Seismic Performance Assessment of Low-Rise Unreinforced Stone and Brick Masonry School Buildings in the district of Kashmir, Pakistan.

Submitted by

Ali Murtaza

MS Structural Engineering (Fall 2018)

00000277388

Supervisor

Dr. Muhammad Usman

NUST Institute of Civil Engineering (NICE)

School of Civil and Environmental Engineering (SCEE),

National University of Sciences and Technology (NUST), Islamabad

August 2022

Seismic Performance Assessment of Low-Rise Unreinforced Stone and Brick Masonry School Buildings in the district of Kashmir, Pakistan.

Submitted By

Ali Murtaza

00000277388

A thesis submitted to the NUST Institute of Civil Engineering in partial fulfillment of the

Requirements for the degree of

MASTER OF SCIENCE

in

STRUCTURAL ENGINEERING

Thesis Supervisor:

Dr. Muhammad Usman

Thesis Supervisor's Signature

Department of Structural Engineering

NUST Institute of Civil Engineering (NICE),

School of Civil and Environmental Engineering (SCEE),

National University of Sciences and Technology (NUST),

Islamabad, Pakistan

(August 2022)

Thesis Acceptance Certificate

I certify that final copy of MS thesis written by Mr. Ali Murtaza (Registration No.00000277388) of NICE (SCEE) has been verified by undersigned, found complete in all respects as per NUST Statutes/Regulations, is free of plagiarism, errors and mistakes and is accepted as partial fulfillment for award of MS degree.

Signature: ________________________

Supervisor: Dr. Muhammad Usman

Date: ________________________

HOD Signature: ________________________

Date: ________________________

Dean Signature: ________________________

Date: ________________________

Declaration

I certify that this research work titled "*Seismic Performance Assessment of Low-Rise Unreinforced Stone and Brick Masonry School Buildings in the district of Kashmir, Pakistan.*" is my own work. The work has not been presented elsewhere for assessment. The material that has been used from other sources it has been properly acknowledged / referred.

Signature of Student

Ali Murtaza

00000277388

Copyright Statement

Acknowledgments

All thanks to Almighty ALLAH, Who granted me the capacity and strength to accomplish the MS research thesis, and all respects to His Prophet Muhammad (PBUH, May ALLAH's Blessings and Salutations Be Upon Him).

I owe a debt of gratitude to Dr. Muhammad Usman, my project supervisor, who assisted me during the compilation of this research project. Her encouragement, direction, and kind words have always inspired me to work with dedication. Whenever I found myself in any sort of trouble, I always found her available to cater the issue. This research was sole idea of her sublime thoughts. She supported me in each and every circumstance during this research whether it was financial support or technical support. Without her support, the completion of this innovative project was not possible. She was very kind and cool mind towards me throughout this research. I am once again really grateful to her.

I am grateful to my Parents for their special prayers that helped me to achieve this tedious task in the compilation of this research project.

In the end, I am also thankful to **Mr. Zain** who assisted me in Non-linear Modelling and use of 3-MURI software from day one till last day. I am also thankful to **Mr. Zubair Bashir** who assisted me with his experience in Structural Dynamics and Modeling Techniques. I pay my regards to my GEC members for their utmost support and guidance throughout this research project.

Regards,

Ali Murtaza

Abstract

The performance of unreinforced stone and brick masonry buildings in a seismically active region are considered to be highly vulnerable if proper construction practices and detailing are not implemented. The illustration of the fragile performance of URM buildings especially in event of the 2005 earthquake in Kashmir, Pakistan cannot be disregarded. According to a survey by the Engineering Research Institute, around 67% of educational institutions in the region were destroyed, resulting in hundreds of deaths and enormous infrastructure and economic losses. In this context, the paper is focused on the seismic performance evaluation of URM stone and brick school buildings located in Kashmir. The database of existing URM school buildings of Kashmir was utilized which represents zone 04, the seismically most active zone in Pakistan. Two URM stone and brick school buildings are considered typological representative school building stock in the region. The case study buildings were modeled in 3 MURI software based on the equivalent frame method approach and a non-linear static analysis was performed to assess their seismic performance. The performance limit states were defined on a capacity curve of the model according to Euro-code criteria. The performance-based assessment by the N2 method was carried out in 3-Muri software to evaluate the performance points and assess the seismic strengthening of stone masonry school buildings. In addition to that, a set of seismic strengthening measures were adopted in the original models to determine the improvement in the global behavior of the structures.

Keywords: *Unreinforced stone and brick masonry school buildings, Seismic Performance, Equivalent Fame Modeling, Seismic safety measures, Static push over analysis, Performance based assessment.*

Table of Contents

List of Figures

List of Tables

List of Abbreviations/ Symbols

URM	Unreinforced masonry
SPO	Static push over analysis
B.M	Bending moment
S.F	Shear force
DL	Limited damage
SD	Significant damage
NC	Near Collapse
M- θ	Moment rotation curve
K	Lateral Stiffness
Ko	Normalised initial stiffness
V	Shear force
∂	Drift
M_{Rd}	Flexural Moment
N	Axial Force
σ_n	Normal Stress
f_d	Design Compressive Strength of masonry
l_c	Length of compressed portion of masonry panel transversal section
t	Thickness of panel

f_{vd}	Shear strength of panel
c	Cohesion of mortar joint
μ	Friction coefficient
fbt	Tensile strength of bricks
μc	Equivalent tensile strength of spandrels
φ	Coefficient of internal friction
lt	Length of transversal section of panel

Chapter 1

Introduction

1.1 Background

Masonry structural type is pervasive in all parts of the world. It is one of the earliest structural systems and is still widely used in historic structures such as Egyptian pyramids, temples, caverns, mosques, heritage buildings, residences, schools, and hospitals, among others. Prior to the nineteenth century, the most popular construction material was unreinforced masonry (URM), which was replaced by concrete, steel, and wood [1]. URM is still one of the most prevalent building materials in developing nations. In Pakistan, unreinforced masonry constitutes a major portion of the existing building stock. It include practically all residential homes and the vast majority of educational buildings. Local availability of materials, low cost, effective fire resistance, thermal insulation, and basic building techniques are the primary reasons for its widespread use. The performance of unreinforced masonry under gravity loads is sufficient, but it is extremely sensitive and weak when subjected to lateral pressures, particularly earthquakes [2]. Numerous earthquakes in the past have demonstrated that URM structures are ineffective. The Bhuj Earthquake (2001) [3], the Kashmir Earthquake (2005) [4], and the Ankara Earthquake (2007) all caused substantial loss of life and property [5]. Due to its brittleness, low tensile strength, heterogeneous and anisotropic behavior, and absence of an appropriate technical design, unreinforced masonry performs poorly against seismic action.

The mechanical behavior of masonry is complicated, and its shape, unit type, and mortar quality vary significantly throughout the world. This inconsistency makes it more difficult to design and adapt masonry buildings. Despite the pervasive construction of masonry around the world, its behavior is little understood, and it is categorized as a "natural material." In previous years, there has been an increase in study interest, particularly in analyzing the seismic susceptibility of masonry buildings, and a number of global projects have been conducted to estimate the seismic response of masonry structures. Masonry is now widely recognized as a robust building material, particularly in seismic zones. Unreinforced Masonry (URM) has better compressive strength but inadequate tensile strength, rendering it susceptible to lateral pressures. According to previous seismic measurements, the primary causes of URM structure failure are in-plane shear and out-of-plane bending of walls. These primary types of failure are depicted graphically in Figure1-1 [6]

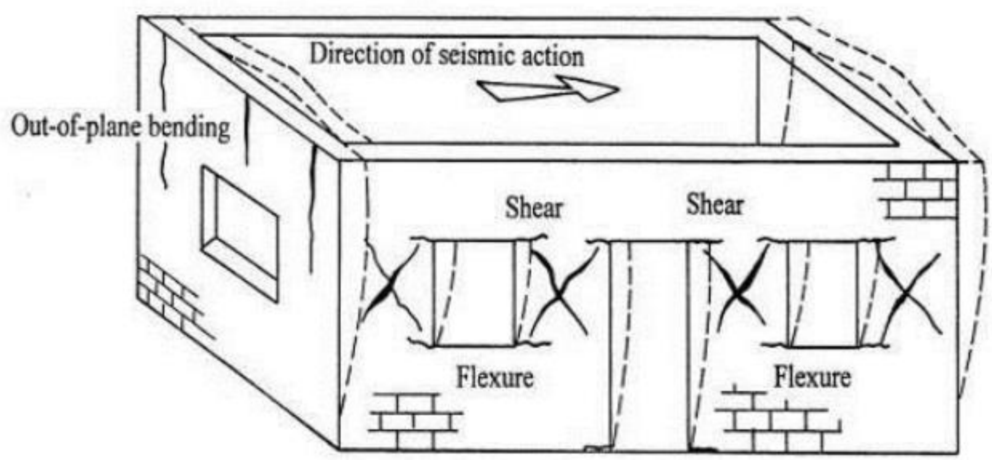


Figure 1- 1: Typical Failure Modes of masonry under seismic loadings [6]

The URM walls are particularly susceptible to out-of-plane movement and require support from orthogonal walls to sustain the seismic inertia force. Solid URM walls, on the other hand, provide adequate strength to withstand lateral loads for low-rise buildings. However, the openings formed by doors and windows lower the cross-sectional area and act as the principal cause of failure. In addition, if openings are placed close to corners, the connection between orthogonal walls, weakened and the entire box's motion is undermined. Due to their susceptibility to B.M and S.F, the piers between openings are the most important structural components for resisting in-plane forces. These piers are subjected to larger pressures than the wall portions above and below the openings. Low-rise buildings may experience sliding shear failure if the normal load perpendicular to the bed joints is less and the masonry offers resistance to sliding failure via friction at the bed joints. The substantial vertical load may cause failure of URM panels into shear. Failure of narrow piers may be brought about by rocking (flexure). Figure 1-2 depicts typical in-plane failures resulting from past earthquakes (Kashmir 2005).

Figure 1- 2: In plane failure of masonry building

Under out-of-plane seismic excitation, walls are subject to inertia forces that are transferred to cross-walls through slab/foundation above and below walls. This inertia force tends walls to fail in out of the plane , which ultimately produces massive tensile stresses and wall failure. This failure pattern is typical in structures with high ceilings and flexible floors/roofs shown in figure below,

Figure 1- 3: Out of plane bending failure of masonry walls

Inadequate roof diaphragm anchoring into masonry walls may also lead to out-of-plane wall failure.

In recent years, several modelling methodologies for existing structures have been developed [7]. Several authors approaches for micro modelling and micro modelling simplification include discretization of unit, mortar, and interface using nonlinear finite element techniques [8,9].These models may be used to represent any sort of masonry construction, but their processing requirements are so great that they are restricted to two-dimensional substructures. Equivalent continuum models with homogenized material [10] are able to apply to any irregular masonry type, however analysis of whole three-dimensional buildings is deemed impracticable due to the required calculation time. Meso-scale modelling, a discrete element modelling technique [11,12], appears as an alternative to finite element approaches. These methods are not applicable because increased model complexity necessitates considerable calculation time. The researcher proposes simplified models such as the equivalent truss model [13] and limit analysis [14] that are useful for practicing engineers and design offices, but are restricted in performance and inappropriate for particular buildings. Equivalent frame modeling (EFM) is the most successful and extensively used approach for evaluating the global behavior of structures [15-19]. Reasonable computing effort, time, and mechanical characteristics are required. International standards propose this method [32-36].

Possible failure modes of masonry buildings include diagonal cracking, shear sliding, and rocking. It may be assumed with good precision that the EFM technique for three-dimensional URM buildings works under a simplified principle in which a box structure is assumed. The behavior is valid if orthogonal wall connections are suitable and the slab above the walls may perform a diaphragm action as a result of strong bonding. In this approach, several formulations have been developed in which damage is associated only in piers using a lumped plasticity approach, while spandrels and rigid nodes are kept elastic [37]. However, other researchers, such as Lagomarsino and Penna, have proposed the 3-Muri software package, in which nonlinearity is assumed for both spandrels and piers [37]. Different researchers use both procedures, despite the fact that there is no significant difference in the outcomes of the two methods. The axial load, aspect ratio, and end condition play significant roles in the behavior of masonry against lateral motion. By adding P-M interaction hinges in piers, a number of studies [38] have evaluated the effect of variable axial stress on masonry panel. As observed by kumar [39], the results are more accurate if a single masonry panel is evaluated, but the modification of axial load has no substantial effect on the reaction of the entire structure in the 3-dimensional global response of the building.

In view of performance based concepts application of non-linear static pushover analysis in the seismic evaluation of R.C.C., steel and masonry buildings are frequently utilized. Researchers have included both pushover and nonlinear time history analysis in the evaluation of unreinforced masonry [40]. According to of Lagomarsino and Pastcier [41], the findings of time history analysis and non-linear static pushover analysis are comparable.

1.2 Problem Statement

Pakistan is particularly considered as seismically vulnerable region in Asia. This region is situated at the junction of three tectonic plate borders, namely the Indian, Eurasian, and Arabian plates. These seismic tectonic settings have positioned Pakistan among the world's most seismically active locations, creating a potential seismic danger for the country [42]. Numerous historical earthquakes of great magnitude in Pakistan have caused loss of life, property, and infrastructure. The 1935 Quetta and 2005 Kashmir earthquakes were the most devastating, claiming tens of thousands of human lives [43].The widespread masonry construction in seismically active part of Pakistan particularly in Kashmir poses a great threat to precious lives and economic rupture of the country. It is imperative to conduct proper seismic performance assessment in existing school building stock and highlight most critical areas and assess the seismic strengthening work to remove the deficiencies in masonry work. To achieve this motive seismic performance assessment with an efficient modeling strategy is required for large building stock in an area.

1.3 Research Objective

- ✓ To develop a non-linear model of representative single story unreinforced stone masonry school building and representative double story unreinforced brick masonry school building.
- ✓ To evaluate the seismic performance assessment of two representative school buildings.
- ✓ Application of seismic strengthening measures to an existing stone-masonry school building and assessment of the building's improved seismic performance.

Dissertation Organization

Chapter 1: This chapter describes the widely used masonry construction throughout the world especially in Pakistan. The construction practices adopted in this type of construction and associated behavior has also been discussed. The complex behavior and different modeling strategies pertaining to model the masonry structures has been elaborated. This chapter also includes the motivation and objectives of this research. In last section is a summary of outline of this thesis how it proceeds.

Chapter 2: This chapter describes the behavior of masonry against gravity and lateral loads. The failure modes of masonry discussed in this section based on the already conducted post-earth quake assessment studies and experimental works. An insight of different modeling techniques related to masonry has been discussed. The limit states proposed by different standards and researcher's corresponding to performance based assessment are also the part of chapter.

Chapter 3: This chapter includes the research methodology adopted for this study. The data base of existing school building stock is presented. The detail of building configuration has been the part of the chapter. The nonlinear modeling technique and performance based assessment adopted for the study is explained comprehensively.

Chapter 4: This chapter describes the results of push over analysis and capacity curves developed. The results of performance based assessment by N2 method is also discussed in this section.

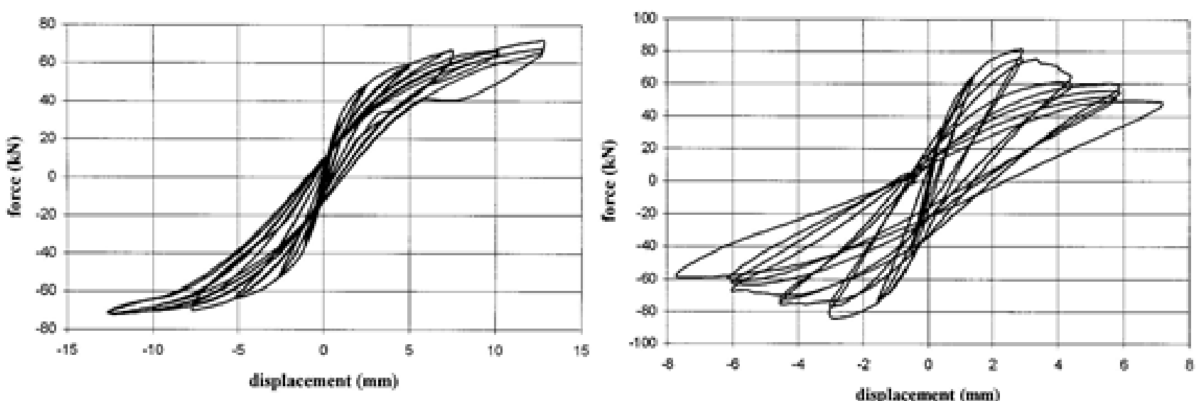
Chapter 5: This chapter contains the conclusion of the study and recommendations pertaining to the work.

