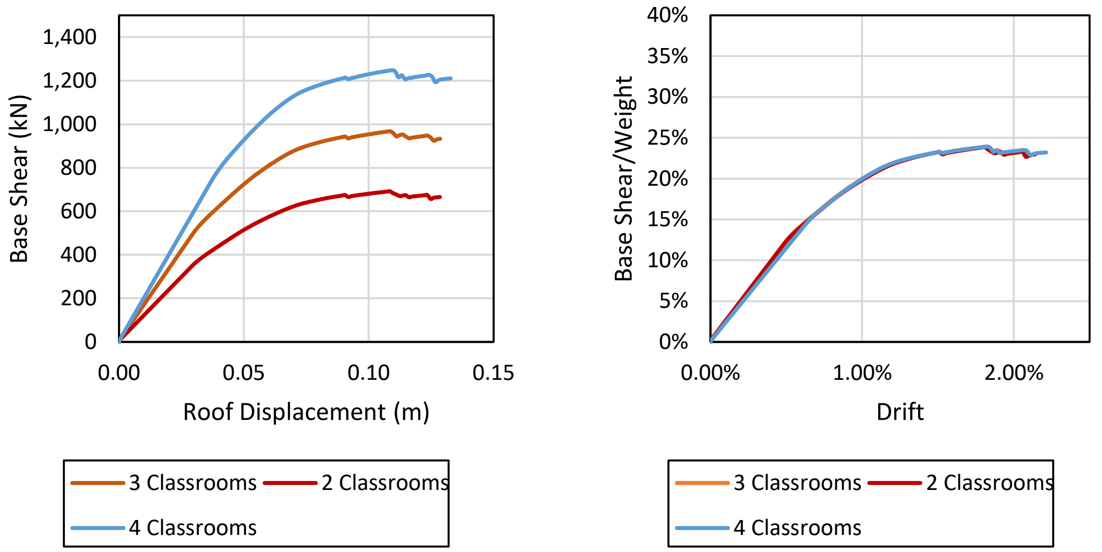
The first sensitivity analysis is made for eventual and expected geometrical variations of the school's layouts. Analysis was performed using as a basis the computer model of index building IBRC-2 (RC1-MR-LD). Three different layouts were selected for the analysis as illustrated in Figure 4-72, representing three (the most common), two and four typical classrooms. All models were considered two story height.



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

Figure 4-72. School buildings modules

Figure 4-73 presents the capacity curves relating the maximum roof displacement associated to different total base shear forces. Normalized pushover curves are also included in order to compare relative behavior between the different models considered. As it is shown in these figures, the normalized capacity curves for the three models do not present significant variations from each other. Considering that the Engineering Demand Parameters (EDP) are obtained using the N2 method (see Chapter 4.2.6), no significant variations are expected in the final vulnerability functions for the three models. Therefore, it is concluded that the proposed 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.

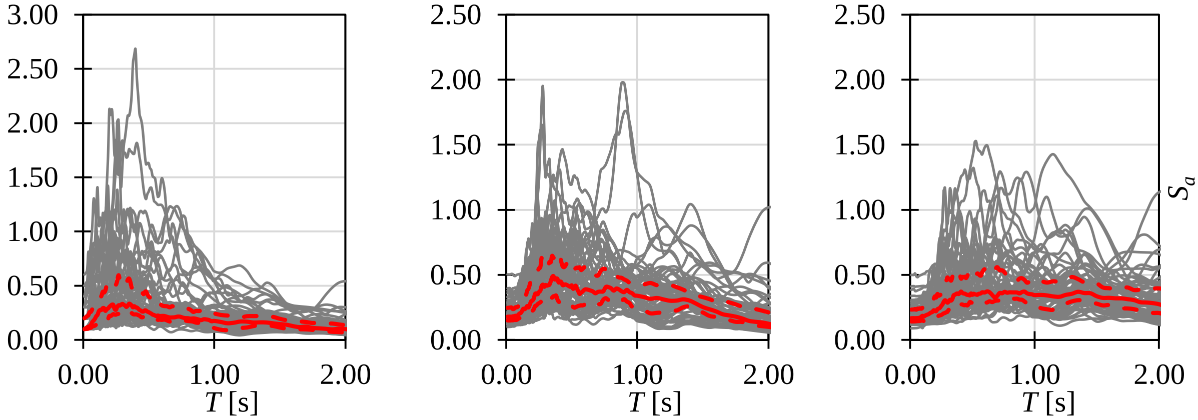


a) Capacity curves b) Normalized capacity curves

Figure 4-73. Capacity curves for different geometries

4.5.2.2 Ground motion records for different soil types

Ground motion sensitivity analysis was performed using the computer model developed for index building IBRC-3 (RC1-MR-HD). Three different ground motion sets were obtained analytically for the following representative soil profiles: stiff soil (rock), intermediate, and soft soils. Figure 4-74 shows the acceleration response spectra for each set of records.



a) Stiff soils b) Intermediate soils c) Soft soils

Figure 4-74. Ground motions acceleration response spectra for different soil profiles

Figure 4-75 presents the resulting vulnerability functions for the same Index Building but using the abovementioned ground motion sets. From these results it can be concluded the following:

- No significant variations are obtained in the mean damage ratio for low seismic intensities (Sa(T) less than 0,5 g).
- For larger intensities, maximum variations of about 20% are obtained in the mean damage ratio when using soft soil typical records as compared to stiff soil ones.
- Soft soil records tend to generate greater expected building level damages.
- Using only stiff soil records can underestimate the building damage for high seismic intensities. In case that the soil profile conditions are not known, it is recommended to use a combination of stiff and soft soil typical ground motions.

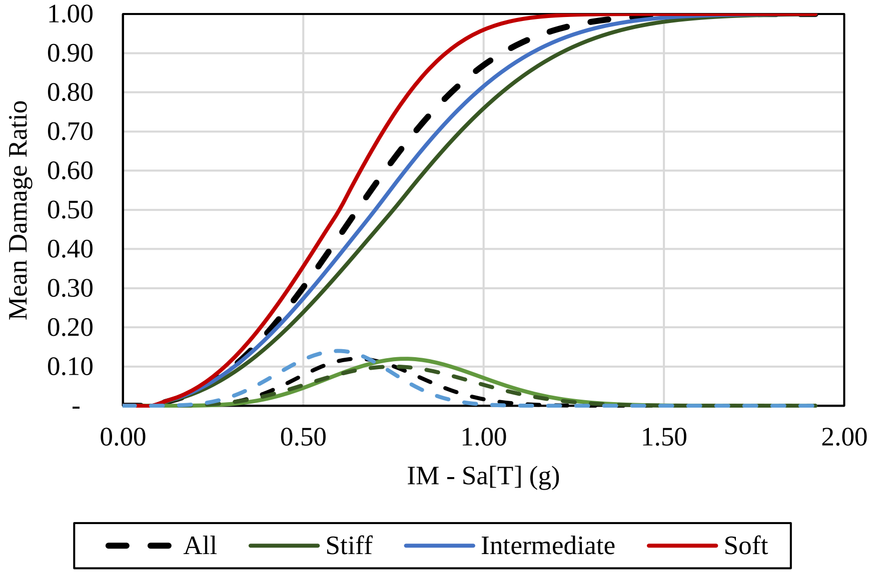


Figure 4-75. Vulnerability functions for the same building in different type of soils

