Behaviour of Unreinforced Masonry

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Dedicated to

My Family & Guide...

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Certificate

This is to certify that the thesis entitled, BEHAVIOUR OF UNREINFORCED MASONRY submitted by Nikhil P. Zade in partial fulfillment of the requirement for the award of Master of Technology degree in Civil Engineering with specialization in Structural Engineering at the National Institute of Technology, Rourkela is an authentic work carried out by him under my supervision and guidance. To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any degree or diploma.

Pradip Sarkar ***Robin Davis P.***

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Abstract

Unreinforced Masonry (URM) structures can simply defined as structure without any reinforcement. URM is a common material for building construction but is known for its seismic vulnerability due to its heavy weight, high stiffness and negligible strength. URM structures are commonly used in developing countries like India for low rise building up to two story in rural area. Damage to those structures results in loss of life and cultural heritage.

The main objective of the present thesis is to know the lateral behaviour of URM structure, and understand the concept of equivalent frame modelling (EFM). In the present work inverted triangular and uniform distribution lateral loads are used to study the nonlinear behaviour of masonry. There are several methods to carry out Static Pushover (SPO) analysis of URM, but Equivalent Frame Modelling is the simple one. EFM is being used for modeling the non-linear behavior of masonry by providing flexural and shear hinges in the model. EFM is nothing but assuming wall with opening as combination of horizontal and vertical members. The plastic hinges were used in SPO analyses since they allow the user to accurately follow the structural performance beyond the elastic limit at each step of the incremental analysis. Perfectly rigid plastic hinges were assumed as recommended in literature reviews and modelling is done in SAP2000 software.

In order to know which property of masonry is sensitive to lateral behaviour, sensitivity analysis is carried out. Sensitivity analysis was carried out by varying all parameters with 5%, mean and 95% value. Tornado diagram is used to represent the results of sensitivity. It was found that except compressive strength all other parameters are affecting the lateral behaviour.

The fragility can be regarded as one of the most important tool for performance based design of structures. The fragility curves are developed by using HAZUS methodology. Different damage levels such as slight, moderate, extensive and complete damage state are considered to represent variability in seismic performance of building and finally fragility curves were obtained for three damage state quality levels of masonry based on spectral displacements and damage probability. It is observed that the building have more probability for moderate damage. Different brick masonries are considered, to compare the results of the pushover.

***Keywords*:** URM; EFM; SPO; Sesmic performance; sensitivity; fragility.

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List of Abbreviations

URM	Un-Reinforced Masonry
ATC	Applied Technology Council
ASCE	American Society of Civil Engineers
FEMA	Federal Emergency Management Agency
IS	Indian Standard
SAP	Structural Analysis Program
EFM	Equivalent Frame Modell
RCC	Reinforced Cement Concrete
AAC	Autoclave Aerated Concrete
CLC	Cellular Lightweight Concrete
SPO	Static Pushover
DCM	Displacement Coefficient Method
CSM	Capacity Spectrum Method
CLM	Clay Masonry
FLM	Fly Ash Masonry
CM	Cement Mortar
TD	Tornado Diagram
LHS	Latin Hypercube Sampling
CLM	Clay Masonry
FAM	Fly Ash Masonry

List of Notations

Roman Symbols

M_u	ultimate moment
V_u^f	shear strength with respect to diagonal cracking
V_u^s	shear strength with respect to sliding
D	pier width
t	pier thickness
k	coefficient taking into account the vertical stress distribution at the compressed toe
f_m	design compression strength
f_{v0d}	design shear strength
H_0	effective pier height
V_u	spandrel shear strength
h	spandrel depth
E_m	Elastic modulus of masonry
G_m	shear modulus of masonry
S_{du}	ultimate spectral displacement
S_{dy}	yield spectral displacement

Greek Symbols

Υ	unit weight
σ_0	mean vertical stress
μ_f	friction coefficient
ε	coefficient related to the pier geometrical ratio
γ_m	safety factor

δ_u	ultimate shear displacement
ϕ_u	ultimate rotation
μ	Poisson's ratio
Ø	normal cumulative distribution function
β_{ds}	variability parameter
$\bar{s}_{d,ds}$	median spectral displacement
β_c	variability in the demand spectrum
$\beta_{M(ds)}$	uncertainty in the estimation of the damage state