Chapter 2

Literature Review

2.1 Response of non-linear behavior of unreinforced walls

Nonlinear behavior of masonry is extremely complex. Cracking and crushing of heterogeneous and anisotropic brickwork is principally responsible for nonlinearity. It depends on the axial load, mechanical properties, aspect ratio, and boundary conditions of the wall. Each mode of failure has unique displacement and strength properties. Before proceeding to the nonlinear behavior of masonry, its failure mechanism must be identified. The hysteretic response of the walls are shown in figure below[44],



a) b)

Figure 2- 1: a) Flexural behavior b) shear behavior against cyclic load patterns [44]

In the case of flexural response, such as the swaying of a pier, the reaction is non-linear elastic and low hysteretic energy dissipation, high displacement capacity, and limited strength degradation, as seen in a). Shear-dominated reactions, such as diagonal tension failure, are characterized by enhanced hysteretic energy dissipation, restricted displacement capacity, and large degradation of strength and stiffness.

2.2 Modes of Failure of Un-Reinforced Masonry Walls

The understanding of non-linear behavior is basically depending on the post-earthquake assessment due to complex behavior of it. The basic four in-plane failure modes of unreinforced masonry walls including bed joint sliding, diagonal tension failure along masonry units , toe crushing and rocking shown in figure below,

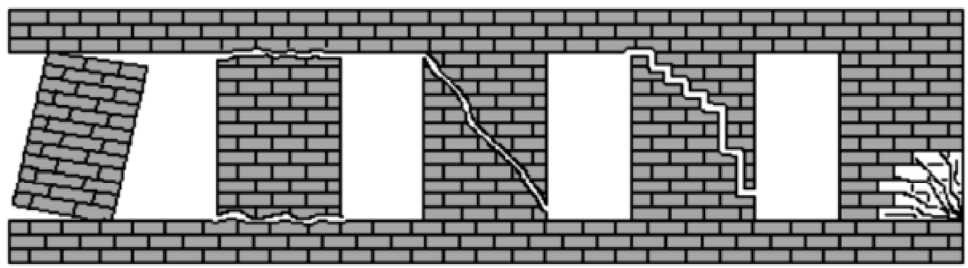


Figure 2- 2: In-plane failure modes of masonry walls against lateral loading [45]

Four independent failure modes are insufficient to describe the inelastic behavior of a masonry wall. Overturning moment owing to lateral loads may increase or decrease axial force on the walls of a URM structure, hence shifting flexural failure to shear failure [45].The eventual collapse of a pier may therefore be seen as result of four primary failure modes defined by FEMA 307 [37].

The aspect ratio and normal stress are the two most influential parameters in predicting unreinforced masonry failure mechanisms. Low levels of axial force and a high aspect ratio influence the rocking and sliding response. These sorts of failure are capable of creating significant ultimate drifts. Failures due to toe-crushing and diagonal stress are more likely when axial loads and aspect ratios are significant. Despite the common belief that these failure modes are brittle, considerable displacement capacities have been found when diagonal fractures develop in a stair-stepped fashion due to the subsequent sliding deformations [46]. Four primary URM wall failure scenarios are defined here, along with their nonlinear response characteristics.

2.2.1 Mechanism of Rocking

Rocking is characterized by the formation of flexural cracks at the base and top of the wall. Under the overturning action of lateral loads, the wall turns firmly in the direction of the horizontal force around the compression zone. During force reversals, large deformations are observed without an apparent decrease in strength, as flexural fractures close and un-cracked sections withstand the overturning moment in each reversal. Despite the fact that some writers [45] consider rocking to be a functioning state and not a state of failure, a displacement-based failure mode with a generalized force-deformation connection has been developed for rocking.

$$M_{Rd} = NB/2*(1- \sigma_n /0.85f_d) \quad (1)$$

$$V_{Rd,flex} = M_{Rd}/h_0 = NB/2h_0(1- \sigma_n /0.85f_d) \quad (2)$$

2.2.2 Shear deformation along bed joints

It is defined by the sliding of bed and head joints in horizontal or stair-stepped pattern. The resistance is provided through friction between masonry units and mortar by relative movement of masonry units against shear stress posed by lateral force. Without a major decrease in strength, significant amounts of energy are lost owing to frictional resistance. Shear strength is determined by the Mohr-Coulomb criteria for sliding shear mechanisms.

$$V_{Rd,s} = l_c t \text{ x } f_{vd} = (c + \mu\ \sigma_n) \text{ x } l_c t \quad (3)$$

2.2.3 Shear mechanism along diagonal cracks

It is a shear failure in which diagonal cracks originate near walls central portion and move towards the corners. When tensile stresses exceeds the masonry’s tensile resistance cracks will appear. If strong bricks and weak mortar are used, bed and head joints will fail; otherwise, fractures would spread diagonally along the bricks and mortar. According to experimental research, cracking over brick units and mortar results in brittle failure with a quick reduction in strength, while the second kind of cracking exhibits rather substantial ultimate drifts. Utilizing the Turnsek and Cacovic criteria, the ultimate shear strength of shear mechanisms with diagonal cracking is determined.

When the major tension stress approaches the tensile strength of the central cross section of the pier, failure is predicted.

$$V_{Rd,d} = f_t B t/\beta \times \sqrt{1+ \sigma_n/f_t} \quad (4)$$

2.3 Maximum Allowable Drift for Unreinforced Masonry Walls

The maximum drift limit is a crucial indication of a wall's deformation capability. The member strength is compared with the demand in forced based approach. In comparison, displacement-based evaluation of structures pushes the structure into a nonlinear range, displacement is considered in this approach. The method in which a masonry wall fails determines its ultimate drift limit. Failures dominated by shear, such as diagonal tension failure, are brittle and have lower ultimate drifts. According to the experimental test data gathered by FEMA 307 [37], Table 2-1 displays the deformation capacity of URM walls for different failure causes.

Table 2- 1: Maximum drift for unreinforced walls proposed in literature

Failure Mode	Ultimate drift (%)	References
Rocking	0.6-1.3	Magenese& Calvi [86]
Rocking/ Toe crushing	0.8	Abrams&Shah [87]
Flexural cracking / Toe crushing / Bed joint sliding	0.8-1.3	Manzouri [88]
Flexural cracking / Diagonal Tension	0.5-0.8	Magenese& Calvi [86,89]
Flexural cracking / Toe crushing	0.2-0.4	Abrams&Shah [87],Epperson and Abrams [89]

According to Magenes and Calvi [51], pure rocking and shear sliding modes are stable and maximum displacement limit for these failure modes is useless because other failure modes, such as diagonal tension and toe crushing determine the limit of displacement prior to the wall failing due to pure rocking or sliding. For instance, in the event of a rocking failure, the ultimate displacement may be restricted to a 10% lateral displacement of the wall height owing to a reduction in strength brought on by P-effects. The drift at final state in diagonal shear failure tends to be a uniform figure with a mean of 0.53 percent and a coefficient of variation of 10 percent, according to experimental data made by Magenes and Calvi.

2.4 Masonry Non-linear Modeling Approach

Various approaches have been utilized in the study unreinforced masonry thus far. The researchers investigate for alternative constitutive models due to the variety and high degree of complexity inherent in masonry. Existing research offers a wide range of numerical methods, from complex micro finite element models to limit analysis approaches. User-friendly and needing fewer data, equivalent frame models are developed. In contrast to FEM models, neither of these approaches can simulate nonlinearity distribution, force redistribution, coupling effect between orthogonal walls, failure mode prediction, etc. Although FEM are the most precise, the ideal approach is "the technique that provides the necessary data with minor error and the lowest cost" [52]. As demonstrated in Figure 2.5, [53] summarizes finite element modelling methodologies described in the literature depending on the amount of structural analysis refinement.

• Micro-modeling involves discrete modelling of brick units and mortar with continuous parts, and mortar unit interaction with discrete elements.

• Simplified micro-modeling entails with continuum elements to model brick units, whilst the behavior of unit-mortar interface and mortar joints is aggregated using discontinuous components.

• Macro-modeling considers units, mortar, and their interaction are dispersed over the continuum. In the sections that follow, each modelling approach, from the most complex to the simplest, will be reviewed in detail.

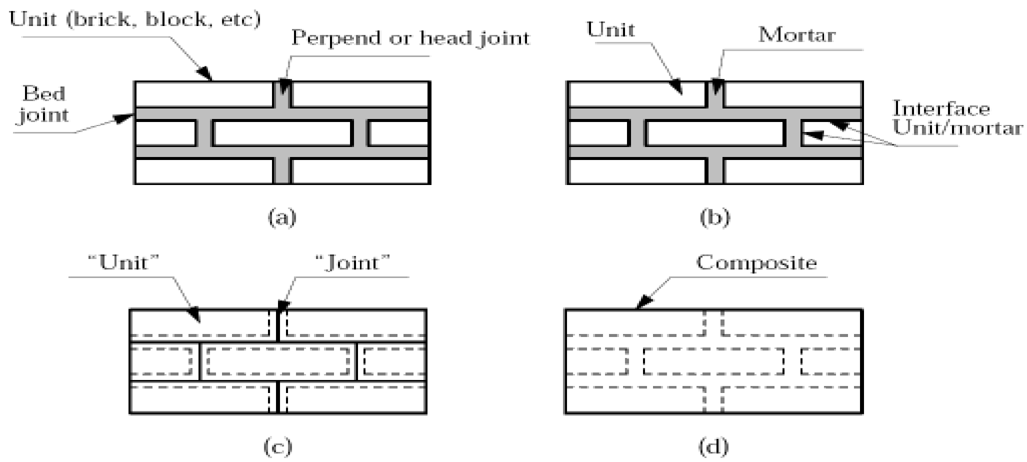


Figure 2-3: Different Modelling Techniques a) Masonry Sample , b) micro modeling with detail, c) micro-modeling with simplified approach , d) macro-modeling [53]

2.4.1 Finite Element Micro-Model

Masonry is the construction of bricks and mortar joints. To effectively depict the complexity of masonry, a complete model must account for the different mechanical characteristics of Mortar joints, brick units and its interaction. Micromodels based on finite elements portray masonry according to the mechanical characteristics of each component and the unit mortar contact, which must be empirically defined. According to Lourencho [53], "the properties of masonry are affected by a various of factors, such as the material properties of the units and mortar, the arrangement of bed and head joints, the anisotropy of the units, the dimensions of the units, the joint width, the quality of workmanship, the degree of curing, the environment, and age."

In addition, FEM micro models require significant computing resources and complex failure criteria [54]. Therefore, this technique is applicable to limited structural components with a variety of stress and strain states. Lourencho [53] concludes, based on a comprehensive analysis of finite element modelling techniques, that "for enormous structures, the memory and time requirements become too great, and if a compromise between accuracy and economy is required, a macro modelling strategy is likely to be more effective."

2.4.2 Finite Element Macro-Model

Given the fact heterogeneous character of local stress distribution has minimal effect on the global response. In modeling and simulating massive structural components, a connection can be developed between average masonry stresses and average masonry strains. In lieu of discrete modelling, a macro finite element model of masonry is constructed by homogenizing brick units and mortar joints. The construction of macro models involves fewer material parameters, which corresponds to less experimental effort, compared to micro models. Prior to developing micro models, it is necessary to test brick units and mortar cubes to evaluate the mechanical properties of masonry components and the brick mortar interface. While testing conducted on suitably sized composite materials are sufficient for characterizing the mechanical characteristics of URM walls in order to develop macro models, tests conducted on larger composite materials are necessary. The homogeneity of structural characteristics results in an inaccurate simulation of the local vulnerabilities of the bricks or mortar under weak mortar-strong brick unit combinations or vice versa. In addition, macro models did not account for some failure processes of masonry, such as diagonal stress or stair-stepped sliding.

2.4.3 Equivalent Frame Model

URM structures may be analyzed nonlinearly with relative ease using the equivalent frame approach. Homogeneous, isotropic material idealization requires less data to represent material properties than other modelling methodologies. Each wall's local nonlinear behavior is characterized by plastic hinges whose force displacement characteristics are normally established by results of experimental results. Numerous research have been undertaken to increase the EFM's reliability, as it is both easy and effective. The following is a review of attempts to emulate the nonlinear behavior of URM using similar frame models. Gilmore [55] presented an EFM for doing pushover analysis on confined masonry constructions. Shear behavior is linked to the structural deterioration of confined masonry walls, and a rotating shear spring is proposed to idealize the nonlinear response of limited masonry walls. A spring is utilized to determine the relationship between S.F on the wall and inter-story drift having shear deformation. The hinge is situated at the base of the wall, as indicated below.

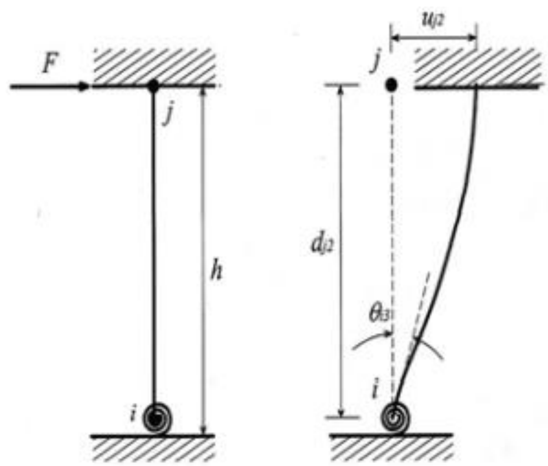


Figure 2- 4: Modified Model Column for PO Analysis [55]

The limitation of the model is that the force-deformation relationship proposed for springs not consider axial load intensity and aspect ratio shown in figure 2.7. It is observed using an expected backbone curve for walls built with 25 restricted handmade clay bricks in Mexico. The authors stress that the proposed shear hinge model is intended to depict a general condition of structural deterioration only in the case of identically planned and constructed Mexican buildings.

In conclusion, a lateral load distribution proportionate to the modal form of the fundamental mode is used for pushover analysis of a typical confined brick construction in Mexico, whose experimental findings are replicated sufficiently by the proposed computer model. Kappos [56] performed elastic and plastic comparative evaluations on two-dimensional and three-dimensional brick structures to evaluate the accuracy of the comparable frame modelling technique. When examining two-dimensional perforated walls, similar frame and finite element models are constructed. According to the investigation, the frame model with complete horizontal and vertical stiff offset has outcomes that are most similar to those of the finite element model. In addition, it is shown that the influence of diaphragm constraint on planar structures is minor, but it is crucial for three-dimensional structures.

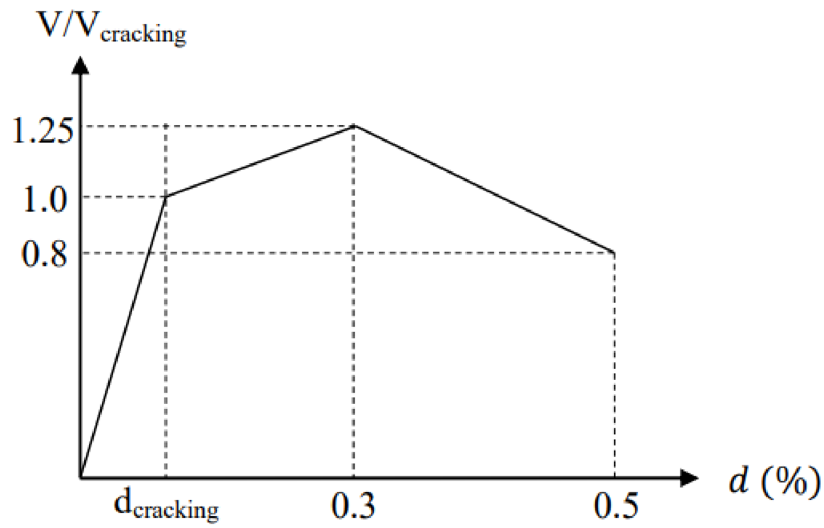


Figure 2- 5: Backbone Curve Modelled for Confined Masonry Walls [55]

Both the ANSYS FEM model and the SAP2000 EFM are constructed for nonlinear analysis. After validation against test data obtained at the Ismes laboratory and University of Pavia, it has been proven that EFM is effective and relatively accurate for nonlinear analysis of brick structures. Salonikios [57] have conducted their work with comparative inelastic analyses on non-linear FEM based models of two-dimensional masonry frames. Flexural and shear hinges are used in similar frame modelling of masonry components. It is assumed that when an earthquake impacts a URM structure, both bending and shear processes are activated, with collapse occurring at the weakest point. Therefore, moment-rotation hinges are positioned at both ends of the element, whereas shear-displacement hinges are positioned in the middle shown in figure 2-6. The FEMA 273 standard specifies the material requirements for plastic hinges.