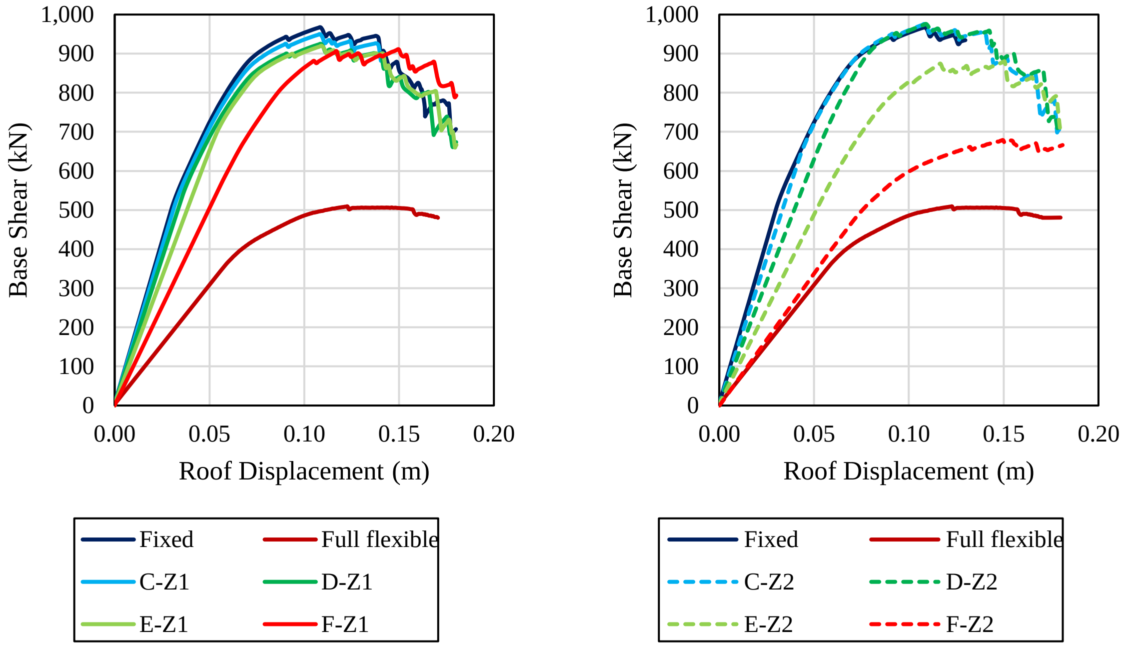
4.5.2.3 Foundation-soil flexibility

In order to assess the possible variations in the vulnerability functions when the soil-foundation stiffness is considered, index building model IBRC-3 was used as reference for the analysis. Two different foundations configurations were tested (1.0 m by 1.0 m (Z1) and a 0.5 m by 0.5 m. (Z2) isolated footings) when combined with four different soil types as indicated in Table 4-20.

Table 4-20. Soil properties for foundation stiffness calculation

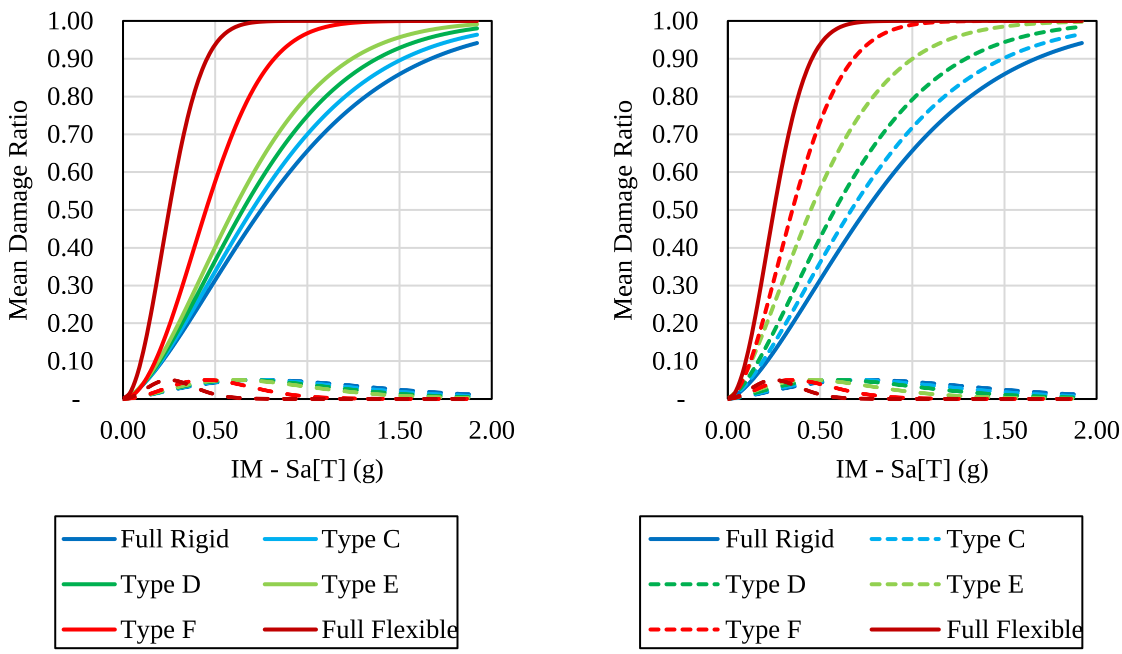
Type	G/G0	Soil	Density $_{sat\ (kN/m3)}$	Vs30 (m/s2)	v
C	0.9	Lime	22	500	0.35
D	0.81	Clay	18	300	
E	0.47	Clay	18	200	
F	0.32	Clay	18	100	

Resulting capacity curves are presented in Figure 4-76 for all possible combinations of foundation configuration and soil type. Corresponding vulnerability functions are presented in Figure 4-77.



Z1 foundation Z2 foundation

Figure 4-76. Capacity curves with foundation in different soil types



Z1 foundation Z2 foundation

Figure 4-77. Vulnerability functions with different foundation stiffness

The following conclusions van be drawn from these results:

- Good foundation configurations (represented by Z1 footings) will generate pushover curves and vulnerability functions in close relation to the rigid base model, except for a considerable flexible soil, in which case some significant variations in response would be expected.

- For relatively weak foundation configurations (represented by Z2 footings), considerable variations would be expected for different soil types. For stiff soil profiles (soil types A, B, 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 in the previous table) the expected behavior will approximate the hinged base assumption.

- In general, 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 condition for the building under consideration.

4.5.2.4 Masonry infills quality

To test the relevance of masonry infills quality in the final vulnerability assessment, different masonry properties are selected as is described in Table 4-21 to perform a sensitivity analysis. In this case index building model IBRC-9 (RC2/MR/LD) was selected.

Table 4-21. Masonry properties

Quality	Age	Country	Block Material	Dimensions (bxLxt)	fv (Mpa)	E (Mpa)	Friction coefficient
High	New	Colombia	Clay brick	10x20x6	0.9	8700	0.7
Medium	Intermediate	USA	Clay brick	10x28x6	0.13	1050	
Poor	Old	Colombia	Clay tile	11x30x20	0.1	1560	

Figure 4-78 presents the capacity curves obtained using the three previous masonry quality conditions as compared to the bare frame (no masonry infills) conditions. From the figure 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. In some cases, weak floor failure mechanism can be generated when the first-floor infill walls fail under horizontal seismic loading.

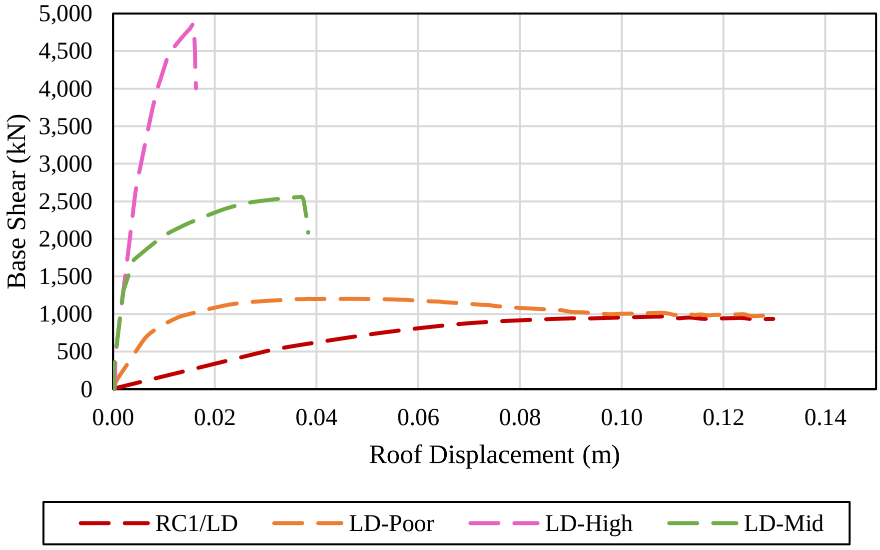


Figure 4-78. Capacity curves using different masonry qualities

Figure 4-79 illustrates that a great variability of results are expected for the range of masonry infills qualities considered. It's worth noting that the curves are not directly comparable because the building structural predominant period will significantly change depending on the quality of the masonry infills and therefore different intensity parameter will be used for the risk assessment.