Chapter 1

Introduction

1.1 Overview

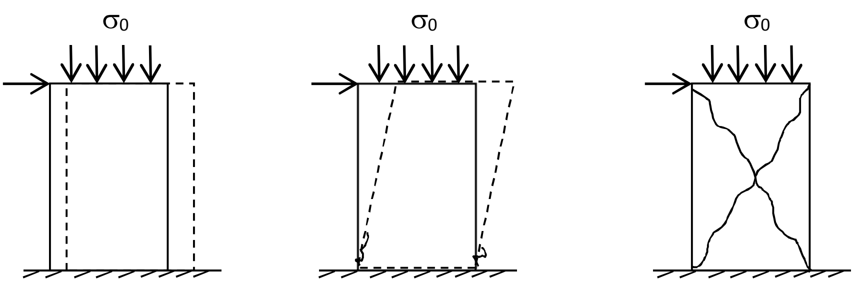
Unreinforced masonry (URM) is common construction practice in a large number of places in the world. It is very popular primarily due to economy, easily availability, good thermal insulation and fire protection, durability and no super skill is required to its construct. Normally, masonry is designed for vertical loads since it has good compressive strength. Due to good compression strength, the structures will behave well when loads are gravity load only but when lateral horizontal earthquake forces act, they start to develop shear and flexural stresses as shown in Fig. 1.1 and Fig. 1.2. Since less research and technical development is done in this field and due to little intelligence required, URM construction is usually done without any technical information. Hence URM construction poses threat to earthquakes damages and is the reason for the replacement of URM construction with steel and RCC. The existing URM construction possesses a risk during earthquakes. Therefore, for performance-based earthquake engineering concepts need for non-linear static analyses arises. In recent years, non-linear methodologies like Pushover Analysis are being used for retrofitting and rehabilitating existing buildings. Pushover analysis is an approximate analysis method in which the building model is subjected to a predefined load pattern and the loads are increased monotonically until some members yield.

Fig. 1.1: Combined in-plane and out-of-plane failure mode in Kashmir 2005 (Naseer *et al*. 2010)

Fig. 1.2: Piers and spandrel failure (Ingham, 2011)

The structure is modified for decreased stiffness of the yielded members and the loads are again increased until a controlled displacement is reached or the structure becomes unstable. For Pushover analysis, non-linear hinges are required to be inserted in the model. The non-linear properties of these hinges are based on the failure mechanisms occurring in the masonry. The various failure mechanisms are shown in the Fig. 1.3 are described as follows:

- Rocking: It is a flexure-dominated failure in which flexural cracks are developed at the bottom and top of a wall.
- Diagonal shear: It is described by stair-stepped cracks along head and bed joints or horizontal cracks along bed joints.
- Diagonal tension: Failure due to shear with diagonal cracking in the centre of the wall.
- Toe Crushing: It is characterized by crushing of masonry at high compression zone, which is generally located at the base end of the wall.



(a) (b) (c)

Fig. 1.3: Various types of failure in masonry pier:
(a) sliding shear, (b) rocking and (c) diagonal shear cracking

There is a great threat of earthquake damage to URM building since it is weak in carrying lateral loads. There are many URM historical important structures as well as housing units in India which may damage due to the earthquake. Still, it is difficult to predict the post-earthquake performance of such structures. SPO analysis is an important tool to evaluate the seismic performance of the building.

1.2 Objectives

Principal objectives of the present study are as per the following:

a) To study the behaviour of URM buildings using nonlinear analysis of equivalent frame concept
b) To ascertain the results obtained from the Equivalent Frame Analysis and the current code provisions FEMA 356 for URM structures subjected to seismic loading
c) To develop fragility curves for URM buildings and
d) To carry out a sensitivity analysis.

1.3 Scope of the Study

Due to lack of experimental data, the present review is constrained to medium strength clay brick, fly ash brick, AAC and CLC brick masonry. However, variation in properties of masonry in a different region is not considered and hollow block masonry is kept outside the extent of the present study.

Two-dimensional wall panels with door and window opening are used for analysis to define in-plane lateral load-deformation behaviour of the wall panel. Rigid wall i.e. without opening is not considered in present study. Due to the very small contribution of out-of-plane lateral strength in the lateral behaviour of the wall, it is ignored in the present study.

1.4 Methodology

The various steps undertaken in the present study to accomplish the previously mentioned goals are:

a) Carry out the extensive literature review, to establish the objectives of the research work.
b) Understand the concept and procedure of performing a pushover analysis.
c) Develop Equivalent Frame Model in SAP2000 to represents unreinforced masonry wall.
d) Obtain lateral force versus top displacement relation from previous experimental result reported in the literature and compare them with existing hinge models.

e) Developing the fragility curve and carry out sensitivity analysis based on value available from the literature.

1.5 Organization of Thesis

The overall idea about the present study is given in the Chapter 1.

Chapter 2 begins with a depiction of the past work done on unreinforced stone masonry by different specialists and results acquired by them. It has two parts. First part gives an idea about experimental research, whereas in second part analytical research done on pushover analysis of URM is given.

Chapter 3 gives the idea of pushover analysis and various terms used in the analysis. Different load patterns specified by the codes, equivalent frame modelling concept are explained in this chapter. Validation of EFM model is done in this chapter.

Chapter 4 begins with the geometric details of the selected wall and masonry properties used and different load pattern considered in the present study. Lastly, this chapter presents and compares the nonlinear static analysis (pushover) results carried out for the same wall specimen with different masonry properties.