Different lateral load distributions applied on the structure including first mode shape distribution, uniform distribution, inverse triangular distribution. The distribution of lateral loads has no effect on the base shear capacity of the structure, since the ultimate condition is achieved upon shear failure of all first-story piers, as determined by the results of the study. Uniform distribution results in the greatest initial stiffness comparing early stiffness under different distributions of lateral loads, although the same shear force is delivered regardless of the load distribution at the base story piers, the shear force at the higher story piers is greater for inverse rectangular, modal, and uniform distributions. When lateral stresses are distributed uniformly, roof movement is subsequently reduced

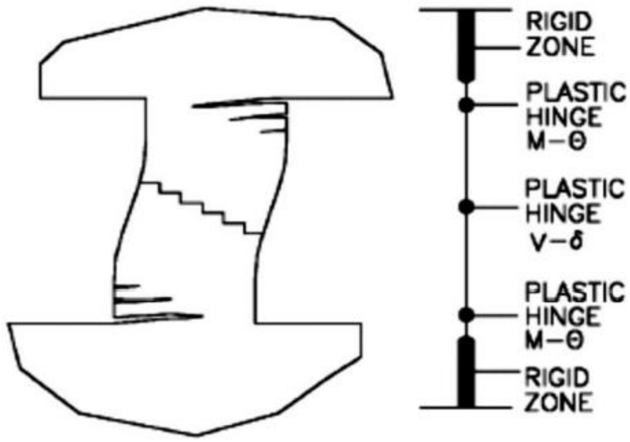


Figure 2- 6: Modeling detail of URM panels [57]

Pasticier [58] intended utilizing SAP2000 for EFM-based seismic assessments of brick structures. In the nonlinear modelling of masonry piers, two rocking hinges are used at the extremities of stiff offsets and one shear hinge is used in the center of the pier. For nonlinear modelling of spandrels, on the other hand, a single shear hinge was developed. For lateral loads, the inverted triangular distribution is assumed. Limitation of SAP2000's inability to immediately update shear strengths in response to a change in axial load level. As shown by study, various approaches for determining the axial load distribution on piers have minimal effect on the ultimate strength and top displacement. The main shortcoming of SAP2000, which is the inability to update pier strengths due to changing axial force, does not seem to be as significant in pushover tests on similar frames..

Two distinct lateral load distributions are utilized for static push over (SPO) analysis. In inverted triangular distribution, the mechanism on the second story is responsible for the collapse, but in uniform distribution, it happened on the first floor. For seismic analysis of masonry buildings, Belmouden and Lestuzzi [59] develop a frame model that is comparable. The analytical model, unlike previously given models, is based on a smeared fracture and scattered plasticity technique. In addition, the link between axial force and bending moment as well as axial force and shear force is examined. Inelastic shear and flexural deformations are permitted for horizontal and vertical members. Shear springs and flexural hinges are placed at the span's midway. Due to the discretization of piers and spandrels into a sequence of slices, nonlinearity is spread over the entire span length shown in Figure 2-7.

The length of spandrels that provide coupling to piers is initialized to zero moment length and changed at each step of the SPO analysis based on the spandrels' end moments. Maintaining the correctness of the model by comparing its findings to those of University of Pavia experiments.

Roca [60] examined two-dimensional masonry elements as comparable systems of 1-d components known as identical frames. Modeling the usual force deformation of compressed brickwork using the Kent and Park model. Using the Mohr-Coulomb criteria as the biaxial stress envelope, the interplay between axial force and shear force is examined. This approach is viable to determine the failure mechanism and maximum load carrying capacity of masonry structure which is based on experimental and numerical data collected by DAsdia in 1972.

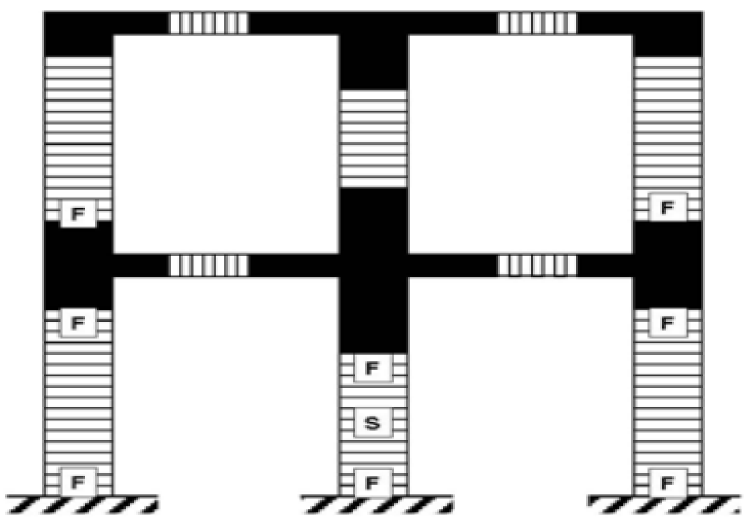


Figure 2- 7: Spread Nonlinearity Approach in EFM [59]

The approach for EFM-based pushover analysis of URM structures was developed by Penelis [61]. At the extremities of structural components, lumped plasticity-based rotating hinges are utilized against the demand. The constitutive law of nonlinear springs is defined by the Moment-rotation curve of each element under constant axial stress, where rotation included both flexural and shear actions. As the material model disregards the relationship between axial force and bending moment, the axial load level at which hinge characteristics are established on piers is determined by a linear analysis that takes both vertical and lateral direction loads comparing shear capacity consideration. In conclusion, the Penelis model is supported by experimental data.

Magenes [62] presented the SAM approach seismic analysis of masonry frames by EFM approach against simplified non-linear analsysis against in-plane action. Shear strength is determined using basic strength equations and connections are modelled as elastic perfectly plastic. Assign a shear failure limit of 0.5% and a flexural failure limit of 1% to the total chord rotation. Using the rigid end offsets provided by Dolce [63] for determining the stiffness matrix in the elastic range, the effective height of a structural component is obtained.

2.5 Seismic Strengthening Measures

Due to the constraints imposed by the existing circumstances, the retrofitting of an existing building is a far more challenging procedure than the design of a new building. For seismic retrofitting to strengthen the lateral load resistance of the structural system, either the existing members/components can be reinforced or new members can be added. Prior to deploying any strengthening strategy, it is necessary to pick a cost-effective and technically effective strengthening technique from the available literature.

Using reinforcing bars and rods to increase the strength and ductility of URM, the center core technique was developed approximately three decades ago in Europe for the retrofitting of masonry buildings. The methodology is outlined by Plecnik [65]. In this technique, a vertical hole is drilled from the top of the wall to the basement. The diameter of a drill normally runs from 50 to 125 mm, although its actual size is determined by the wall thickness and the degree of reinforcing required for each wall/component. Following the drilling of the hole, reinforcing and filler material are injected from the top. Grout operates as a single structural component by adhering to the inner and outer wythes. The wall's core provides strength for both in-plane and out-of-plane motions. This approach gives the benefits of being inexpensive and keeping the architectural appeal of the structure. This method has limitations due to the fact that it generates zones of varying strength and stiffness.

Taghdi have developed a steel strip seismic retrofitting solution for low-rise concrete block masonry structures (2000). Cross and vertical steel strips are fastened to the top of the walls using through-thickness bolts in this sort of reinforcement. Use of stiff angles and anchor bolts for fastening. This technique increases the in-plane toughness, ductility, and energy dispersal capability of low-rise unreinforced walls. The inclusion of steel strips on each side of the walls considerably enhances the out-of-plane performance of the walls. Due to the difficulty of drilling holes and affixing to older brick masonry walls, this approach has not been shown to be successful for brick masonry walls.

Several research have studied the application of FRP stripes to increase the ductility and strength of masonry at the component level. However, some researchers (Islam [66] and Tumialan [67] emphasize the need for more study into the modelling and validation of FRP-reinforced masonry buildings, as well as their practical applicability.

Abboud [68] conducted full-scale experimental bending tests on masonry walls externally reinforced with steel. They observed that longitudinal reinforcing significantly increases the strength and ductility of URM. Shotcrete is the most common method for reinforcing URM walls in Asian nations [69]. Shotcrete is a concrete application using small particles. Since it is sprayed under pressure, it does not need to be compacted like conventional concrete. In addition, the effects of creep and shrinkage are negligible because to the low water-to-cement ratio, and shotcrete with lowered permeability has a reduced tendency to corrode reinforcement. This kind of reinforcement does not necessitate the use of formwork and may be applied to both vertical and sloping surfaces.

Ashraf [70] investigated the impact of ferrocement overlays on brick masonry and restricted masonry walls in Pakistan by conducting quasi-static stress experiments on scaled brick masonry walls. This method is effective for unreinforced masonry walls, but only modestly favorable for reinforced masonry walls, according to the study. The same retrofit approach paired with grout injection was also used to a full-scale URM structure that underwent cycle load testing and demonstrated a 30% drop in strength [71]. They discovered an increase in the building's lateral load capability. In addition, they discovered that the damage mechanism's rocking mode had been eradicated following the retrofitting. Most often used methods for reinforcing Unreinforced Masonry (URM) Walls are externally bonded or near-surface embedded steel strips, bars, wires, or welded wire mesh (WWM) [72-74]. These approaches are now rising in prominence since they are much less expensive than Fiber Reinforced Polymer (FRP) and Engineering Cementitious Composite (ECC) systems and provide substantial increases in masonry strength.

2.6 Performance Evaluation of Unreinforced Masonry Structures

2.6.1 Limit States of Performance for URM Buildings

From the component force-displacement relationship, the global structure's capacity curve may be derived. To assess the performance of a structure, the global resistance curve limitations must be defined. Several writers examine the link between structural deterioration and structural performance. Tomazevic [75] conducts lateral resistance tests on masonry walls and shaking table studies on masonry structures in order to establish a correlation between damage incidence, limit states, and lateral displacement capacity. To assess the seismic resistance of masonry structure created four limit states on the capacity curve (see Figure 2-8).It is characterized by the emergence of the earliest wall cracks. Serviceability limit state the structure itself, maximum resistance, design ultimate limit state: The system's resistance falls below an acceptable threshold, resulting in a 20% decrease in maximum resistance and collapse limit condition: partial or complete building collapse

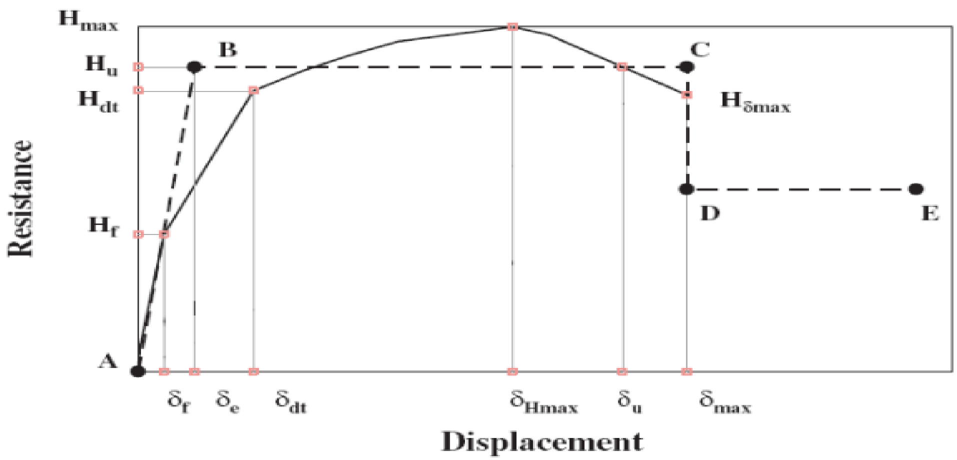


Figure 2-8: Displacement Limits Illustrating Performance Limit States Defined by FEMA 356 [36] dashed lines and Bosiljkov [50]

Tomazevic [75] suggests the following story rotation values linked with corresponding limit states for masonry buildings based on laboratory tests conducted on limited and URM masonry structures. The numbers shown in table below are, however, generated from a compilation of test findings for simple and limited masonry buildings. Due to the fact that confinement improves the maximum displacement capacity, the collapse limit states for URM structures will be lower than the prescribed values. By adjusting drift limits based only on the results of URM construction tests, the collapse limit state will be decreased to between 1% and 2%.

Calvi [76] evaluate the vulnerability of different types of masonry structures. Calvi hypothesized a connection between inter-story drift and the four limit states, beginning with the idea of limit states as performance levels. Namely,

LS1: No damage. It is assumed that the elastic reaction will be linear

LS2: Minor structural damage. After the earthquake, the structure may be utilized without requiring any repairs.

LS3: Significant structural damage. After the earthquake, the building cannot be utilized without major repairs.

LS4: Collapse. Repairing the building is neither practicable nor economical. Beyond this level, a catastrophic collapse endangering human life is expected.

Calvi [76] proposed the following values for story drift for limit states of masonry structures shown in table below, Omer Onur Erbay [77] chose the following threshold values for his study of URM buildings based on prior experimental investigations.

Table 2- 2: Performance Drift Limits for URM Buildings

	Performance levels (%)		
Standard/Author	**IO**	**LS**	**CP**
FEMA 356 [36]	0.3	0.6	1
Tomazevic [75]	0.2-0.4	0.3-0.6	1.0-2.0
Calvi [76]	0.1	0.3	0.5
Erby [77]	0.1	0.6	1

Chapter 3

Methodology

3.1 Data Collection

To collect data on school buildings, three districts, Poonch Valley, Neelam Valley, and Muzaffarabad District, were selected. The school buildings in these regions were visited and inspected, resulting in the identification of building typologies based on the materials used in their construction. According to the Building Code of Pakistan –Seismic Provision 2007, all three districts lie inside seismic zone 4 of Pakistan. This chapter describes the process established and implemented for data gathering in districts under consideration. The collected data showed significant information on the architectural and structural design of schools in the areas under consideration. During the data collecting process, professional interviews were performed to acquire architectural and, more specifically, structural drawings of as-built structures. It was determined that there are three types of school buildings: reinforced concrete (RC) frame constructions, stone masonry schools, and brick masonry schools.

The presence of the Himalayan Range with the highest peaks in the world is proof of the country's ongoing tectonic activity. Consequently, the districts of Muzaffarabd, Neelam, and Poonch are extremely seismically active and prone in Pakistan. They have a long and extensive history of violent earthquakes, with the 2005 Hazara Kashmir Earthquake being the most damaging in recent history. The majority of the recorded earthquake damage happened in Khyber Pakhtunkhwa (KPK), and of the estimated deaths, more than 250,000 school-aged children perished due to the collapse of school buildings and the time of the earthquake, which struck at 8:50 a.m. (Khazai et al. 2006). This occurrence revealed that, due to its geological features, KPK is vulnerable to greater earthquake damage.

Despite the high danger of earthquakes, the school buildings in KPK lack seismic structural safety and are built and erected without seismic design considerations in a manner similar to that of normal residential structures.

Presently, no effort has been made to evaluate the earthquake risk of public schools in these region therefore, it is imperative to conduct the performance assessment of existing school buildings in the public and private sectors to determine their seismic vulnerability in order to suggest the most appropriate retrofit designs to improve their seismic performance. Evidently, the bulk of school buildings are highly susceptible to any type of seismic activity, which is alarming**.**

Three districts of Kashmir, Muzzafarbad, Poonch, and Neelum, were surveyed to compile a database of existing unreinforced Masonry school buildings. The database provides all information on the structural characteristics of schools, including the number of story's, locations and measurements of walls, slab system thickness and type, and opening location and dimensions. The database also comprises the design and as-built structural drawings that were obtained during the data gathering process. Later, the acquired designs were compared to the existing structures in order to get insight into the real execution and current state of the school building structures.

3.1.1 Field Survey Method

The generic building models depict the relative variety and dynamic reaction of the stock's diverse buildings. To establish generic models that can effectively anticipate the structural reaction, it is necessary to have a thorough understanding of the local building designs and construction methods of the research region; thus, a field survey was undertaken in the study area to collect the necessary data.

To acquire information on the structures, a field survey was undertaken using the FEMA 154 Report questionnaire. Eventually, the collected data was used to develop statistics and identify different school building typologies in the visited areas, while a detailed vulnerability assessment was conducted by considering one school building from each building type to assess the structural vulnerability to seismic activities.

Initially, a reconnaissance study was undertaken on every school building for the purpose of data collection, with all information gathered via visual examinations of the exteriors of sample buildings.

Due to the fact that reconnaissance is often a simple activity, the time necessary to collect data from a sample building was brief, and as a result, the field team was able to collect sufficient information about the architectural arrangements of schools in the region under consideration.

After doing a reconnaissance walk for each school's sidewalk, comprehensive tape measures (measurement of structural member dimensions, span lengths, etc.) were collected from the inside and outside of the building with consent from the relevant parties.

At this point, the total number of bays and their corresponding span lengths, the cross-sectional dimensions and positions of the columns (if any), and the thickness of load-bearing walls and infill walls, as well as their locations, were documented via drawings of the first-floor plan. National Engineering Services of Pakistan (NESPAK), a domestically renowned construction supervision firm that works with the Earthquake Reconstruction and Rehabilitation Authority (ERRA) of Pakistan, which was formed after the 2005 Kashmir-Hazara earthquake, provided as-built drawings for the purpose of ensuring adequate data quality and accommodating various data sources and domestic construction practises. It is important to note that NESPAK could only offer three architectural and structural variants for school buildings in the region under consideration. It was noticed that there was neither a centralized nor authority that could regulate school building.

3.1.2 Data Collection in the Examined Region

In regional seismic loss evaluations, a vast number of structures are examined, making it almost difficult to derive fragility functions for each building due to the enormous computing effort required. To overcome this challenge, buildings with similar features are grouped together, and the fragility curves for each group are calculated in this study without a major loss of precision. The subsequent chapter provides specifics on vulnerability assessment. The data was obtained from the districts of Muzaffarabad, Neelam, and Poonch using the method outlined in the section before this one.