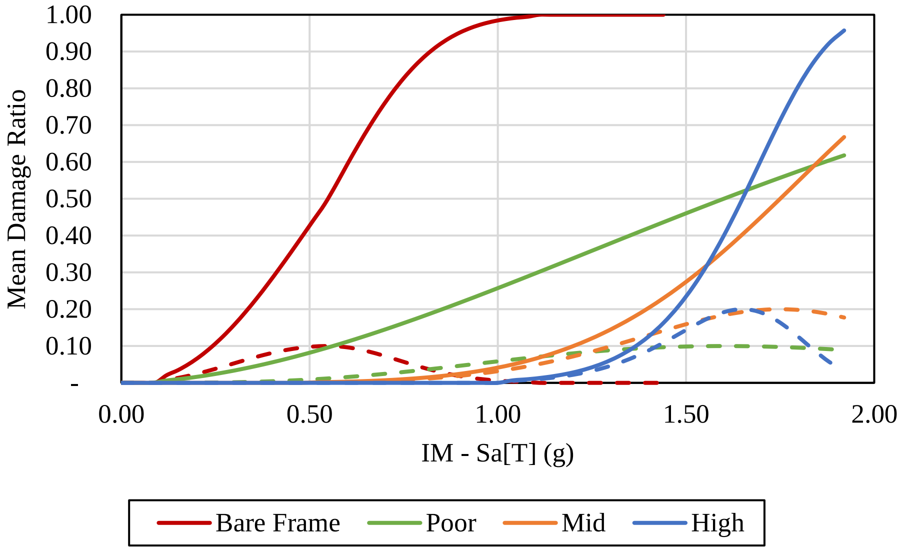


Figure 4-79. Vulnerability functions using different masonry qualities

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.

4.5.2.5 Non-structural vulnerable elements

The objective of this sensitivity analysis is to identify the effect of considering non-structural elements (NEE) in the loss calculation process. For this, index building model IBRC-3 (RC1/MR/HD) is selected. The following three conditions are considered: (i) No non-structural elements; (ii) poor quality fragile non-structural elements and; (iii) high quality ductile non-structural elements. Table 4-22, Table 4-23 and Table 4-24 presents the component models for these three conditions.

Table 4-22. Only structural elements component model

Story	Group	Subgroup	Description	Quantity	Fragility curve	EDP	Correlation
1	E	C1	Column-one beam	8	B1041.091a	Drift	0
1	E	C2	Column-two beams	21	B1041.091b	Drift	0
2	E	C1	Column-one beam	8	B1041.091a	Drift	0
2	E	C2	Column-two beams	21	B1041.091b	Drift	0

Table 4-23. Poor quality component model

Story	Group	Subgroup	Description	Quantity	Fragility curve	EDP	Correlation
1	E	C1	Column-one beam	8	B1041.091a	Drift	0
1	E	C2	Column-two beams	21	B1041.091b	Drift	0
1	A	F2	Masonry facade	14	C1011.006a	Drift	1
1	A	M4	Masonry wall	6	C1011.006b	Drift	1
1	C	S2	Contents	13	E2022.010a	Drift	0
2	E	C1	Column-one beam	8	B1041.091a	Drift	0
2	E	C2	Column-two beams	21	B1041.091b	Drift	0
2	A	F2	Masonry facade	14	C1011.006a	Drift	1
2	A	M4	Masonry wall	6	C1011.006b	Drift	1
2	C	S2	Contents	13	E2022.010a	Drift	0

Table 4-24. High quality component model

Story	Group	Subgroup	Description	Quantity	Fragility curve	EDP	Correlation
1	E	C1	Column-one beam	8	B1041.001a	Drift	0
1	E	C2	Column-two beams	21	B1041.001b	Drift	0
1	A	F2	Masonry facade	14	C1011.001a	Drift	1
1	A	M4	Masonry wall	6	C1011.001a	Drift	1
1	C	S2	Contents	13	E2022.010a	Drift	0
2	E	C1	Column-one beam	8	B1041.001a	Drift	0
2	E	C2	Column-two beams	21	B1041.001b	Drift	0
2	A	F2	Masonry facade	14	C1011.001a	Drift	1
2	A	M4	Masonry wall	6	C1011.001a	Drift	1
2	C	S2	Contents	13	E2022.010a	Drift	0

Figure 4-80 presents the vulnerability curves for each one of the cases explained above.

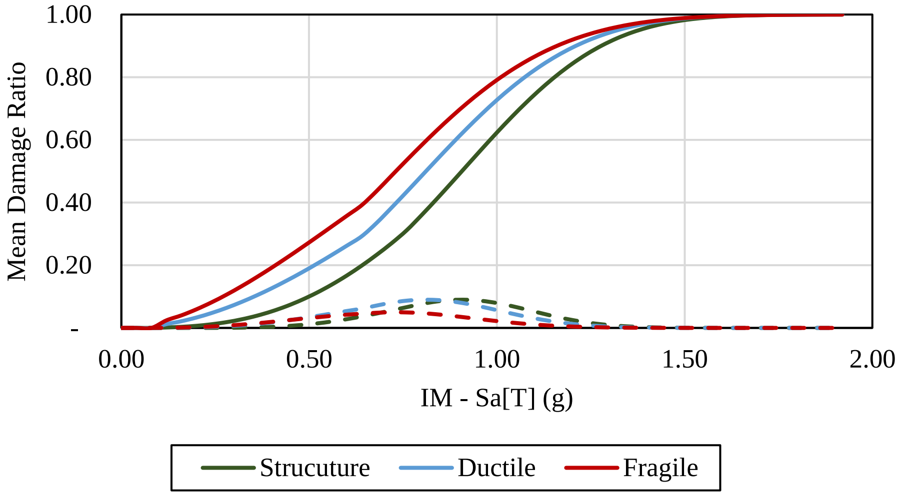


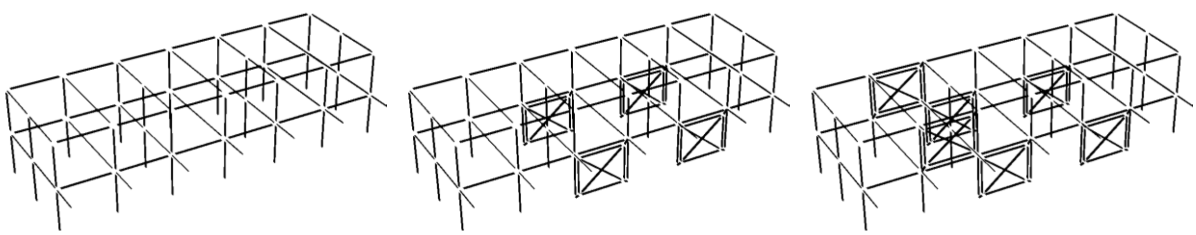
Figure 4-80. Vulnerability functions using different component models

From the previous results it can be concluded that variations on the order of 20% in the mean damage ratio could be expected when considering fragile NEE as compared with a building with no NEE for the lower ranges of seismic intensities. In addition, lower relative variations are expected in the higher range of seismic intensities, due to the fact that global building collapses would control the losses in that intensity range.

As a general recommendation, NEE shall be included in the vulnerability assessment when they represent a significant replacement values as compared to the structure itself, and when they are expected to observe a fragile behavior and significant damage after an earthquake (no seismic design). The consideration of the NEE 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.

4.5.2.6 Analysis type

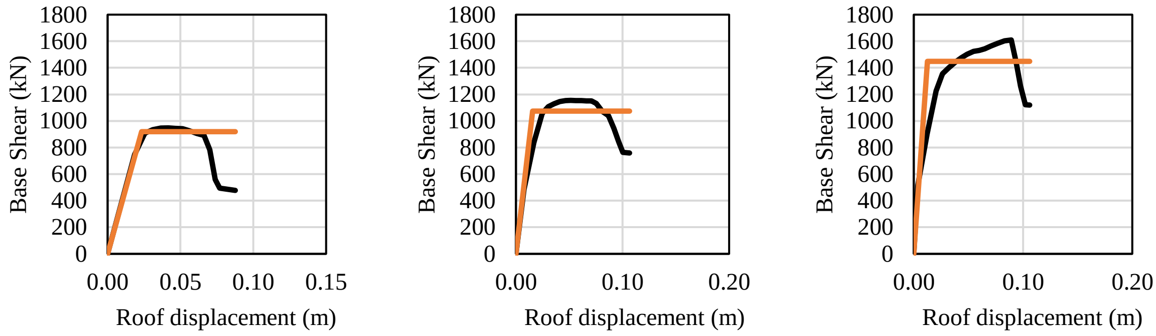
In order to establish the reliability of the N2 (see Chapter 4.2.6), results are compared with equivalent incremental dynamic analysis (IDA). For that, three different models are considered, one RC1 model and two RC4 models (RC4-LD and RC4-HD), as shown in Figure 4-81.



a) RC1-LD b) RC4-LD c) RC4-HD

Figure 4-81. Analytical models for sensitivity analysis

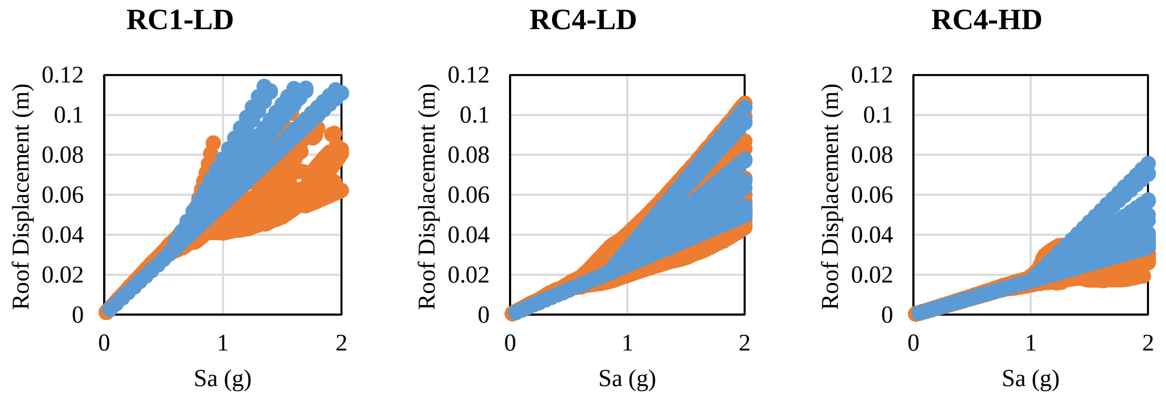
Capacity curves for these three models are presented in Figure 4-82. These were obtained using the methodological approach explained in detail in Chapter 4.2:



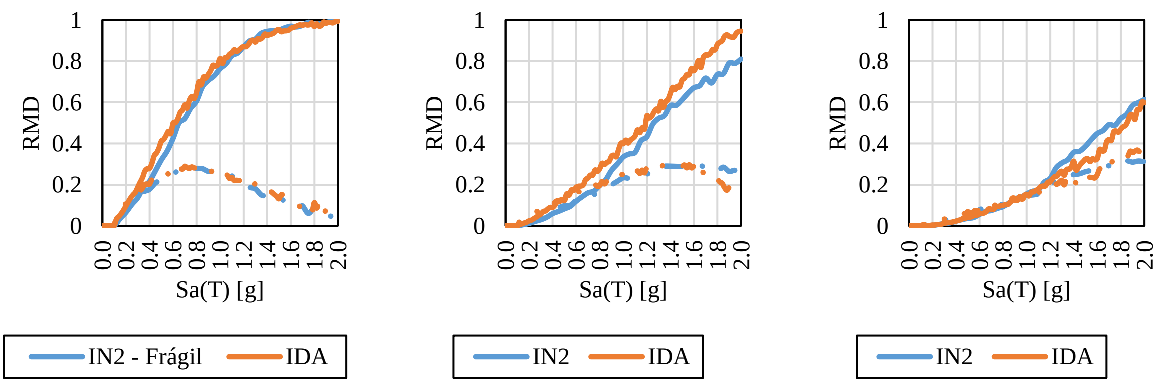
a) RC1-LD b) RC4-LD c) RC4-HD

Figure 4-82. Capacity curves

Engineering demand parameters (EDP) and the corresponding vulnerability functions are presented in Figure 4-83.



Engineering demand parameters (EDP)



Vulnerability functions

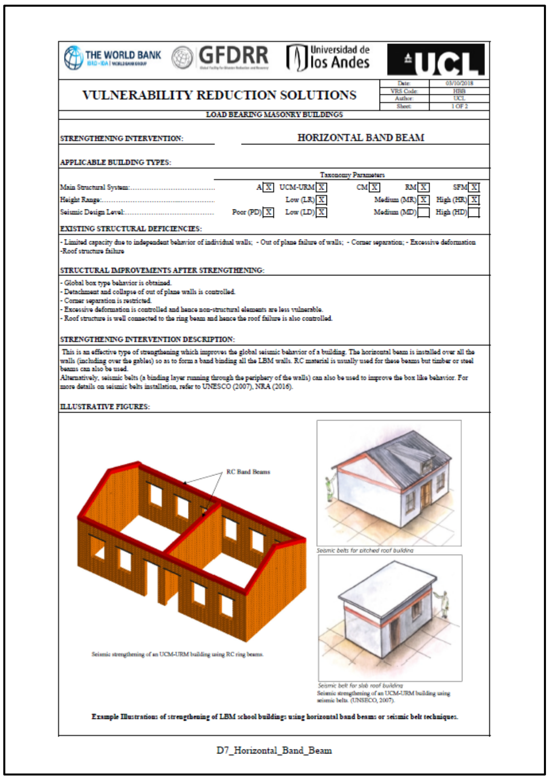
Figure 4-83. EDP and vulnerability curves

From those 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. Both methodologies generate similar mean and dispersion values. For the vulnerability assessment of typical school buildings, the N2 method is clearly a reliable option for EDP calculations. Caution shall be exerted when considering non–typical school buildings which behavior may be influenced by irregularities, variations in height, combined structural systems or any other special characteristic.

5. REVIEW OF METHODS FOR VULNERABILITY REDUCTION SOLUTIONS

This chapter documents different retrofitting interventions that can be applied to different building types of LBM and RC school buildings. It should be noted that the strengthening methods mentioned in this document are collected from literature and are not independently verified by the authors. Also, the retrofitting intervention methods proposed in this document are general which are not specifically designed. One needs to design the details of these interventions for application to a real school building. Further detailed structural analysis and assessment/design is required for the validation of the proposed methods or to develop new ones.

For some IB in the GLoSI, the main weaknesses in the buildings and the collapse mechanisms are identified, and typical practical retrofitting or reinforcement options to improve the seismic performance are proposed. These would represent possible economical vulnerability reduction solutions which could conform the bases for a risk mitigation plan for the school infrastructure. Figure 5-1 and Figure 5-2 the format developed to summarize existing information on possible structural interventions in order to reduce vulnerability. **Annex L** summarize several vulnerability reduction options that have been reported in recent interventions programs in the school sectors of different countries, some of them as part of risk mitigation programs and some other as part of reconstruction program after the occurrence of a particular seismic event.