2417 schools were specifically visited. There were a total of 2,417 schools; 1158 were single-story, 847 were two-story, and 412 were three-story. Figure 3.1 is a graphical depiction of the gathered data. Figure 3.2 depicts the number of schools based on the principal material employed in their construction. There were a total of 1900 RC frame school buildings, 417 brick masonry structures, and 100 stone masonry school buildings, as seen in Figure 3.2. Table 3.1 displays all facts in percentage form. It is crucial to note that just one of the stone masonry schools was determined to be a two-story construction, while the remainder were single-story ones. 804 reinforced concrete (RC) frames were single-story, 684 were double-story, and 412 were three-story out of a total of 1900 RC frames. Two-story schools had the greatest number of students, although three-story constructions featured superior auxiliary facilities, such as labs, libraries, etc. 255 of the total 417 brick and mortar schools were two-story, while 162 were one-story. The top roof and water storage tanks on the roof were not counted while determining the number of floors. It was asserted fiercely that practically all brick masonry schools were privately owned and operated in residential structures as opposed to purpose-built infrastructure. It was also noticed that these schools charged exorbitant fees, and when private investors ran out of funding or resources to operate the schools, they were permanently shuttered. Therefore, brick-masonry schools did not serve the entire community. However, the present study also analyses them for performance assessment of existing building stock of unreinforced stone and brick masonry school buildings.

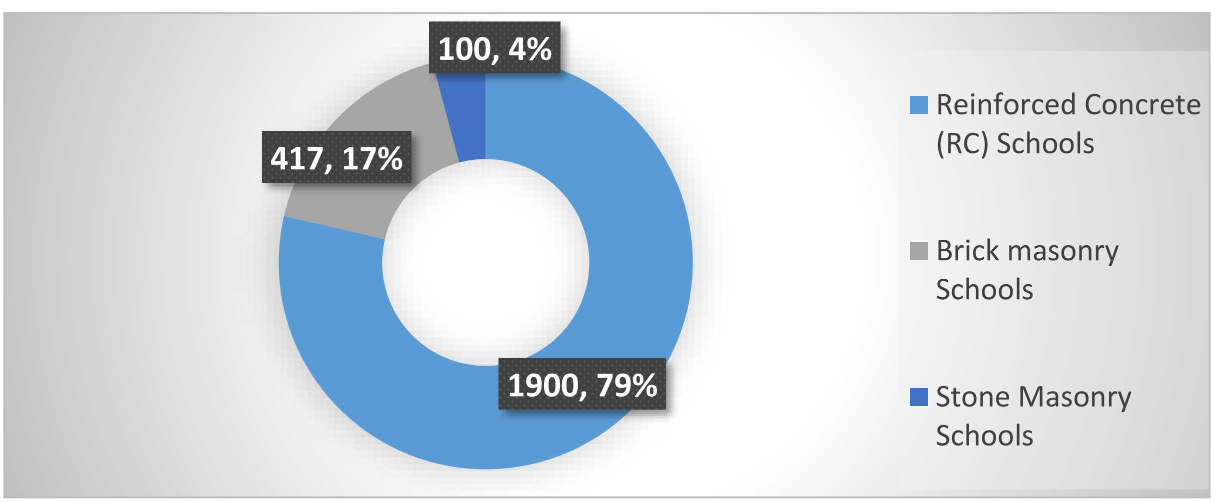


Figure 3- 1: Graphical representation of different school building typologies

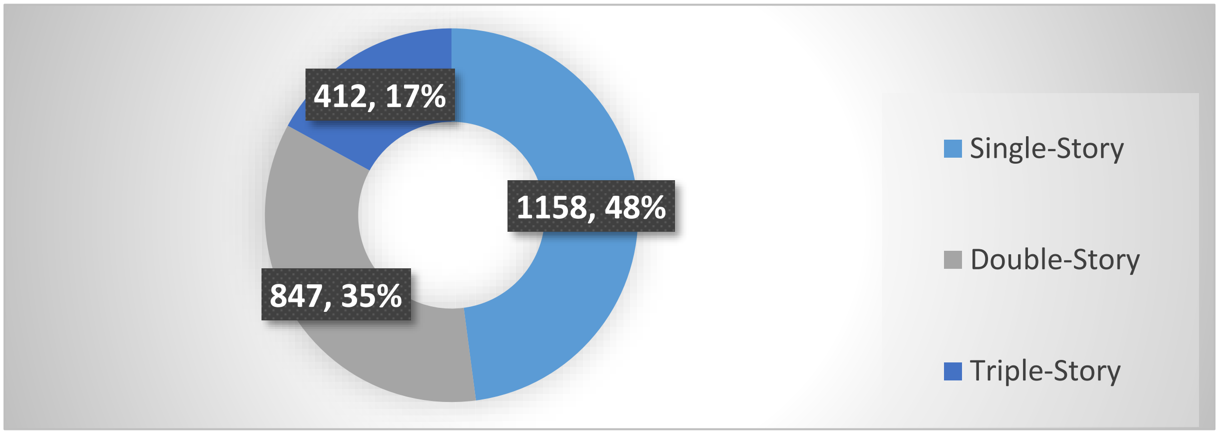


Figure 3- 2: Graphical representation of different school building typologies with different no of stories

Table 3-1: Database of existing school building typology

No. of Stories	**School Building Typology**			
	Reinforced Concrete	Brick Masonry	Stone Masonry	Total
Single Story	804	255	99	1158 (48%)
Double Story	684	162	1	847 (35%)
Triple Story	412	0	0	412 (17%)
Total	1900 (79%)	417 (17.25%)	100 (4.14%)	2417 (100%)

Table 3- 2: Detail of existing stone and brick masonry school buildings

Name of model	Structural configuration	Structural Material	No of story	Total building Height (m)	Total No of Class rooms	Total covered area	No of schools with nearly similar configuration
BLR-1	URM Brick	Brick with C/S Mortar	01	3.6	05	240	102
BLR -2	URM Brick	Brick with Mud Mortar	01	3.0	02	90	24
BLR -3	URM Brick	Brick with C/S Mortar	01	3.1	04	180	129
BLR -4	URM Brick	Brick with C/S Mortar	02	6.1	07	308	162
BLR -5	URM Rubble Stone	Stone Masonry with C/S mortar	01	3.3	03	135	11
BLR -6	URM Ashlar Stone	Stone Masonry with C/S mortar	01	3.6	02	101	85
BLR -7	URM Stone	Stone Masonry with mud mortar.	01	3.1	01	30	03
BLR -8	URM Ashlar Stone	Stone Masonry with C/S mortar	02	6.1	06	310	01

Through professional interviews and field investigations, 08 distinct layouts of URM school buildings were found. Table 3.2 displays pertinent school information, including the number of bays, story height, and slab opening areas. For each of the 08 URM school configurations, each name begins with BLR , which stands for "Building-Low Rise".

3.2 Building configuration

3.2.1 Double story unreinforced brick masonry school building

This structure comprised of 7 no class rooms and 1 no office room based on double story unreinforced brick structure. The plan is regular with general dimension of 17m length and 18m total width as a larger dimension. The story height is 3.1 m for each story. The slab of ground floor is comprised of 150 mm thick reinforced concrete while roof of 1st story is made up of traditional king post roof truss on which rafters as a primary member and joist as a secondary member rest. Top is covered by corrugated metal sheeting. The thickness of wall is 228mm as per site measurement. The foundation was made up of same brick masonry bearing wall of depth 1m below the ground. The plans of the building and 3-dimensional views are shown in figure 3-3 below,

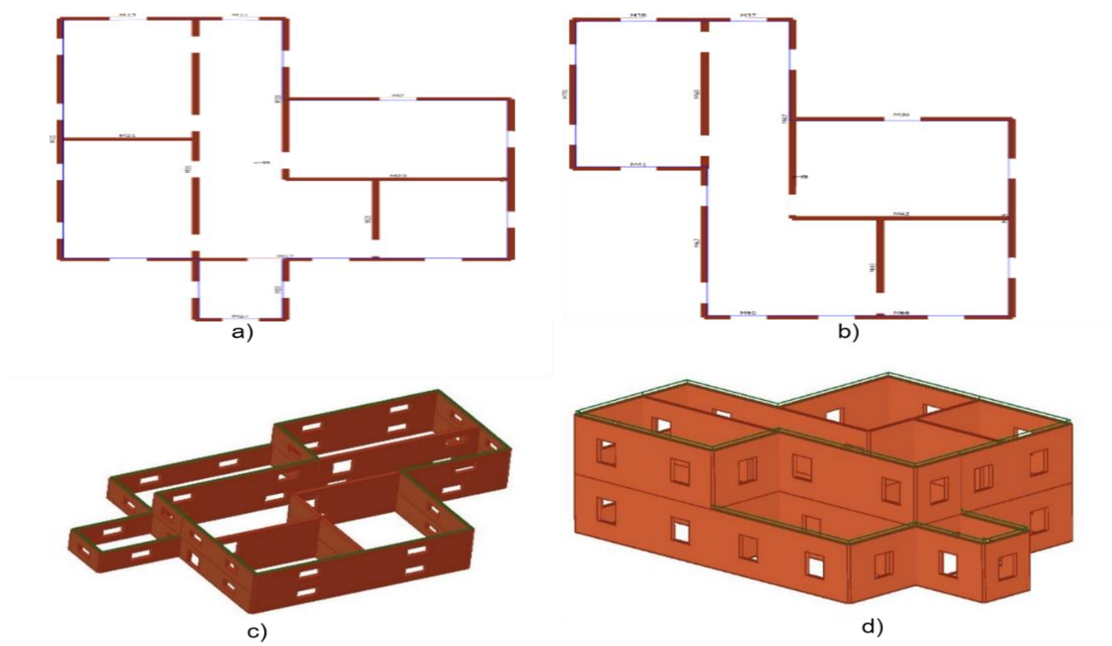


Figure3-3: a) Ground floor Plans, b)1st story plan, 3-d plans of BM1

3.2.2 Single story unreinforced stone masonry school building

This structure comprised of 2 no class rooms and 1 no office room based on single story unreinforced stone ashlar masonry structure. The plan is regular with general dimension of 13.5 m feet length and 7.5 m total width as a larger dimension. The story height is 3.6m. The roof is made up of traditional hip roof truss of four hip rafters. The thickness of wall is considered as 330mm as per site measurement. The foundation was made up of same stone masonry bearing wall of depth 1m below the ground.

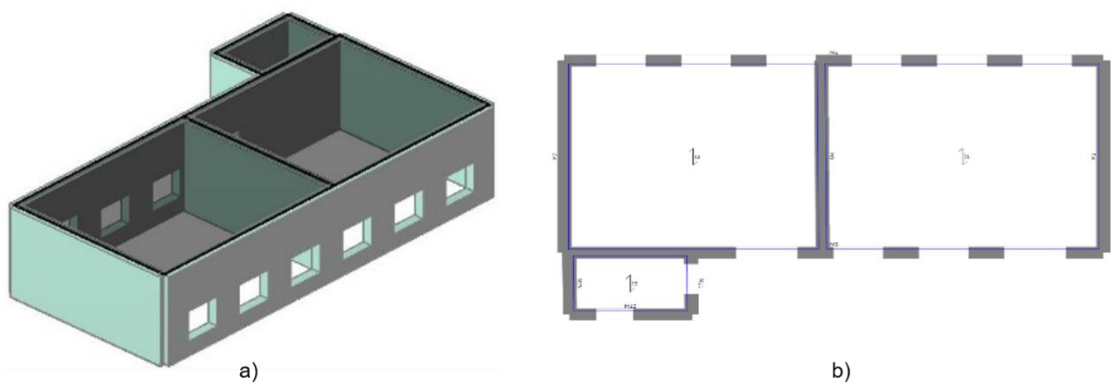


Figure 3- 4: Ground floor Plans, 3-d plan of SM1

3.3 Non-linear modeling

The non-linear structural modelling of both structures was created with the help of commercial software 3 MURI based on TREMURI program. The entire masonry building divided into series of structural elements and global response of the structure depends on the proper interpretation of each anticipated structural element. Non-linear beam element with lumped inelasticity idealization is considered in the TREMURI formulationBy specifying correct force-displacement relationships and drift limitations, the structural element's reaction is evaluated in terms of its global stiffness, strength, and maximum displacement capacity. This modeling approach considered few mechanical parameters and simplified approximations for adequate simulation of the structural response. Implementation of strength criteria chosen for flexural and shear failure modes accordance with recommendations in multiple seismic codes ASCE41-07, EUROCODE-8, and NTC2008 may simply determine the lateral strength of various structural parts.

3.4 Macro element discretization of URM walls with opening

In equivalent frame modelling, the whole brick wall is discretized into piers, spandrels, and stiff nodes, which are the primary structural components. The height of exterior piers was estimated based on the potential development of fractures at the lintel's edge. In the 3-Muri programme, inter-story height and opening height are averaged; however, for internal piers, the height is presumed to be equal to the opening height. Elements of spandrels were determined according to overlap of opening and vertical alignment. The spandrel's length and height are determined by the distance and breadth of the adjacent space. The dimensions of stiff nodes were derived directly from pier and spandrel components that were previously established.

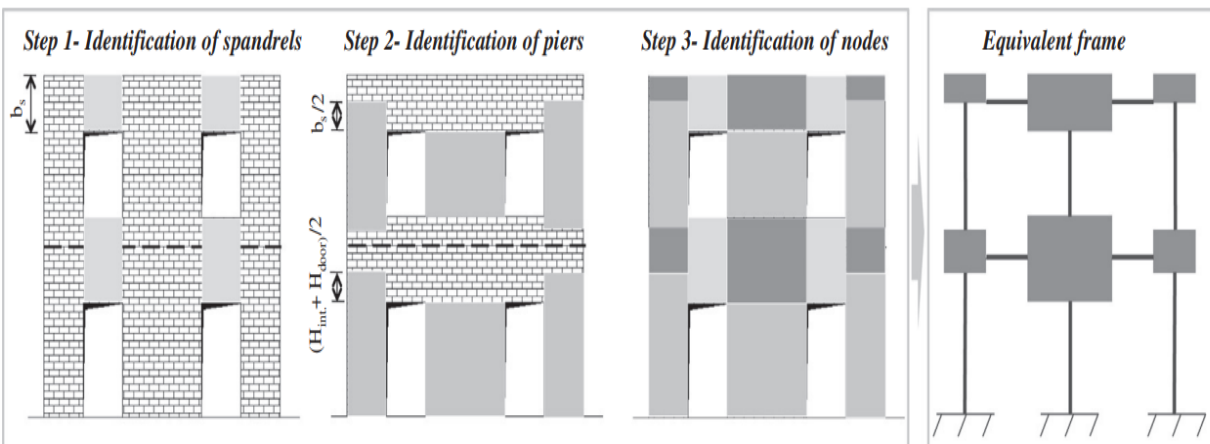


Figure 3-5: Macro-element discretization criteria adopted in 3-Muri program [24]

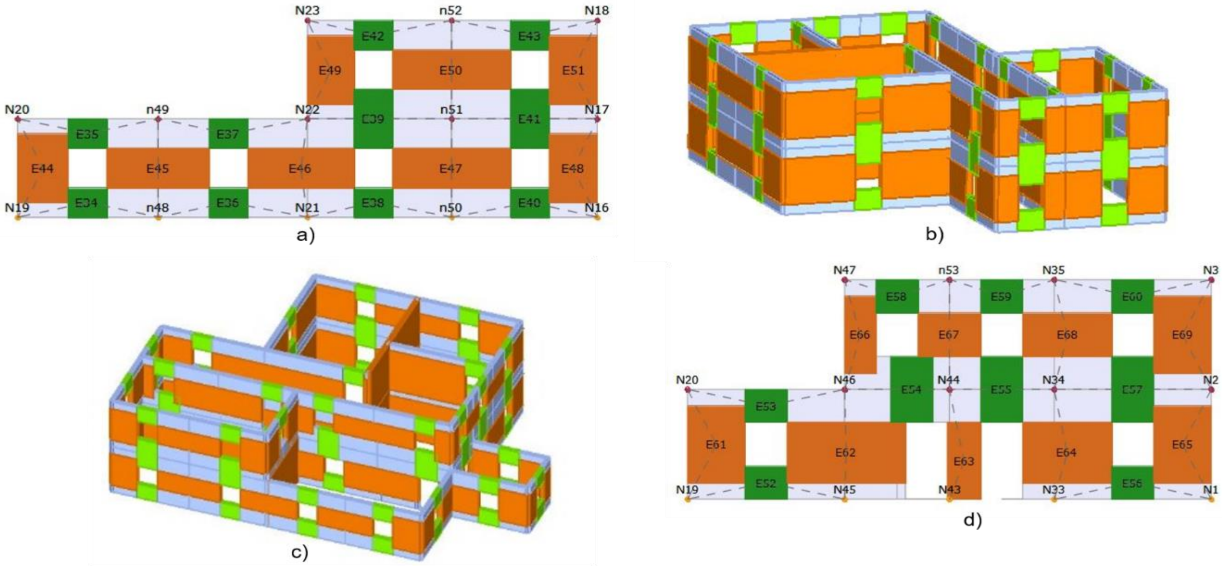


Figure 3-6: Double story URM school building macro-element mesh for each wall of building piers, spandrels and rigid nodes respectively in orange, green and light blue in color.