VULNERABILITY REDUCTION SOLUTIONS

Date:	03/10/2018
VRS Code:	HBB
Author:	UCL
Sheet:	1 OF 2

LOAD BEARING MASONRY BUILDINGS

STRENGTHENING INTERVENTION: HORIZONTAL BAND BEAM

APPLICABLE BUILDING TYPES:

Taxonomy Parameters

Main Structural System: A [X] UCM-URM [X] CM [X] RM [X] SFM [X]
Height Range: Low (LR) [X] Medium (MR) [X] High (HR) [X]
Seismic Design Level: Poor (PD) [X] Low (LD) [X] Medium (MD) [ ] High (HD) [ ]

EXISTING STRUCTURAL DEFICIENCIES:

- Limited capacity due to independent behavior of individual walls; - Out of plane failure of walls; - Corner separation; - Excessive deformation
-Roof structure failure

STRUCTURAL IMPROVEMENTS AFTER STRENGTHENING:

- Global box type behavior is obtained.
- Detachment and collapse of out of plane walls is controlled.
- Corner separation is restricted.
- Excessive deformation is controlled and hence non-structural elements are less vulnerable.
- Roof structure is well connected to the ring beam and hence the roof failure is also controlled.

STRENGTHENING INTERVENTION DESCRIPTION:

This is an effective type of strengthening which improves the global seismic behavior of a building. The horizontal beam is installed over all the walls (including over the gables) so as to form a band binding all the LBM walls. RC material is usually used for these beams but timber or steel beams can also be used.
Alternatively, seismic belts (a binding layer running through the periphery of the walls) can also be used to improve the box like behavior. For more details on seismic belts installation, refer to UNESCO (2007), NRA (2016).

ILLUSTRATIVE FIGURES:

RC Band Beams

Seismic strengthening of an UCM-URM building using RC ring beams.

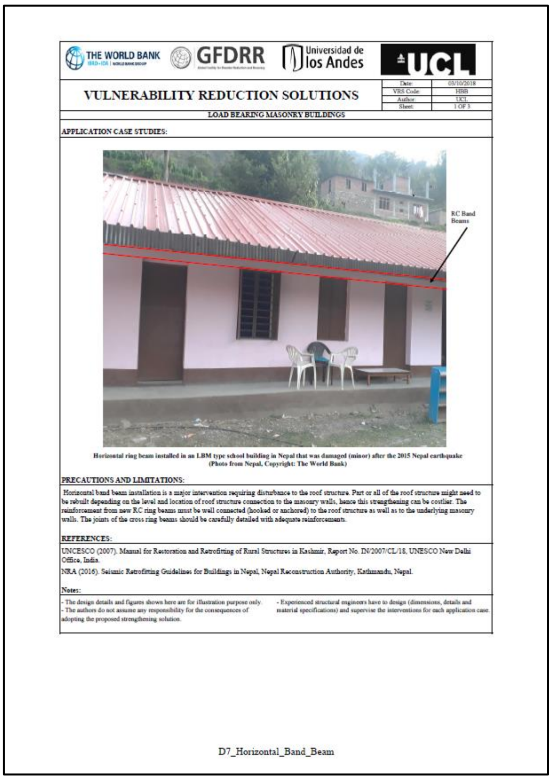
Seismic belts for pitched roof building

Seismic belt for slab roof building

Seismic strengthening of an UCM-URM building using seismic belts. (UNSECO, 2007).

Example Illustrations of strengthening of LBM school buildings using horizontal band beams or seismic belt techniques.

D7_Horizontal_Band_Beam



VULNERABILITY REDUCTION SOLUTIONS

Date:	03/10/2018
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Sheet:	1 OF 3

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APPLICATION CASE STUDIES:

RC Band Beams

Horizontal ring beam installed in an LBM type school building in Nepal that was damaged (minor) after the 2015 Nepal earthquake (Photo from Nepal, Copyright: The World Bank)

PRECAUTIONS AND LIMITATIONS:

Horizontal band beam installation is a major intervention requiring disturbance to the roof structure. Part or all of the roof structure might need to be rebuilt depending on the level and location of roof structure connection to the masonry walls, hence this strengthening can be costlier. The reinforcement from new RC ring beams must be well connected (hooked or anchored) to the roof structure as well as to the underlying masonry walls. The joints of the cross ring beams should be carefully detailed with adequate reinforcements.

REFERENCES:

UNCESCO (2007). Manual for Restoration and Retrofitting of Rural Structures in Kashmir, Report No. IN/2007/CL/18, UNESCO New Delhi Office, India.

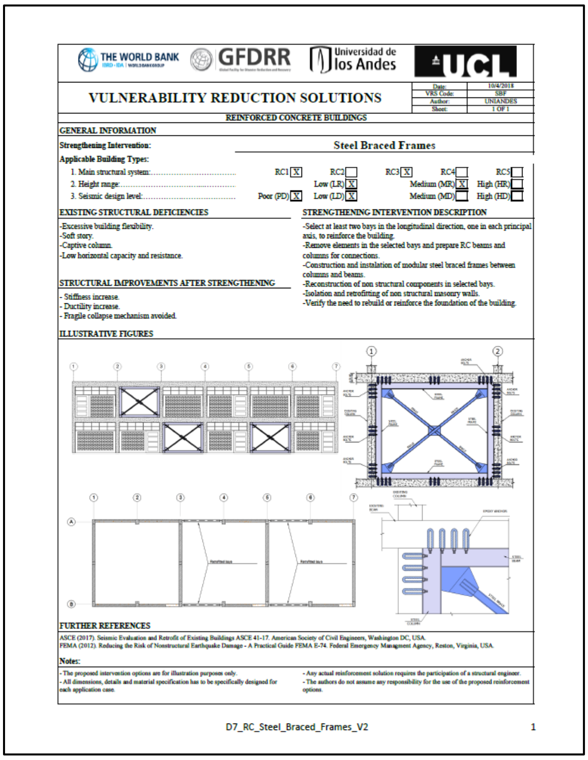
NRA (2016). Seismic Retrofitting Guidelines for Buildings in Nepal, Nepal Reconstruction Authority, Kathmandu, Nepal.

Notes:

- The design details and figures shown here are for illustration purpose only.
- The authors do not assume any responsibility for the consequences of adopting the proposed strengthening solution.
- Experienced structural engineers have to design (dimensions, details and material specifications) and supervise the interventions for each application case.

D7_Horizontal_Band_Beam

Figure 5-1. LBM Vulnerability Reduction Solution Example



VULNERABILITY REDUCTION SOLUTIONS

Date:	10/4/2018
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Author:	UNIANDES
Sheet:	1 OF 1

REINFORCED CONCRETE BUILDINGS

GENERAL INFORMATION

Strengthening Intervention: Steel Braced Frames

Applicable Building Types:

1. Main structural system: RC1 [X] RC2 [ ] RC3 [X] RC4 [ ] RC5 [ ]
2. Height range: Low (LR) [X] Medium (MR) [X] High (HR) [ ]
3. Seismic design level: Poor (PD) [X] Low (LD) [X] Medium (MD) [ ] High (HD) [ ]

EXISTING STRUCTURAL DEFICIENCIES

-Excessive building flexibility.
-Soft story.
-Captive column.
-Low horizontal capacity and resistance.

STRUCTURAL IMPROVEMENTS AFTER STRENGTHENING

- Stiffness increase.
- Ductility increase.
- Fragile collapse mechanism avoided.

STRENGTHENING INTERVENTION DESCRIPTION

-Select at least two bays in the longitudinal direction, one in each principal axis, to reinforce the building.
-Remove elements in the selected bays and prepare RC beams and columns for connections.
-Construction and instalation of modular steel braced frames between columns and beams.
-Reconstruction of non structural components in selected bays.
-Isolation and retrofitting of non structural masonry walls.
-Verify the need to rebuild or reinforce the foundation of the building.

ILLUSTRATIVE FIGURES

FURTHER REFERENCES

ASCE (2017). Seismic Evaluation and Retrofit of Existing Buildings ASCE 41-17. American Society of Civil Engineers, Washington DC, USA.
FEMA (2012). Reducing the Risk of Nonstructural Earthquake Damage - A Practical Guide FEMA E-74. Federal Emergency Managment Agency, Reston, Virginia, USA.

Notes:

- The proposed intervention options are for illustration purposes only.
- All dimensions, details and material specification has to be specifically designed for each application case.
- Any actual reinforcement solution requires the participation of a structural engineer.
- The authors do not assume any responsibility for the use of the proposed reinforcement options.

D7_RC_Steel_Braced_Frames_V2 1

Figure 5-2. RC Vulnerability Reduction Solution Example

It shall be noted that the proposed intervention options are for illustration purposes only. All dimensions, details and material specification have to be specifically designed for each application case. Any actual strengthening solution requires the participation and supervision of a structural engineer. The authors do not assume any responsibility for the use of the proposed strengthening options.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General conclusions

The main aim of the Global Program for Safer Schools (GPSS) of the Global Framework for Disaster Reduction and Recovery (GFDRR) of the World Bank is to make the educational facilities and the communities they serve more resilient to natural hazards. Within the program two main components have been developed:

a) A collaboration platform at global level that enables sharing of risk information and measuring progress towards comprehensive safety of school facilities.

b) The Global Library of School infrastructure GLoSI which serves as a repository of data and information about the structural performance of school buildings and alternatives to reduce their seismic vulnerability.

For the implementation of the platform and the global library, the following components have been defined and implemented in order to support the GPSS activities in countries or regions worldwide:

a) A global building taxonomy for different construction types of schools. This taxonomy shall serve to have a common language for seismic vulnerability and risk communication with respect to school infrastructure, identify the distinct global construction types, rank the vulnerability parameters from the generic to the specific to define and characterize the seismic response, identify and describe the various taxonomy parameters (and their associated attributes) that affect the seismic performance of LBM and RC framed school buildings. Pre-disaster and Post-disaster data collection forms (PRE-DCF and POST-DCF) are developed along with manuals to collect data and information on schools that are required for classification of school buildings.

b) A set of index buildings which represent predominant building typologies within the complete portfolio of school buildings worldwide. The seismic behavior of each index building represents a group of buildings, with uniquely defined geometry, loads, materials, characteristics and dynamic behavior. The seismic vulnerability assessment of school buildings in a given portfolio would be represented by the collection of index buildings representative of the building inventory. All the relevant information for each IB identified is summarized in a format in order to facilitate the sharing and selection of important information.

c) A methodological approach to derive the seismic fragility and vulnerability (F/V) of selected school index buildings (IBs). Considering that each IB represents a typology that can be found in several countries, a reliable analytical assessment of its expected seismic performance is an important contribution towards a robust seismic risk assessment process in any particular country or region worldwide.

d) A database of seismic F/V functions for representative IBs of different school building types in the Global Library of School Infrastructure (GLoSI). A total of 37 F/V functions are included in the GLoSI and ready to be used in risk assessment projects worldwide.

e) Preliminary database of vulnerability reduction solutions. For some IB in the GLoSI, the main weaknesses are identified, and typical practical retrofitting or reinforcement options are schematized. Those would represent possible economical vulnerability reduction solutions which could conform the bases for a risk mitigation plan for the school infrastructure.

6.2 Opportunities for future developments

The following are the opportunities identified for future developments and the recommendations for the following phases of the GPSS:

a) The current classification system shall be expanded to include other very common constructions typologies such as timber, steel frames, adobe and earthen construction, prefabricated and other type of constructions. These were not included in the first phase as proportionally less common worldwide.

b) The proposed Index Buildings are based on the different Building Types found in the different country databases of school buildings. The catalogue intends to represent the most common school building types for different countries. There are usually different structural systems, construction typologies and sensitive variations in the attributes of the secondary parameters within a country or more commonly across different countries which demands the consideration of more than one index building per building type. There is an opportunity through the GPSS Technical Assistance engagement at national level, to enrich the repository with more detailed data on statistics of school infrastructure and choices for more representative index buildings.

c) Additionally, the selected Index Buildings are focused only on LBM and RC school construction types. Timber or steel framed structures should be included in the future. Also, specific work should be done country by country, analyzing each portfolio to identify the dominant building types and their main characteristics. According to this, when those parameters correspond to one particular Index Building from the GLoSI database, the particular fragility/vulnerability (F/V) function could be used directly. If no correspondence is found, a new independent and complete F/V assessment should be done for those particular building typologies.

d) Sensitivity analysis are required in order to:
   a. Completely define the range of applicability of each index building in relation to geometric characteristics, section and material properties, number of stories, seismic hazard characteristics and others.

b. Establish the need to include new relevant parameters and associated attributes or redefine some of them in the defined taxonomy in order to improve the diagnosis of the expected seismic behavior of each group of buildings.

e) Vulnerability reduction solutions are to be expanded and complemented. Typical intervention options could be proposed and taken to the detailed design level. In that way, more specific information could be provided in the GLoSI such as a complete reinforcement option case study, architectonic and structural drawings, detailed budget estimation, technical specifications, recommended constructions sequences and other related information.

f) Technical Guidelines are to be designed, published and disseminated through practical workshops in relation to the following main aspects:

a. PRE-DCF to populate school building databases.
b. School buildings exposure data bases.
c. Vulnerability assessment methodology and specific applications.
d. Risk assessment processes and applications in DRM
e. Post-DCF to assess building safety after an earthquake and provide guidance on evacuation, further inspections or repair and retrofit.

g) Additional topics that are recommended to be included in the GPSS agenda are the following:
a. Methodological approach and database for damage assessment of school constructions after significant earthquakes.
b. Vulnerability and risk assessment validation program.
c. Financial protection programs and options for the school infrastructure
d. Emergency response in case of disaster including contingency plans, evacuation of school buildings, alarm systems, etc.
e. Reconstruction programs (temporary constructions) or new developments (permanent constructions) in the school sector: several typical new constructions models in different materials, geometries, seismic hazard locations, etc. could be designed and documented to promote new construction at low costs.
f. Multicriteria approaches for decision making in relation to selection of the best possible intervention measures and prioritization considering budget availability, structural aspects, non-structural components, and functional aspects such as sanitary conditions, comfort, emergency response, architectural considerations and others.

REFERENCES

ABAQUS (2013) Finite Element Analysis (Theory manual), Dassault Systèmes Simulia Corporation, Providence, USA.

Abrams, D. P. et al. (2017) Out-of-Plane Seismic Response of Unreinforced Masonry Walls: Conceptual Discussion, Research Needs, and Modeling Issues, *International Journal of Architectural Heritage*. Taylor & Francis, 11(1), pp. 22–30. doi: 10.1080/15583058.2016.1238977.

ASCE (2013) ASCE/SEI 41-13: Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers, Virginia, USA.

Applied Technology Council (1985) *ATC-13: Earthquake Damage Evaluation Data for California*, Applied Technology Council, California, USA.

Arya, S., Boen, T., & Ishiyama, Y. (2013) Guidelines for Earthquake Resistant Non-Engineered Construction, UNESCO, France.

ARUP (2015) *Global Program for Safer Schools: Structural Typologies*, The World Bank Report, ARUP, London.

ARUP (2017) *Measuring Seismic Risk in Kyrgyz Republic*, The World Bank Report, ARUP, London.

ASI (2018) Extreme Loading for Structures, Applied Science International, LLC, North Carolina, USA.

ATC (1985) ATC-13 *Earthquake Damage Evaluation Data for California*. Applied Technology Council, USA.

Augenti, N. and Parisi, F., (2010) Learning from construction failures due to the 2009 L'Aquila, Italy, earthquake. *Journal of Performance of Constructed Facilities*, 24(6), pp.536-555.

Bhagat S. Samith Buddika, H.A.D., Adhikari, R.K., Shrestha, A., Bajracharya, S., Singh, J., Maharjan, R., Wijeyewickrema, A.C. (2017) Damage to Cultural Heritage Structures and Buildings Due to the 2015 Nepal Gorkha Earthquake, *Journal of Earthquake Engineering*, pp. 1–20. doi: 10.1080/13632469.2017.1309608.