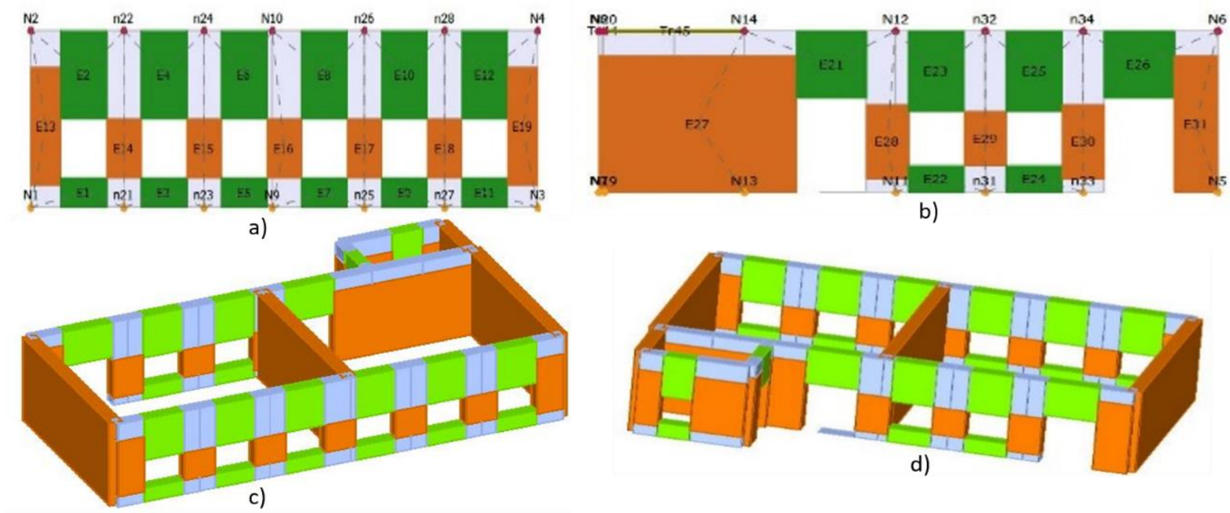


Figure 3- 7: Single story URM school building macro-element mesh for each wall of building piers, spandrels and rigid nodes respectively in orange, green and light blue in color.

3.5 Material Properties

The input mechanical properties of brick masonry building had been taken from the work of are taken from the work of “Seismic resistance evaluation of unreinforced masonry buildings” [42] conducted in Pakistan. In his work a detailed experimental program was executed to know the mechanical properties i.e elastic modulus, compressive strength, poisons ratio, PH value, specific weight etc. The characterization of brick in term of size and shape represents the brick of our site so same values are used as input values in the work are shown in the table below,

Table 3-3: Material properties of brick masonry

Name	Brick Masonry
Elastic Modulus (MPa)	2000
Shear Modulus (MPa)	500
Unit Weight (KN/m3)	14.6
Masonry Compressive strength (MPa)	4.83

Stone masonry building is made up of well-dressed laid stone with cement sand mortar of good quality by appearance. It is categorized as Ashlar stone masonry consisted of lime stone. There is limited information, experimental work and research related to characterization of lime stone masonry typical of Azad Kashmir region in Pakistan. The input values of mechanical properties to characterize the load bearing walls were taken through the central material testing laboratory of Wapda.

Table 3-4: Material properties of stone masonry

Name	Stone Masonry
Elastic Modulus (MPa)	1800
Shear Modulus (MPa)	720
Unit Weight (KN/m3)	21.4
Masonry Compressive strength (MPa)	10.86

3.6 Strength Criteria Adopted For URM Panels

3.6.1 Flexure Mechanism

The maximum flexural strength is defined as a function of compressive force acting on the panel. Shear strength corresponding to ultimate bending moment is achieved by dividing it with the distance ho between the section corresponding to null value of bending moment and pier base.

$$M_{Rd} = NB/2*(1- \sigma_n /0.85f_d) \tag{1}$$

$$V_{Rd,flex} = M_{Rd}/h_0 = NB/2h_0(1- \sigma_n /0.85f_d) \tag{2}$$

3.6.2 Shear Mechanism with diagonal cracking

Turnsek and Cacovic criterion is adopted to define the ultimate shear strength for shear mechanism with diagonal cracking. When principal tension stress attains the tensile strength in the pier central cross section a failure condition is supposed to be taken place.

$$V_{Rd,d} = f_t Bt/\beta \text{ x } \sqrt{1+ \sigma_n/f_t} \tag{3}$$

3.6.3 Sliding Shear Mechanism

The ultimate shear strength is based on the Mohr-Coulomb criterion in sliding shear mechanism, l is the friction co-efficient

$$V_{Rd,s} = l_c t \text{ x } f_{vd} = (c + \mu\ \sigma_n\ ) \text{ x } l_c t \qquad (4)$$

3.6.4 Flexural Mechanism

$$M_u = f(N, f_{tu} / f_{hu}\ , \mu c, \mu c) \qquad (6)$$

$$f_{tu} = \min (\ f_{bt}\ ; c + \mu\sigma_s\ \varphi) \qquad (7)$$

3.6.5 Shear (Bed Joint Sliding)

$$Vu = htc \qquad (8)$$

3.6.6 Shear (Diagonal Cracking)

$$V_{u,dc2} = 1/b\ (lt{\sim}c + \mu N) \qquad (9)$$

The collapse of the panel is determined by assuming a limit value for the drift, which is obtained

$$\delta = \frac{(u_j - u_i)}{h} + \frac{(\varphi_j + \varphi_i)}{2} \leqslant \delta_u$$

The anticipated limit value (u) fluctuates based on the predominant failure mode that occurs in the panel. According to various guidelines suggested in codes Euro-code 8, FEMA 356, and NTC-2008, it typically varies from 0.4% to 0.8% for URM masonry piers; however, recent experimental campaigns have revealed generally higher values for spandrels. Once collapse occurs, the element transforms into a strut; this assumption is conservative because residual shear and bending strengths are not taken into account, while the axial load is still supported and checked to ensure it does not exceed the axial strength Nu.

3.7 Seismic Analysis procedure

Macro-element modelling has the capability to simulate hysteric behavior of masonry panels for non-linear time history analysis and code-based procedure on series of monotonic pushover analysis, equivalent single degree of freedom structure and simplified method to evaluate the maximum displacement. The time-history results are consistent with the pushover approach despite the latter has been developed for other structural typologies characterized by a different non -linear cyclic behavior [38].

To study the seismic performance of school buildings in Pakistan's seismic zone 4, a nonlinear static pushover analysis is used to capture the structural response. Sa (0.2 SEC) g = 2.1 g and Sa (1 SEC) g = 0.6 were assumed for the rocky soil location (class B) of the analyzed structure with Sa (0.2 SEC) g = 2.1 g and Sa (1 SEC) g = 0.6. In comparison to non-linear dynamic analysis, the fundamental advantage of pushover analysis is its relative simplicity in terms of modelling and processing needs, as well as the interpretation of results.

The 3MURI software conducted a static pushover study for each principal direction, including positive and negative X and Y directions. Consideration was given to two load patterns: I uniform, which is proportional to mass, and (ii) pseudo-triangular, which is proportional to the product of mass and height. Pseudo triangular loading is regarded as more accurate than the modal distribution, which is proportional to the shape of the first mode, since the form of the first mode has less mass involvement in flexible floors. According to EC8 [32] and the Italian code [33], the analysis was terminated when the maximum base shear decayed by 20%, and the ultimate displacement capacity of the structure was determined.

3.8 Performance Based Assessment

N2 method is utilized for performance assessment in 3 MURI program following the criteria adopted in both EC8(CEN, 2004).The idea of using non-linear procedures is the identification of performance point, target displacement dt which is computed between the capacity curve as a result of push over analysis and hazard curve which is defined for seismic action of that area. The safety verification of the structure consists of the fact the fact that target displacement dt should be lesser that the structure ultimate displacement capacity du which is assumed to be corresponding to 80% maximum base shear. The seismic demand was defined according to the recent studies conducted by Asad-ur-Rehman and Fawad Ahmad Najam "An updated probabilistic seismic hazard assessment (PSHA) for Pakistan"[78] with 475 years return period and 2475 years return period and 5% critical damping coefficient. The response spectrum for Kashmir region of Pakistan is presented below which is used in the study.

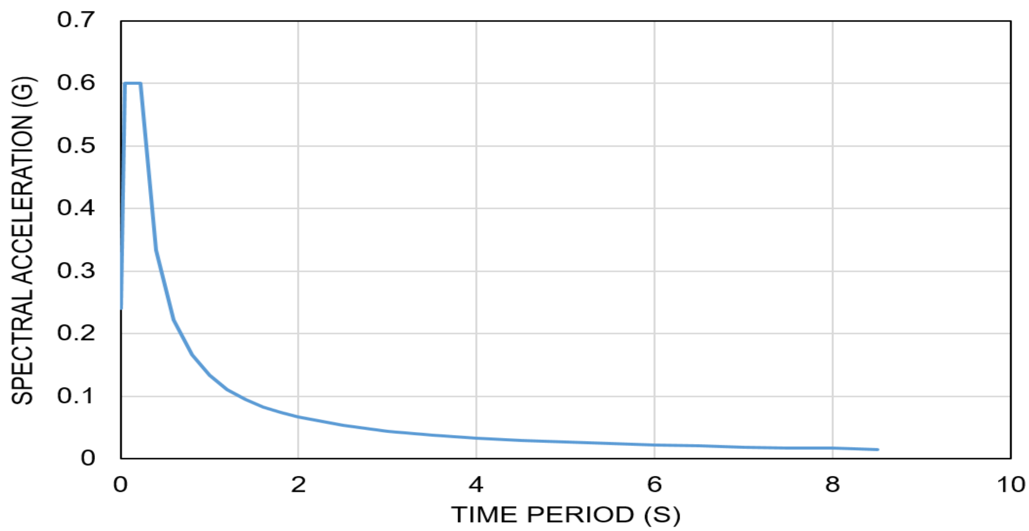


Figure 3-8: Spectrum Diagram at (475) years Return Period [78]

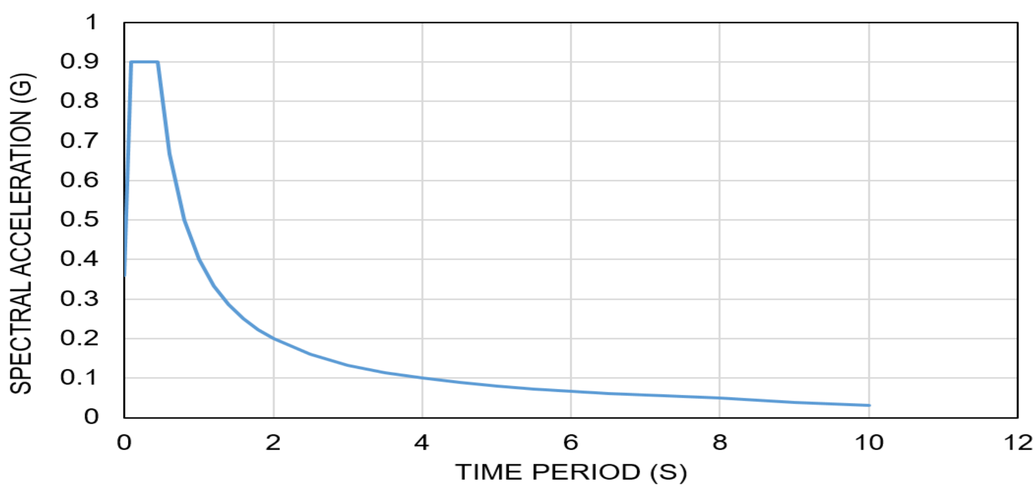


Figure 3- 9: Spectrum Diagram at (2475) years Return Period [78]

3.9 Seismic Strengthening Measures

Seismic strengthening measures to improve the structural global behavior has been adopted in two groups. There are several innovative developments for retrofitting the masonry buildings including the timber floors, wall to wall, roof to wall through steel connections, reinforced plaster, inclusion of tie rods etc. [79-82]. The seismic safety measures adopted which was based on the same solutions proposed at Faial rehabilitation process [83-84] are utilized in this study.

In this study authors opted two set of measures for improvement of wall to wall, roof to wall connections and improving of stiffness of flexible floors. In first set of seismic safety measure P1 steel threaded tie rods 16mm thick are placed beneath roof at both sides of the wall and restrained at the ends by steel anchor plates. The other improvement in the structural performance is by the inclusion of stiffening provisions through 75 mm thick diagonal timber braces between timber joists, anchored with galvanized steel threaded rods and steel angle bracket with addition of new timber sheathing laid perpendicular to existing one with nails. In second seismic safety measure P2 implementation of reinforced plaster in two layers were executed. In first layer 1:3 cement sand mortar may be applied than 0.5 mm thick welded steel mesh of S275 steel and 10mm spaced ribs and then 3 cm thick sound layer of C/S mortar applied to the walls [85].

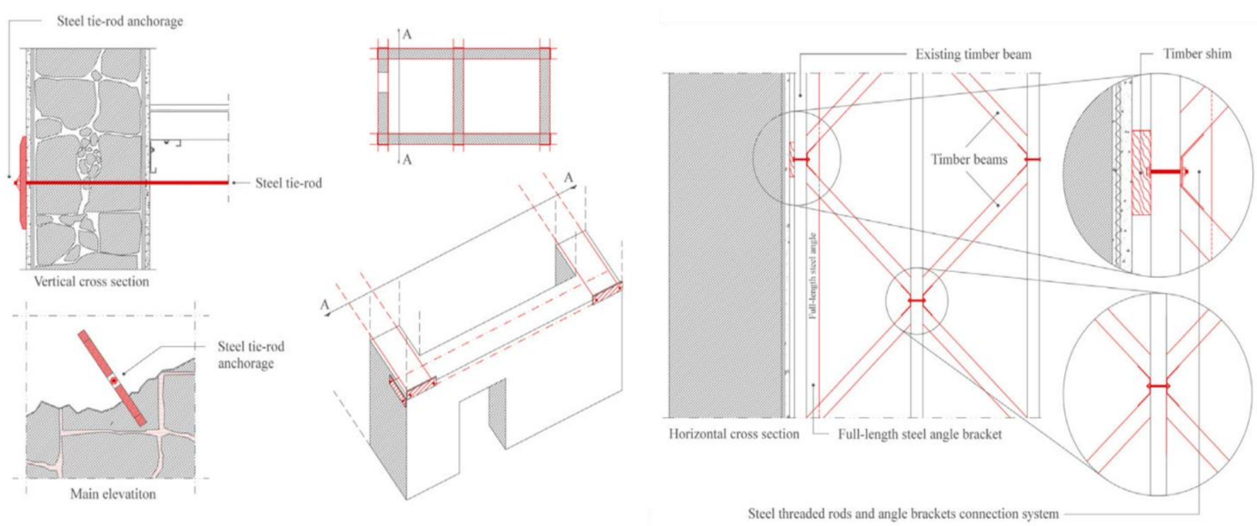


Figure 3- 10: Application of tie rods and cross-timber bracing in the roof [85]

The simulation of tie rods in the models can be implemented in the 3-Muri programme, but it has certain limitations on designing timber bracing in flexible diaphragms as part of P1 seismic safety measure, so following NZSSE guidelines, after ASCE(2014) proposed stiffness multipliers 9.0 were applied to Gd values of 285 and 215 MPa in the load direction acting parallel and perpendicular to timber joists, respectively. In addition to that a double straight sheathing with diagonal sheathing chorded membrane typology using fair rating condition included in P1 measure. However, P2 package was simulated in 3-Muri program by enhancing the parameters of masonry. The mechanical properties of stone masonry typology was increased to 1.5 times the mechanical properties of original stone masonry structure.

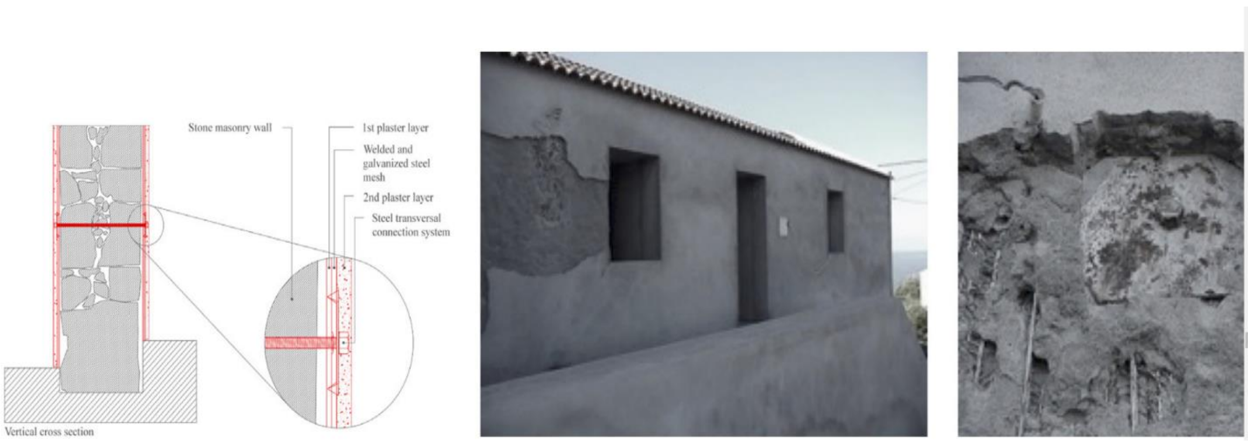


Figure 3-11: Detail of reinforced plaster at external and internal walls [85]

P1 seismic safety measure package enhances the building integrity by improving connections with exterior load bearing walls with frontal walls which is crucial requirement to prevent out of plane collapse but also increase the stiffness of the roof diaphragm. The elements were considered well connected with the walls so effect of elastic and geometric properties was considered in the analysis and ensuring the box like behavior of the structure. While P2 seismic safety measure package improve the confinement and shear strengthening of masonry

Chapter 4

Results and Discussions

The outcome of the study is the performance-based assessment of two numerically models which represent the entire school building stock in Kashmir, Pakistan. The capacity curves were obtained by 3-MURI software for original and seismic strengthened models of stone. The performance-based assessment was carried out by N2 method simulated by 3-MURI software and by the application of limit states proposed by EN 1998-1, EN 1998-3for safety verification at ultimate limit state with 475 years and 2475 years return period.