Bothara, J.K., Dhakal, R.P. and Mander, J.B., (2010) Seismic performance of an unreinforced masonry building: An experimental investigation. *Earthquake Engineering & Structural Dynamics*, 39(1), pp.45-68.

Bothara, J. K., Guragain, R., and Dixit, A., (2002) Protection of Educational Buildings against Earthquakes: A Manual for Designers and Builders. National Society for Earthquake Technology-Nepal, Kathmandu, Nepal.

Brzev, S., Scawthorn, C., Charleson, A.W., Allen, L., Greene, M., Jaiswal, K.S. and Silva, V., (2013) *GEM Building Taxonomy (Version 2.0).* GEM Technical Report 2013-02, GEM Foundation.

CEN (2004) EC 8: Design of structures for earthquake resistance—Part 1: General rules, seismic actions and rules for buildings (EN 1998-1: 2004).

Coburn, A. and Spence, R., (2003) Earthquake Protection. John Wiley & Sons.

Chopra Anil, K. (1995) *Dynamics of structures: Theory and Application to Earthquake Engineering.* Prentice Hall, New Jersey, USA.

D'Ayala, D., Meslem, A., Vamvatsikos, D., Porter, K., Rossetto, T. (2015) Guidelines for Analytical Vulnerability Assessment - Low/Mid-Rise. *GEM Technical Report*, GEM Foundation, Pavia, Italy.

D'Ayala, D., Spence, R., Oliveira, C. and Pomonis, A., (1997) Earthquake loss estimation for Europe's historic town centres. *Earthquake Spectra*, 13(4), pp.773-793.

D'Ayala, D.F., (2008) Numerical Modelling of Masonry Structures, *Structures & Construction in Historic Building Conservation*, pp. 151–172. doi: 10.1002/9780470691816.ch9.

D'Ayala, D.F. and Paganoni, S., (2011) Assessment and analysis of damage in L'Aquila historic city centre after 6th April 2009. Bulletin of Earthquake Engineering, 9(1), pp.81-104.

Decanini, L., De Sortis, A., Goretti, A., Langenbach, R., Mollaioli, F. and Rasulo, A., (2004) Performance of masonry buildings during the 2002 Molise, Italy, earthquake. Earthquake Spectra, 20(S1), pp.S191-S220.

Decreto Ministeriale del 14/01/2008 (2008) Norme Tecniche per le Costruzioni. Gazzetta Ufficiale della Repubblica Italiana, 29., Rome

De Luca F., Vamvatsikos D., Iervolino I. (2013) Improving Static Pushover Analysis by Optimal Bilinear Fitting of Capacity Curves. In: Papadrakakis M, Fragiadakis M, Plevris V (eds) Computational Methods in Earthquake Engineering. Springer

De Luca F, Vamvatsikos D, Iervolino I (2013b) Near-optimal piecewise linear fits of static pushover capacity curves for equivalent SDOF analysis. Earthq Eng Struct Dyn 42:523–543 . doi: 10.1002/eqe.2225

Dixit, A.M., Yatabe, R., Dahal, R.K. and Bhandary, N.P., (2014) Public School Earthquake Safety Program in Nepal. Geomatics, Natural Hazards and Risk, 5(4), pp.293-319.

Erberik, M.A. (2008) Generation of fragility curves for Turkish masonry buildings considering in-plane failure modes. Earthquake Engineering & Structural Dynamics, 37(3), pp.387-405.

ERN (2009) Central American Probabilistic Risk Assessment (CAPRA) User Manual.

Fajfar, P (2000) A nonlinear analysis method for performance-based seismic design. Earthquake Spectra 16.3: 573-592.

FEMA (1997) FEMA 273: NEHRP Guidelines for the seismic rehabilitation of buildings. Federal Emergency Management Agency Washington, D.C., USA.

FEMA (2013) Hazus®-MH 2.1 Advanced Engineering Building Module (AEBM), Technical and User's Manual, Federal Emergency Management Agency, Washington, D.C., USA.

FEMA (2005) Improvement of nonlinear static seismic analysis procedures, FEMA 440. Washington, D.C.

FEMA (2012) Hazus – MH 2.1 Technical Manual, Washington D.C.

Grünthal, G., (1998) European macroseismic scale 1998. European Seismological Commission (ESC).

Guragain, R. (2015), Development of Earthquake Risk Assessment System for Nepal, PhD thesis, The Universtiy of Tokyo, Japan.

GEM (2018) The OpenQuake-engine User Manual. Global Earthquake Model (GEM) Open-

Quake Manual for Engine version 3.1.0. doi: 10.13117/GEM.OPENQUAKE.MAN.ENGINE.3.1.0, 198 pages.

Jaiswal, K.S. and Wald, D.J., (2008) *Creating a Global Building Inventory for Earthquake Loss Assessment and Risk Management*. U.S. Geological Survey Open-File Report 2008-1160, 103 p.

Jaiswal, K., Wald, D. and D'Ayala, D., (2011) Developing empirical collapse fragility functions for global building types. *Earthquake Spectra*, 27(3), pp.775-795.

Kaplan et al., (2008) 29 October 2007, Çameli earthquake and structural damages at unreinforced masonry buildings. Natural Hazards and Earth System Sciences, 8(4), pp.919-926.

Karbassi, A. and Nollet, M. J. (2013) Performance-based seismic vulnerability evaluation of masonry buildings using applied element method in a nonlinear dynamic-based analytical procedure, *Earthquake Spectra*, 29(2), pp. 399–426. doi: 10.1193/1.4000148.

Lang, K., (2002) Seismic Vulnerability of Existing Buildings, Doctoral Thesis, Swiss Federal Institute of Technology Zurich, Switzerland.

Lagomarsino, S., Penna, A., Galasco, A. and Cattari, S., (2013) TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings. *Engineering Structures*, *56*, pp.1787-1799.

Maganes, G. (2006) Masonry Building Design in Seismic Areas: Recent Experiences and Prospects from a European Standpoint, First European Conference on Earthquake Engineering and Seismology, Geneva, Switzerland

MARN (2012) *Modelacion Probabilistia de Escenarios de Reisgo Sismico Para el Area Metropolitana de San Salvador, Incluye Analysis de Los Portafolios de Educacion, Salud y Gobierno*. Ministerio de Medio Ambiente y Recursos Naturales, El Salvador. (In Spanish)

Meguro, K., and Tagel-Din, H. (2000) Applied Element Method for Structural Analysis, Doboku Gakkai Ronbunshu, 2000(647), 31-45.

Moon, L., Dizhur, D., Senaldi, I., Derakhshan, H., Griffith, M., Magenes, G., & Ingham, J. (2014) The demise of the URM building stock in Christchurch during the 2010–2011 Canterbury earthquake sequence. Earthquake Spectra, 30(1), 253-276.

Mouroux, P., and Benoit, L.B. (2006) Presentation of RISK-UE Project, *Bulletin of Earthquake Engineering*, 4:323-339.

Murty, C.V.R. (2003) How do Brick Masonry Houses behave during Earthquakes? IITK-BMTPC Earthquake Tip 12, Indian Institute of Technology Kanpur, Kanpur, India

NBC (1994) *NBC 203: 1994 Guidelines for Earthquake Resistant Building Construction: Low Strength Masonry*, Nepal National Building Code, Department of Urban Development and Building Construction, Kathmandu, Nepal.

Meli et al., (2011) *Seismic Design Guide for Low-Rise Confined Masonry Buildings*, The Confined Masonry Network, EERI & IAEE.

Pandey, B. H. and Meguro, K. (2004) Simulation of brick masonry wall behavior under in-plane lateral loading using applied element method, *13th World conference on earthquake engineering, Vancouver, BC, Canada, August*, pp. 1–6.

Pannell, D. J. (1997) Sensitivity analysis of normative economic models: theoretical framework and practical strategies. Agricultural economics, 16(2), 139-152.

Penna, A., Morandi, P., Rota, M., Manzini, C. F., Da Porto, F., & Magenes, G. (2014) Performance of masonry buildings during the Emilia 2012 earthquake. Bulletin of Earthquake Engineering, 12(5), 2255-2273.

Paquette, J. and Bruneau, M., (2003) Pseudo-dynamic testing of unreinforced masonry building with flexible diaphragm. Journal of structural engineering, 129(6), pp.708-716.

Rodgers, J.E. (2012) Why schools are vulnerable to earthquakes. In Proceedings, *15th world conference on earthquake engineering*, Lisbon (pp. 24-28).

Rossetto T., D'Ayala D., Ioannou I., Meslem A. (2014) Evaluation of Existing Fragility Curves. In: Pitilakis K., Crowley H., Kaynia A. (eds) SYNER-G: Typology Definition and Fragility Functions for Physical Elements at Seismic Risk. Geotechnical, Geological and Earthquake Engineering, vol 27. Springer, Dordrecht

SIDA (2016) *Structural Integrity and Damage Assessment for Educational Infrastructure in Nepal: Results and Findings (Phase I and II)*, The World and Report.

Shome, N. and Cornell, C. A.. (1999) Probabilistic Seismic Demand Analysis of Nonlinear Structures. Reliability of Marine Structures Program Technical Report RMS-35. Stanford Digital Repository. Available at: http://purl.stanford.edu/qp089qb1141

Yamin, L.E., Rincón, J.R., Hurtado, Á.I., Becerra, A.F., Reyes J.C., López L.L., Dorado, J.F., Estrada, J., Cortez, F.R., Atoche J.C., and Obando, L.D. (2015) *la Evaluación del Riesgo Sísmico del Portafolio de Edificaciones En Locales Escolares del Perú a Partir De Los Resultados del CIE.* Universidad de los Andes, Bogota, Columbia. (In Spanish)

Yamin, L., Hurtado, A., Barbat, A. H., & Cardona, O. D. (2014) Seismic and wind vulnerability assesment for the GAR-13 global risk assessment. *International Journal of Disaster Risk Reduction*, *10*, 425–460.

Yamin, L. E., Hurtado, A., Rincon, R., Dorado, J. F., & Reyes, J. C. (2017) Probabilistic seismic vulnerability assessment of buildings in terms of economic losses. Engineering Structures, 138, 308-323.

ANNEX A. CONSTRUCTION TYPOLOGIES FOR BUILDING CLASSIFICATION IN DIFFERENT COUNTRIES

Identification and comparison of national level construction typologies from different regions and countries is an important step to develop an internationally applicable taxonomy for the building classification of school infrastructure. Based on the database on school buildings collected by the World Bank in different countries, country-wise construction types of LBM and RC school buildings are presented below. This Annex shall be complemented in future projects as the GPSS programs is adopted in other countries worldwide.

Nepal

The main construction types of LBM school buildings present in Nepal according to the SIDA database (SIDA, 2016) are identified as: unconfined/unreinforced masonry walled buildings and masonry infilled light steel framed structures. A very small percentage of adobe schools with sun dried bricks are also present.

The masonry element varies from rubble stone, dressed stone, bricks to concrete blocks while the mortar is either mud or cement sand mortar. The masonry walls of the masonry infilled light steel framed buildings are critical to seismic damage compared to the steel frames, as observed in the 2015 Nepal earthquakes. Many people believe these buildings to be steel framed structures, overlooking the relative size, mass and stiffness of the load bearing masonry wall system and the light steel frame supporting the roof structure. The seismic behavior of the buildings is mainly governed by the failure mechanisms associated to the masonry walls. Hence these school buildings type should be classified into LBM building type rather than steel framed structures.

On the other hand, the RC buildings represents the 10.5% of the total school buildings. These buildings are mostly single or two-story non-engineered constructions (SIDA, 2016).

El Salvador

Different construction types or LBM School buildings in San Salvador city of El Salvador (MARN, 2012) are identified as: confined masonry construction, reinforced masonry construction, light steel framed structures with confined/reinforced masonry infill walls. A very small percentage of school buildings with unconfined/unreinforced masonry walls are also present. The masonry element varies from burnt clay bricks (in older constructions) to hollow concrete blocks (in reinforced masonry construction).

On the other hand, RC buildings are found but in a very low percentage. Most common RC structural system are moment resisting RC frames with masonry infills generating short column effect, which represent less than the 8% of the entire portfolio.

Peru

Major construction types of LBM and RC school buildings in Peru (CIE, 2013) are identified as: adobe, confined masonry construction, reinforced masonry construction. A very small percentage of school buildings with unconfined/unreinforced masonry walls are also present. The masonry element varies from burnt clay bricks (in older constructions) to hollow concrete blocks (in reinforced masonry construction).

The most common RC building type is known as 780PRE type. This kind of building is a moment resisting RC framed structure with masonry infills usually generating short column effects. There also exists another building type named 780POST type which corrected the short column problem by means of isolating the masonry infills and dimensioning stiffer columns.

Kyrgyz Republic

Major construction types of LBM school buildings in Kyrgyz Republic (ARUP, 2017) are identified as: adobe buildings with sun dried adobe bricks and unconfined/unreinforced masonry construction with stone or brick as masonry units.

Similarly, typically found construction types of RC school buildings are RC frame with masonry infill walls and RC walled buildings.

Thus, the review of the information on construction types of school buildings from these four different countries reveals that the global LBM construction types are:

- Adobe (A)
- Unconfined/unreinforced masonry (UCM-URM)
- Confined masonry (CM)
- Reinforced masonry (RM)
- Light steel framed structures with LBM masonry walls (SFM).

Similarly, the global construction types for RC school buildings are identified as:

- RC moment resisting frame (RC1)
- RC moment resisting frame with masonry infills (RC2)
- RC moment resisting frame with masonry infills that generate short column (RC3)
- Combined structural systems as RC walls and frames (RC4)
- Non-engineered RC constructions (RC5).

ANNEX B: PRE -DISASTER DATA COLLECTION FORM

Pre-disaster data collection form and manual.

ANNEX C. POST-DISASTER DATA COLLECTION FORM

Post-disaster data collection form and manual.

ANNEX D. GLoSI TAXONOMY

Excel sheet for GLoSI Taxonomy

ANNEX E. BUILDING TYPE CATALOG

List of building types found in different case study countries (Microsoft word document format)

ANNEX F. GLoSI INDEX BUILDING CATALOGUE

Index buildings catalog for LBM and RC building types.

ANNEX G. SEISMIC RECORDS FOR FRAGILITY/VULNERABILITY ASSESSMENT

List of seismic records (ground motions) for F/V assessment.

Digital format.

ANNEX H. SOFTWARE: N2 METHOD – ENGINEERING DEMAND PARAMETERS CALCULATION

A Excel® based datasheet has been developed for the calculation of engineering demand parameters, given a capacity curve resulting from the 3D non –linear model and will be made available in GLoSI library, at CAPRA website (www.ecapra.org) and at the UCL EPICentre website (https://www.ucl.ac.uk/epicentre)

ANNEX I. SOFTWARE: LEAST SQUARE METHOD FOR FRAGILITY ASSESSMENT

A MATLAB® based software package has been developed for the calculation of fragility functions, given a collection of EDP resulting from the seismic performance assessment at different intensity levels and will be made available in GLoSI library, at CAPRA website (www.ecapra.org) and at the UCL EPICentre website (https://www.ucl.ac.uk/epicentre)

ANNEX J. SOFTWARE: FUNVUL V2.0 FOR COMPONENT BASED VULNERABILITY ASSESSMENT

Digital Annex. Software **IT-Funvul V2.0** (Yamin et al., 2017), available at www.ecapra.org

ANNEX K. CATALOG OF FRAGILITY/VULNERABILITY ASSESSMENT RESULTS

Catalog of fragility/vulnerability assessment results for different LBM and RC IBs.

ANNEX L. VULNERABILITY REDUCTIONS SOLUTIONS

Catalog of vulnerability reduction solutions for different LBM and RC IBs.