The results of performance assessment of the both URM school buildings discussed in this section. Firstly, the results obtained by the implementation of N2 method in the 3 MURI commercial software are discussed for original school building models and seismic strengthened models. Secondly, the roof drifts and seismic shear weight of the models were deliberated in detail after comparing with limits proposed by Eurocode standards.The detail discussion of results presented in forthcoming section.

In 3-MURI software the performance-based assessment of the masonry structures was carried out by the application of N2 method in which maximum displacement and target displacements were compared. The maximum displacement of the structure was obtained from capacity curves as a result of push over analysis under different loading conditions, directions. The structure representative pushover curve being on conservative side is considered having low base shear capacity after application of different loading conditions and directions shown in figure 4-1 below. The maximum displacement was taken corresponding to structure 20% decay of maximum base shear however target displacement was computed by the intersection of hazard curve and push over curve for both 475 and 2475 years return period respectively.

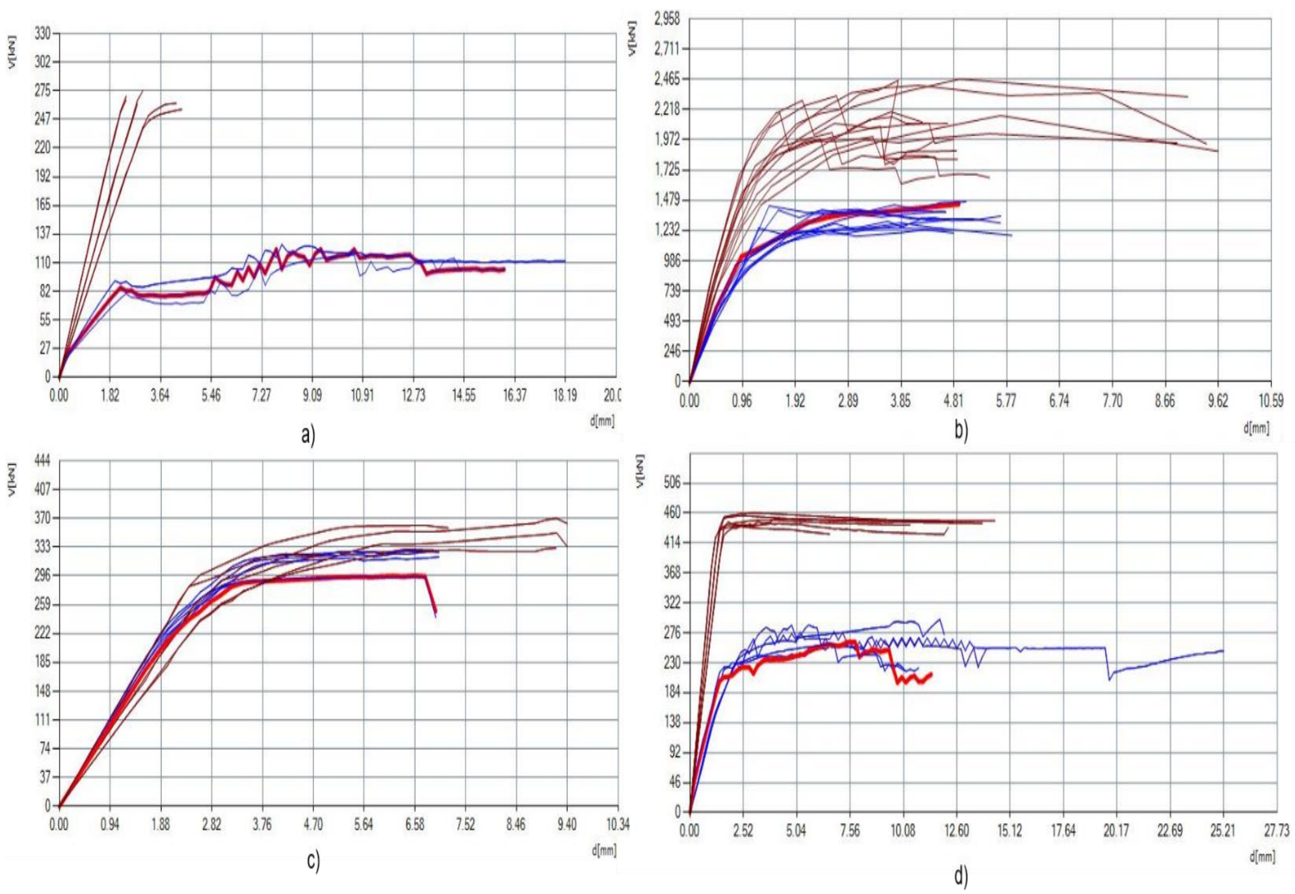


Figure 4-1: Comparison of force-displacement curves under different loading conditions, directions and accidental eccentricity (a) BM1 (b) SM1 (c) SM2 (d) SM3

4.1 Single story URM Stone Masonry School Building SM1, SM2 &SM3

SM1 is unreinforced stone masonry school building in original condition whereas SM2 and SM3 was accompanied with seismically strengthening measures. Push over analysis was performed for the structure considering all the directions and accidental eccentricity 5%. It was observed that most unfavorable analysis was in negative x direction against lateral loading. So, it was taken as a representative structural behavior as shown in figure above below. It was seen that SM1 undergoes maximum displacement of 15 mm corresponding to 119 KN lateral force. Whereas SM2 and SM3 301 KN at 7.5 mm and 260 mm at 11 mm shown in figure below. The prodigious improvement has been observed after implementing seismic strengthening measures on URM stone masonry school building. The displacement capacity of SM2 and SM3 has been increased to 430% and 350% respectively.

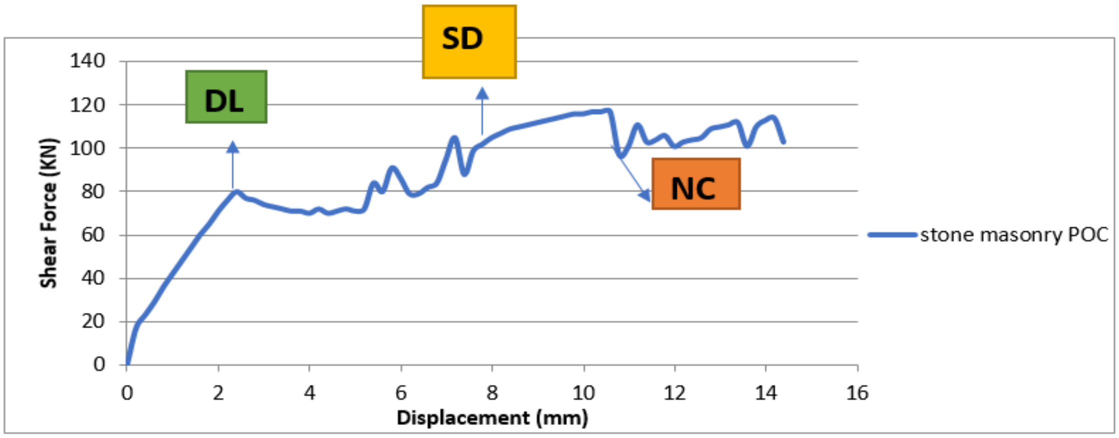


Figure 4- 2: Push over curve SM1

The target displacement calculated by the 3MURI program at 475 years return period for SM1 was 22 mm at SD limit state where maximum displacement taken as 11.3 mm so dm/dt = 0.64 which is less than 1 and found inadmissible against the earthquake having return period of 475 years. For 2475 years hazard at NC limit state the maximum displacement taken as 11.3 mm and target displacement calculated was 36 mm the ratio of dm/dt = 0.39. The performance manifested by SM1 against both level of hazards was quite unsatisfactory and deplorable. The seismic strengthening measures were adopted in SM2 and SM3 models for improved performance.

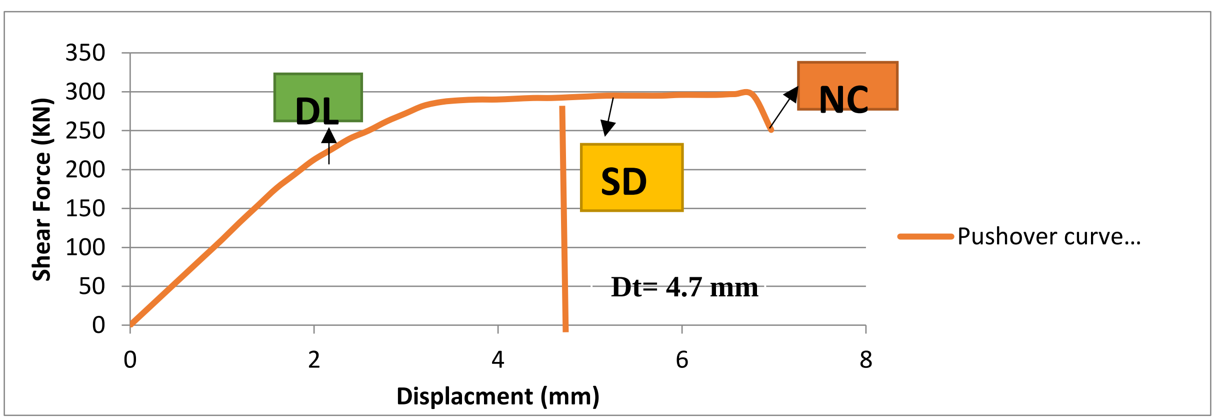


Figure 4- 3: Push over curve SM1 model

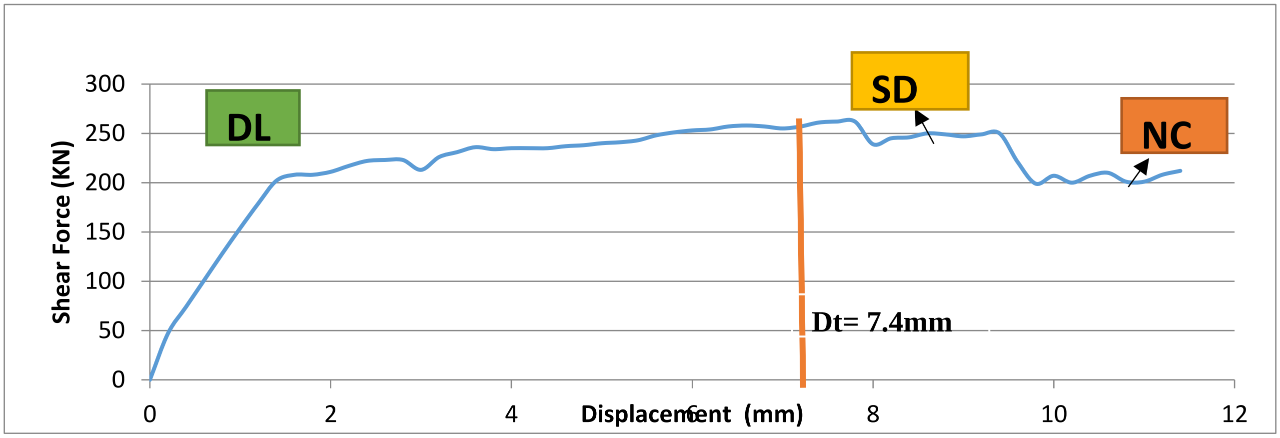


Figure 4- 4: Push over curve SM3 model

The target displacement for SM2 calculated was 4.7 mm at 475 years return period hazard and maximum displacement was 6.97mm which structure can endure. The dm/dt was 1.48 which implies the adequate performance at this level of hazard. Whereas at 2475 years return period hazard maximum displacement was 6.97 mm and target displacement were 7.2 mm, dm/dt = 0.97. In case of SM3 at 475 years return period hazard maximum displacement structure can endure was 11.40 mm corresponding to target displacement of 7.4 mm having dm/dt = 1.54 however for 2475 years return period hazard maximum displacement 11.40 mm and target displacement 12.1 mm bearing dm/dt = 0.94.

Table 4-1: Performance of structural models by N2 method

Structural Model	**Hazard Return Period (years)**	**Target Displacement (dt) (mm)**	**Maximum Displacement (mm)**	**N-2 Method Result dm/dt**	**Performance Limit State**	**Failure Mode**
SM1	475	22	11.3	0.64	NC Crossed	Rocking
SM1	2475	36	11.3	0.39	NC Crossed	Rocking
SM2	475	4.7	6.97	1.48	DL-SD	Diagonal Shear + Rocking
SM2	2475	7.2	6.97	0.97	NC	Diagonal Shear + Rocking
SM3	475	7.4	11.40	1.54	DL-SD	Rocking +Diagonal Shear
SM3	2475	12.1	11.40	0.94	NC	Rocking +Diagonal Shear
BM1	475	5.1	5.3	1.04	SD-NC	Diagonal shear + Rocking
BM1	2475	7.5	5.3	0.70	NC Crossed	Diagonal shear + Rocking

4.2 Double Story Unreinforced School Masonry School Building

BM1 is a unreinforced double story brick masonry school building. The pushover analysis was conducted with same procedure as stated above for stone masonry school buildings. The most unfavorable analysis was in the +X direction. The POC curve shown in figure above encapsulated with maximum base shear 1380 KN and 5.3mm maximum displacement. The performance-based assessment was also carried out for BM1 by the application of N2 method in 3 MURI software. The maximum displacement which structure endured at 475 years return period was 5.3 mm corresponding to target displacement of 5.1 mm which structure can interact with in future hazard. The dm/dt is 1.04 which shows satisfactory performance at this hazard. However, in case of 2475 years hazard maximum displacement was 5.1 mm and target displacement of 7.5 mm. The dm/dt is 0.7.

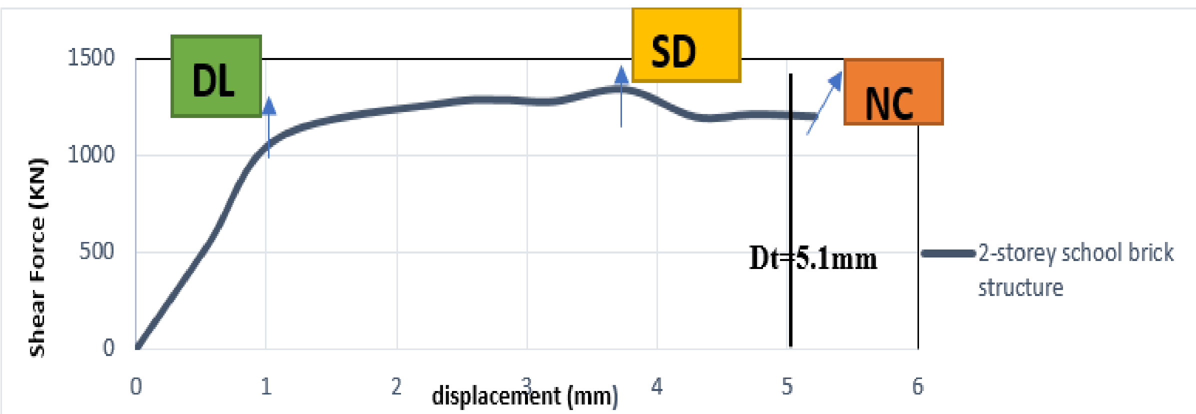


Figure 4- 5: Push over curve of BM1

4.3 Performance Assessment by Limit Sate

Global drift is considered an important parameter to assess the global response of the R.C.C structure and even masonry structures. This criterion has been adopted in several works by researchers and also recommended in EUROCODE, ASCE-41 and FEMA356. In our study performance assessment of unreinforced stone and brick masonry structures was also ascertain by calculating the roof drift and compared with drift limits proposed in EURO Code standards. It defines 3 limit states. Damage limit (DL) at yield point , Significant damage (SD) at ¾ of maximum displacement and Near collapse (NC) at maximum displacement of the structure corresponding to 80% of maximum base shear. When structure is subjected to earthquake and its response is manifested by minor cracking in primary and secondary walls and few corner opening having limited structural damage and minor strength and stiffness degradation corresponds to DL limit states. SD limit states means that significant damage has occurred to structural components. Some structural members are severely damaged having significant strength and stiffness degradation. There is a margin for complete structural collapse still exist. Some minor injuries may occur but overall life safety is not under threat. However, NC is post-earthquake limit state in which structure undergoes nearly complete collapse. A large structural deformation has occurred and building losing substantial strength and stiffness degradation. A significant life threat posed due to falling structural and non-structural components hazards and technically structure may not rehabilitate for re-occupancy.

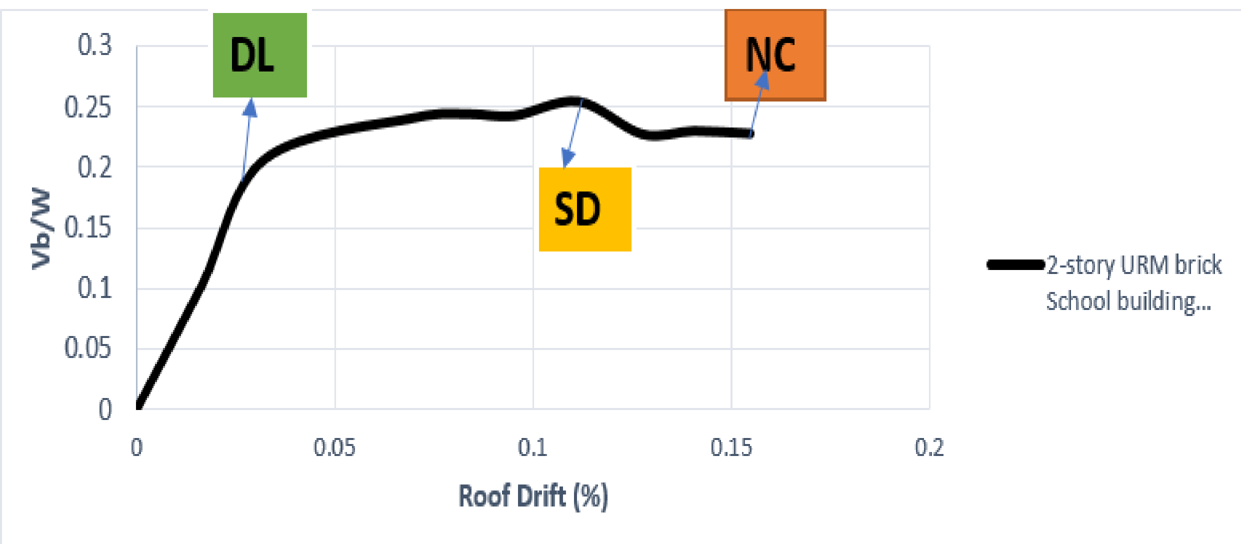


Figure 4- 6: Capacity curve (SRC vs Roof Drift) of BM1 model

The resistance curves were developed for all the models which is graph between seismic resistance coefficient and roof drift. Seismic resistance coefficient is a ratio between resistance and weight of building while roof drift is calculated by ratio of top displacement and height of the building.

In figure 4.5 BM1 model illustrated its 26% maximum capacity to take its normal weight in lateral direction with maximum roof drift of 0.17%. At the target displacement it behave within the range of SD to NC limit state but for higher hazard it crosses NC limit state which means structure have significant structural damage.

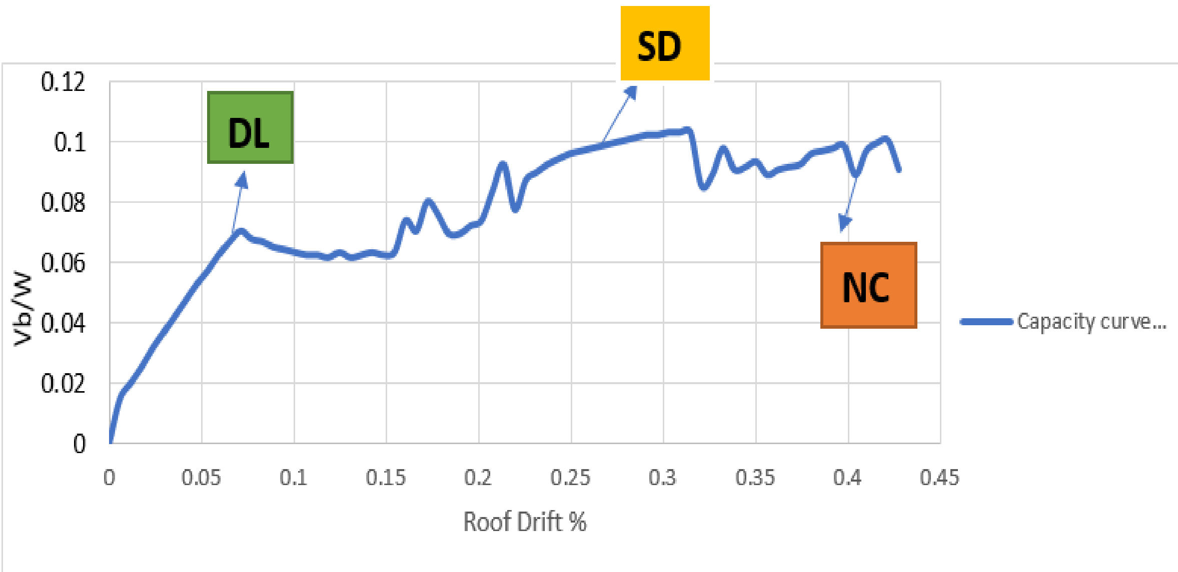


Figure 4- 7: Capacity curve (SRC vs Roof Drift) of SM1 model

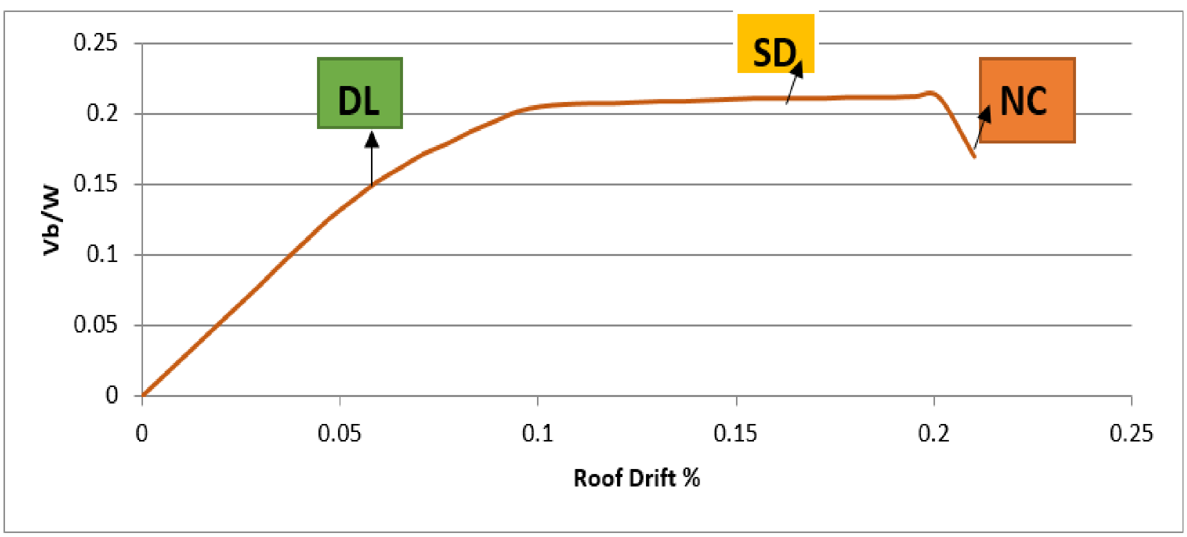


Figure 4- 8: Capacity curve (SRC vs Roof Drift) of SM2 model

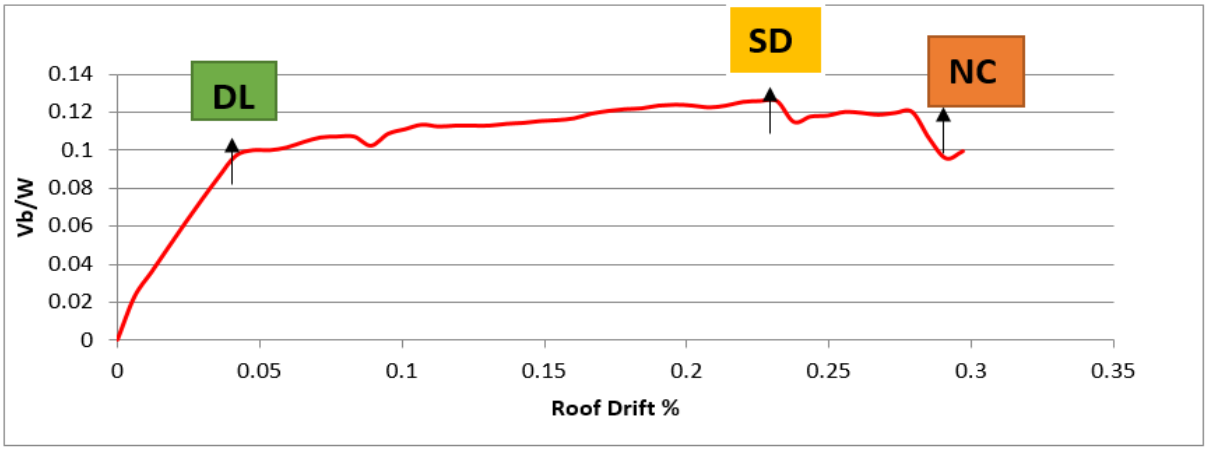


Figure 4- 9: Capacity curve (SRC vs Roof Drift) of SM3 model

However, SM1model showed in figure 4.6 shows it 11% capacity to take normal weight into its lateral direction having maximum roof drift of 0.43%. The URM stone masonry in original condition falls beyond NC limit state at both level of hazards defined earlier. SM2 model accompanied with seismic safety measures in figure 4.7 showed improvement in response with normal load carrying capacity in lateral direction 22% and have maximum roof drift 0.22% and corresponds to SD limit state at 475 years RP and NC at 2475 years RP. In case of SM3 which is also accompanied with seismic safety measure demonstrated with normal weight load carrying capacity in lateral direction of 13% while having maximum roof drift of 0.3% which is also in SD limit state at 475 years RP and NC at 2475 years RP.

4.4 Comparison of Results

The seismic demand parameters such as global displacement and forces were calculated for original URM stone masonry school building SM1 model and for SM2 and SM3 accompanied with seismic safety measures as discussed in above section. The pronounced improvement in response of SM2 and SM3 can be seen compared to SM1 as shown in figure 4.8 below. If we consider 6mm displacement shown in figure below SM1 model undergoes 80KN in shear compared to 300 KN in SM2 and 260 KN in SM3. This implicates a significant improvement of 330% and 320% in force and displacement capacities of SM2 and SM3 models when compared with the original model of the URM stone masonry school building. However, in case of brick masonry structure BM1 the force displacement capacities were significantly higher as compared to stone masonry structures particularly due to improved interconnection between wall to wall , roof to wall, sizeable normal load and rigid diaphragm actions. The maximum load carrying capacity and displacement capacity of BM1 model was 1400KN having displacement of 6mm shown in figure above.

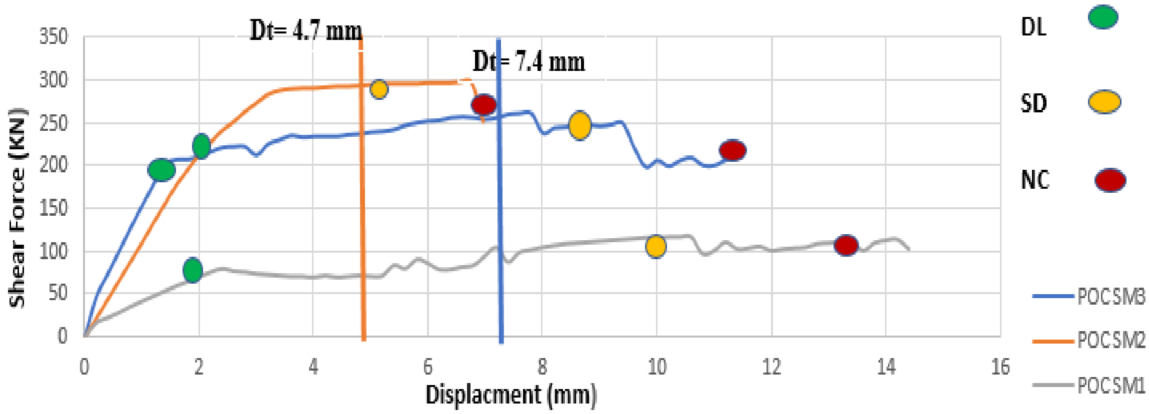


Figure 4- 10: Comparison of Force-Displacement Curves

In figure 4.9 comparison of capacity curves were drawn to unfold the drift capacities and load taking capacity in lateral direction of all the models explained earlier in section. SM1 model bearing normal load in lateral direction 6 % at 0.1% drift compared to 15% and 20% to SM2 and SM3 respectively. However, performance of BM1 model was considered optimal with respect to drift of 0.1% and V_b /W ratio of 25%.

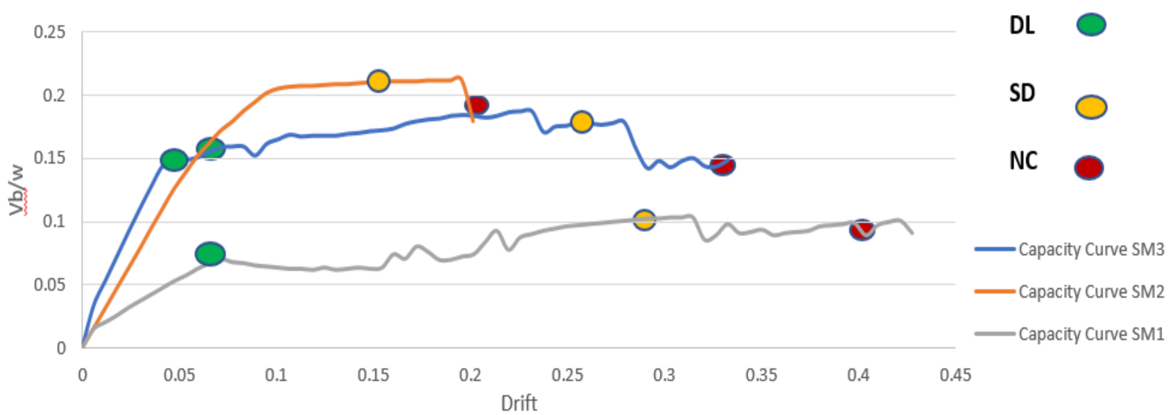


Figure 4- 11: Comparison of Capacity Curves

Chapter 5

Conclusion and Recommendations

5.1 Conclusions

This study is focused on the in-plane response of the structures due to fact that out of plane response have very negligible effect on global behavior of the structure. The performance assessment was carried out earlier in section. The response of SM1 model which represents the BLR-06 majority of URM stone masonry category had a very fragile response against the earthquake hazard of Kashmir at both 475 years and 2475 years return period. The structural model falls beyond the range of NC limit state and also not satisfying dm/dt criteria at both hazards definedwhich implies a huge life risk and infrastructural loss as witnessed earlier in 2005 Kashmir earthquake. The seismic strengthening measures adopted in SM2 model in which wall to wall connection and roof to wall connections were improved with application of tie rods and increased lateral bracing and stiffness of flexible timber roof. The response of SM2 has improved significantly both in terms of load carrying capacity in lateral direction, displacement capacity and global drifts. The performance limit has shifted to significant damage from near collapse. In SM3 model a steel mesh was placed along plaster on the walls of the building. This seismic safety measure also set forth prodigious improvement in response. The performance limit state is shifted from near collapse to life safety after implementing the reinforced plaster on the SM1 model. The inclusion of tie rods and improved stiffness in the roof shifted the behavior of masonry from rocking failure mode to mixed shear and rocking which represents a behavior of actual structures.The drift limits of SM2 was reduced compared to SM1 due to shifting of overall all behavior from rocking which is considered ductile to shear considered brittle. SM3 have more drift capacity as compared to SM2 because majority of panels fails in flexure and some are in shear which is brittle failure.

However, BM1 model which represents BLR-04 majority of double story brick masonry structure has shown satisfactory performance in its original condition. The performance assessment by N2 method stipulate dm/dt ratios were in safe limits SD at 475 years return period and maximum resilience at 2475 years return period and fall at NC limit state for SM2,SM3 and BM1 models.BM1 model fails dominantly in shear with some piers in flexure as well. The overall drift and displacement capacity is less compared to stone masonry models.

5.2 Recommendations

The outcome of the results indicates that integrity and resilience of the structure basically depends on the geometrical configuration, structural stability and loading conditions. There is a huge importance of wall-to-wall connections and roof to wall connections when subjected to lateral loadings. This implicates that a better global response against lateral loadings may be observed if seismic strengthening measures could be adopted on BLR-06 category against both DBE and MCE level hazards in the area. This study is focused on the in-plane response of the masonry structures thus neglecting the out of plane collapse of the structures. The real response of the building contains some part of its out plane response beside in- plane response. In order to get the results more accurate out of plane behavior of the walls should be taken into the account.

References

[1] Abrams, D.P., (2001), “Performance-Based Engineering Concepts for Unreinforced Masonry Building Structures,” Progress in Structural Engineering and Materials, vol.3, pp.48-56.

[2] 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. https://doi.org/10.1007/s10518-013-9496-6.

[3] Agadish, K. S., Raghunath, S., and Nanjunda Rao, K. S. (2003). "Behaviour of masonry structures during the Bhuj earthquake of January 2001." Proc. Indian Acad. Sci. (Earth Planet. Sci.),, 112(3).

[4] Rossetto, T., and Peiris, N. (2009). "Observations of damage due to the Kashmir earthquake of October 8, 2005 and study of current seismic provisions for buildings in Pakistan." Bulletin of Earthquake Engineering, 7(3), 681-699.

[5] Adanur S(2010). “Performance of masonry buildings during the 20 and 27 December 2007 Bala (Ankara) earthquakes in Turkey”. Nat Hazards Earth Syst Sci. 2010;10(12).

[6] Tomazevic, M. (1999). Earthquake Resistant Design of Masonry Buildings, Imperial College Press,London.

[7] Alpa G,Monetto I.Microstructural model for dry block masonry walls within-planeloading.Journal of the Mechanics and Physics of Solid 1994; 47(7):1159–1175.

[8] Lofti HR, Benson Shing P. Interface model applied to fracture of masonry structures. ASCE Journal of Structural Engineering 1994; 120:63–80.

[9] Lourenço PB, Rots J. A multi-surface interface model for the analysis of masonry structures. ASCE Journal of Engineering Mechanics 1997; 123(7):660–668.

[10] Lourenco PB, Rots JG, Blaauwendraad J. Continuum model for masonry: parameter estimation and validation, ASCE Journal of Structural Engineering 1998; 124(6):642–652.

[11] Calderini C, Lagomarsino S. A continuum model for in-plane anisotropic inelastic behaviour of masonry. ASCE Journal of Structural Engineering 2008; 134(2):209–220.

[12] Casolo S, Peña F. Rigid element model for in-plane dynamics of masonry walls considering hysteretic behaviour and damage. Earthquake Engineering and Structural Dynamics 2007; 36(8), 1029–1048

[13] Milani G, Lourenço PB, Tralli A. 3D homogenized limit analysis of masonry buildings under horizontal loads. Engineering Structures 2007; 29:3134–3148. 16. Calderoni B, Marone P, Pagano M. Modelli per la verifica statica degli edifici in muratura in zona sismica, Ingegneria Sismica 1987; 3:19–27 (in Italian)

[14] Abruzzese D, Como M, Lanni G. On the lateral strength of multistory masonry walls with openings and horizontal reinforcing connections. Proceedings of the 10th World Conference on Earthquake Engineering, Madrid, Balkema, Rotterdam, 1992; 4525–4530.

[15] G. Magenes, A method for pushover analysis in seismic assessment of masonry buildings. XII World Conference on Earthquake Engineering, Auckland, 2000.

[16] P. Roca, C. Molins, A.R. Marí, Strength capacity of Masonry wall structures by the equivalent frame method, J. Struct. Eng. 131 (2005) 1601–1610. http:// dx.doi.org/10.1061/(ASCE)0733-9445(2005)131:10(1601).

[17] S. Lagomarsino, A. Penna, A. Galasco, S. Cattari, TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings, Eng. Struct. 56 (2013) 1787–1799. http://dx.doi.org/10.1016/j.engstruct.2013.08.002.

[18] G. Magenes, A. Della Fontana, Simplified Non-linear Seismic Analysis of Masonry Buildings. Proceedings No. 8 of the British Masonry Society, p. 190–195, 1998.

[19] G.G. Penelis, An efficient approach for pushover analysis of unreinforced masonry (URM) structures, J. Earthq. Eng. 10 (2006) 359–379. http://dx.doi.org/ 10.1080/13632460609350601

[32] EN 1998-1, Eurocode 8. Design of Structures for Earthquake Resistance. Part 1-1: General Rules, Seismic Actions and Rules for Buildings, CEN, Brussels, Belgium, 2004

[33] NTC 2008. Decreto Ministeriale 14/1/2008, Norme Tecniche per le cCostruzioni (in Italian), Ministry of Infrastructures and Transportations, 2008.

[34] NZSEE 2015, Assessment and Improvement of the Structural Performance of Buildings in Earthquakes - Section 10 Revision. Seismic Assessment of Unreinforced Masonry Buildings. New Zealand Society for Earthquake Engineering, Corrigendum n° 4, 9 April 2015, Russell, 2015

[35] FEMA 307, “evaluation of earthquake damaged concrete and masonry wall buildings-technical resoruces,” Washington (DC). 1999.

[36] FEMA 356, “Prestandard and commentary for the seismic rehabilitation of buildings,” Washington (DC). 2000.

[37] S. Lagomarsino, A. Penna, A. Galasco, S. Cattari, TREMURI program: an equivalent frame model for the nonlinear seismic analysis of masonry buildings, Eng. Struct. 56 (2013) 1787–1799. http://dx.doi.org/10.1016/j.engstruct.2013.08.002.

[38] Petrovcic S, Kilar V. Sesimic failure mode interaction for the equivalent frame modeling of unreinforced masonry structures. Eng Struct 2013;54:9–22.

[39] Kumar, J., Bhandari, S., Pradhan, S., & Gautam, D. (2020). Simplified frame model for capacity assessment of masonry buildings. Soil Dynamics and Earthquake Engineering, 131(December 2019), 106056. https://doi.org/10.1016/j.soildyn.2020.106056

[40] Pasticier, L., Amadio, C., & Fragiacomo, M. (2008). Non-linear seismic analysis and vulnerability evaluation of a masonry building by means of the SAP2000 V . 10 code. Earthquake Engineering & Structural Dynamic, 37(November 2007), 467–485. https://doi.org/10.1002/eqe

[41] Penna, A., Lagomarsino, S., & Galasco, A. (2013). A nonlinear macroelement model for the seismic analysis of masonry buildings. EARTHQUAKE ENGINEERING & STRUCTURAL DYNAMICS, 43, 159–179. https://doi.org/10.1002/eqe

[42] Waseem, M., C. G. Lai, and E. Spacone. 2017. Seismic hazard assessment of northern Pakistan. Natural Hazards 90(2): 563–600.

[43] Mahmood, I., Kidwai, A. A., Qureshi, S. N., Iqbal, M. F., & Atique, L. (2015). Revisiting major earthquakes in Pakistan. Geology Today, 31(1), 33–38. doi:10.1111/gto.12085.

[44] Magenes, G., Calvi, G.M., Kingsley, G.R., (1995), "Seismic Testing of a Full-Scale, Two-Story Masonry Building: Test Procedure and Measured Experimental Response," Experimental and Numerical Investigation on a Brick Masonry Building Prototype, Report 3.0, University of Pavia.

[45] Yi, T., (2004), Experimental investigation and numerical simulation of an unreinforced masonry structure with flexible diaphragms. PhD dissertation, Georgia Institute of Technology, USA.

[46] Franklin, S., J. Lynch, and D. P. Abrams, (2001), "Performance of Rehabilitated URM Shear Walls: Flexural Behavior of Piers," ST-6 Project Final Report, MidAmerica Earthquake Center Publications, University of Illinois at UrbanaChampaign, Urbana, IL.

[47] Gürel, M.A., Kısa, M., Çılı, F., (2005), "Lateral Stiffness of Unreinforced Masonry Circular Columns under Cracked Conditions."

[48] Alcocer, S.M., Arias, J.G., Flores, L.E., (2004), "Some Developments on Performance-Based Seismic Design of Masonry Structures, Performance-Based Seismic Design Concepts and Implementation," Proceeding of an International Workshop, 233-244.

[49] Tomazevic, M., Lutman, M., Petkovic, L., (1996), "Seismic Behavior of Masonry Walls: Experimental Simulation," Journal of Structural Engineering, no.122, pp.1040-1047.

[50] Bosiljkov, V., Page, A.W., Bokan-Bosiljkov, V., Zarnic,R., (2008), "Evaluation of the Seismic Performance of Brick Masonry Walls",Structural Control and Health Monitoring, vol. 17, no.1, 100-118.

[51] Magenes, G., Calvi, G.M., (1997), "In-plane seismic response of brick masonry walls," Earthquake Engineering & Structural Dynamics, vol. 26, 1091-1112.

[52] Oliveira, Daniel Vitorino de Castro, (2003), "Experimental and Numerical Analysis of Blocky Masonry Structures under Cyclic Loading," PhD Dissertation, University of Minho, Portugal.

[53] Lourenco P.J.B.B., (1996), "Computational Strategies for Masonry Structures", PhD Dissertation, Delft University of Technology, Holland.

[54] Elgawady, M.A., Lestuzzi, P., Badoux, M., (2006), "Analytical Model for In-plane Shear Behaviour of URM Walls Retrofitted with FRP," Composites Science and Technology, vol.66, pp. 459-474.

[55] Gilmore., A.T., Cuevas, O.Z., Garcia, J.R., (2009), "Displacement-Based Seismic Assessment of Low-Height Confined Masonry Buildings," Earthquake Spectra, vol.25, no.2, pp.439-464.

[56] Kappos, A.J., Penelis, G.G., Drakopoulos, C.G., (2002), "Evaluation of Simplified Models for Lateral Load Analysis of Unreinforced Masonry Buildings," Journal of Structural Engineering, vol.128, pp.890-897.

[57] Salonikios, T., Karakostas, C., Lekidis, V., Anthoine, A., (2003), "Comparative Inelastic Pushover Analysis of Masonry Frames," Engineering Structures, no.25, pp.1515-1523.

[58] Pasticier, L., Amadio, C., Fragiacomo, M., (2007), "Non-linear Seismic Analysis and Vulnerability Evaluation of a Masonry Building by Means of the SAP2000 V.10 Code," Earthquake Engineering and Structural Dynamics, no.37, pp. 467-485.

[59] Belmouden, Y., Lestuzzi, P., (2007), “An Equivalent Frame Model for Seismic Analysis of Masonry and Reinforced Concrete Buildings,” Construction and Building Materials, vol.23, No.1, pp. 40-53.

[60] Roca, P., Molins, C., Mari, A.R., (2005), “Strength Capacity of Masonry Wall Structures by the Equivalent Frame Method,” Journal of Structural Engineering, vol.131, pp. 1601-1610.

[61] Penelis, G.G., (2006), “An Efficient Approach for Pushover Analysis of URM Structures,” Journal of Earthquake Engineering. Vol.10, no.3, pp. 359-379.

[62] Magenes, G., Fontana, A.D., (1998), “Simplified Non-linear Seismic Analysis of Masonry Buildings,” Proceeding of British Masonry Society, No.8, pp. 190-195.

[63] Dolce M., (1989), “Models for in-plane loading of masonry walls,” Corso sul consolidamento degli edifici in muratura in zona sismica, Ordine degli Ingegneri, Potenza. (in Italian).

[64] Orduna, A., (2003), “Seismic Assessment of Ancient Masonry Structures by Rigid Blocks Limit Analysis,” PhD Dissertation, University of Minho, Portugal.

[65] Plecnik, J., Cousins, T., and O'Conner, E. (1986). "Strengthening of Unreinforced Masonry Buildings." Journal of Structural Engineering, 112(5), 1070-1087.

[66] Islam, R. (2008). "Inventory of FRP strengthening methods in masonry structures." Advanced Masters in Structural Analysis of Historical Construction, Technical University of Catalonia, Barcelona, Spain

[67] Tumialan, J. G., Morbin, A., Nanni, A., and Modena, C. (2002). "Shear Strengthening of Masonry Walls with FRP Composites " Composites 2001 Convention and Trade Show, Composites Fabricators AssociationTampa,FL.

[68] Abboud, B. E., Hamid, A. A., and Harris, H. G. (1996). "Flexural behavior of reinforced concrete masonry walls under out-of-plane monotonic loads." ACI Structural Journal, 93(3), 327-335.

[69] Ghiassi, B., Soltani, M., and Tasnimi, A. (2012). "Seismic Evaluation of Masonry Structures Strengthened with Reinforced Concrete Layers." Journal of Structural Engineering, 138(6), 729-743.

[70] Ashraf M, Khan AN, Naseer A, Ali Q, Alam B.(2012) “Seismic Behavior of Unreinforced and Confined Brick Masonry Walls Before and After Ferrocement Overlay Retrofitting”. International Journal of Architectural Heritage.;6(6):665- 88.

[71] Ashraf M, Khan AN, Ali Q, Khan S, Naseer A.(2011) “Experimental Behaviour of Full scale URM Building Retrofitted with Ferrocement Overlay”. Advanced Materials Reasearch. 2011;255-260:319-23.

[72] Borri, A., Castori, G., and Corradi, M. (2011). "Shear behavior of masonry panels strengthened by high strength steel cords." Construction and Building Materials, 25(2), 494-503.

[73] Churilov S., and Dumova-Jovanoska E.(2012) "Analysis of masonry walls strengthened with RC jackets." Proc., 15th WCEE, Lisbon,Portugal.

[74] Ismail N, Oyarzo Vera C, and JM, I. (2010). "Field testing of URM walls seismically strengthened using twisted steel inserts." Proc.X Chilean Conference on Seismology and Earthquake Engineering Santiago, Chile 2010.

[75] Tomazevic, M., (2007), “Damage as a Measure for Earthquake Resistant Design of Masonry Structures: Slovenian Experience,” Canadian Journal of Civil Engineering, vol.34, pp. 1403-1412.

[76] Calvi, G.M., (1999), “A Displacement-Based Approach for Vulnerability Evaluation of Classes of Buildings,” Journal of Earthquake Engineering, vol. 3, 411-438.

[77] Erbay, O.O., (2007), “A Methodology to Assess Seismic Risk for Populations of Unreinforced Masonry Buildings,” Mid-America Earthquake Center Report 07-10.

[78] Rahman, A. ur, Najam, F. A., Zaman, S., Rasheed, A., & Rana, I. A. (2021). An updated probabilistic seismic hazard assessment (PSHA) for Pakistan. In Bulletin of Earthquake

Engineering (Vol. 19, Issue 4). Springer Netherlands. https://doi.org/10.1007/s10518-021-01054-8

[79] Faggiano B, Marzo A, Formisano A, Mazzolani FM. Review: innovative steel connections for the retrofit of timber floors in ancient buildings: a numerical investigation. Comput Struct 2009;87(1–2):1–13. http://dx.doi.org/10.1016/ j.compstruc.2008.07.005.

[80] Brignola A, Pampanin S, Podestà S. Evaluation and control of the in-plane stiffness of timber floors for the performance-based retrofit of URM buildings. Bull N Z Soc Earthquake Eng 2009;42(3):204–21.

[81] Araújo AS, Oliveira DV, Lourenço PB. Numerical study on the performance of improved masonry-to-timber connections in traditional masonry buildings. Eng Struct 2014;80:501–13. http://dx.doi.org/10.1016/j.engstruct.2014.09. 027.

[82] Moreira S, Ramos LF, Oliveira DV, Lourenço PB. Design parameters for seismically retrofitted masonry-to-timber connections: injection anchors. Int J Archit Heritage 2016;10(2–3):217–34. http://dx.doi.org/10.1080/15583058. 2015.1113339.

[83] Costa A. Strengthening and repairing earthquake damaged structures. In: Oliveira C, Roca A, Goula X, editors. Assessing and managing earthquake risk. Netherlands, Dordrecht, Netherlands: Springer; 2006. p. 403–26. Ch. 19. R. Maio et al. / Engineering Structures 141 (2017) 41–58

[84] D'Ayala D, Speranza E, Housing Report: Single-family stone masonry house, Tech. rep., Earthquake Engineering Research Institute (EERI) and International Association for Earthquake Engineering (IAEE), Italy; 2002.

[85] Costa A. Determination of mechanical properties of traditional masonry walls in dwellings of Faial Island, Azores. Earthquake Eng Struc Dyn 2002;31 (February 2000):1361–82. http://dx.doi.org/10.1002/eqe.167.

[86] Magenes, G., Calvi, G.M., Kingsley, G.R., (1995), "Seismic Testing of a Full-Scale, Two-Story Masonry Building: Test Procedure and Measured Experimental Response," Experimental

and Numerical Investigation on a Brick Masonry Building Prototype, Report 3.0, University of Pavia.

[87] Abrams, D. P., & Shah, N. (1992). Cyclic load testing of unreinforced masonry walls. Illinois Univ at Urbana Advanced Construction Technology Center.

[88] Manzouri, T. (1995). Nonlinear finite element analysis and experimental evaluation of retrofitting techniques for unreinforced masonry structures. University of Colorado at Boulder.

[89] Magenes, G., & Calvi, G. M. (1992, July). Cyclic behaviour of brick masonry walls. In Proceedings of the 10th world conference on earthquake engineering (pp. 3517-3522).

[90] Abrams, D. P., & Epperson, G. S. (1989). Testing of brick masonry piers at seventy years. In The Life of Structures (pp. 291-298). Butterworth-Heinemann.