Chapter 5 presents details of sensitivity analysis carried out in the present study. Various properties considered for the sensitivity of different masonry. Results obtained from sensitivity analysis is represented in the Tornado diagrams.

In Chapter 6, step-by-step procedure to develop the fragility curve is explained. Various terms used in the fragility curve is explained in details and developed fragility curve for clay masonry.

Finally, Chapter 7 presents a summary including of all work done in the present study. Principal objectives, critical conclusion and future scope are given in this chapter.

Chapter 2

Literature Review

2.1 Introduction

Pushover analysis is a nonlinear static method. A number of literature reviews are available on pushover analysis of RCC and steel building but very few are available for unreinforced masonry building. Pushover analysis is an important tool for the seismic evaluation of the building**.** This chapter describes a few of experimental and analytical works on unreinforced masonry buildings and review of seismic evaluation methods available in the literature. Various results obtained from previous work done on SPO analysis are mentioned in this chapter.

2.2 Experimental Research

Studies were carried out by Krishna and Chandra (1965) and Krishna *et al*. (1966) on SPO analysis. The static in-plane strength of walls with and without reinforcement was studied Various masonry properties required for determining the lateral behaviour are first to determine and later on failure reasons with various methods for strengthening the masonry houses. Key points obtained from results like URM structure results in brittle failure and its energy absorbing capacity limited by elastic deformation. Stronger the mortar grade results in high resistance to the earthquake.

Scrivener (1972) has done a review of the harm to old URM work structures in earthquake zones around the world. Results shows that monotonically increasing load like SPO analysis gives some idea about deformation and initial strength of URM but for detailed seismic analysis dynamic loading gives more accurate results about stiffness reduction, ductility and energy dissipation.

Arioglu and Anadol (1973) studied the various earthquakes in Turkey and concluded that plain URM buildings are most sensitive to earthquake damage. It was suggested to provide at some vertical intervals, horizontal wooden members on both faces to counteract fall of URM structures which results in better performance during earthquake compares to normal masonry construction. Such practices have been traditionally popular in Turkey.

Abrams (1992) examines the in-plane lateral load behaviour of unreinforced masonry elements under monotonic and cyclic loading. He contends that although masonry is considered to be brittle it has considerable deformation capacity after the development of the first crack. Several pieces of advice have been made to evaluate the masonry strength characteristics under lateral loading.

Bruneau (1994) presents the handy utilizations and theoretical procedure another to assess URM bearing wall structures, created in California and as of late coordinated into the new Canadian Guidelines for the seismic evaluation of existing buildings makes a number of conclusions on the seismic performance of un-reinforced masonry buildings. Failure of URM is mainly due to anchorage failure when joists are anchored to a wall or due to lack of anchorage between walls and floor. Different types of failure listed as given by Bruneau:

a) Out-of-plane failure
b) In-plane failure
c) Combined in-plane

Rai and Goel (1996) Lateral behaviour of URM structure mainly depends on pier and spandrel which can be effectively improved by providing steel frame of vertical and horizontal members around the wall with openings. It was concluded that pier with steel member results in 2.5% more displacement with crumbling shows the ductile response. In this paper, only the in-plane behaviour of masonry piers were considered and strengthening results shows better change in stiffness and ductility.

A report by Navalli (2001) in Uttaranchal suggested to utilize flat timber groups at some vertical interim to enhance the integrity of the brickwork structure. These houses undergo little damages during the October 1991 Uttarkashi earthquake as compare to masonry structure without horizontal timber band. A paper by Arioglu and Anadol (1973) and Jai Krishna and Arya (1962) also refers to such practices.

Tianyi *et al.* (2006) A full-scale two-story URM building was tested in order to examine its lateral resistances and output shows that test URM structure shown large initial stiffness, but this stiffness rapid decreases rapidly with small increasing in lateral deformation. Major conclusions obtained by him that damage to URM was found due to large cracks development at the interfaces between masonry mortar and brick. Failure of first story influenced by the failure mechanisms of sliding and rocking of the first-story piers. This paper also concluded that major modifications are needed in the *FEMA 356* technique.

2.3 Analytical Research

Pasticier *et al*. (2008) carried out the pushover analysis of URM by using SAP2000 software and research the convenient outcomes offered by SAP2000 programming with easy to understand interface, for seismic investigations of URM structures. For this, they carry out the SPO analysis of URM building from Italy in SAP 2000 which is already analysed by some other researchers by using advanced programs and results were compared with experimental and outcomes advanced programs. Later on, two storey stone masonry building from northeast of Italy were modelled and SPO curve was developed for two different loadings. Finally, fragility curve was developed by considering seismic input as random variable and variability in mechanical properties of masonry is ignored.

Duan and Pappin (2008) give a procedure for establishing the required fragility curves for various damage states, in particular for the more severe damage states, based on nonlinear pushover analysis results. A solution is proposed for overcoming the difficulty encountered when determining the median spectral displacements for the more severe damage states. An example is given to illustrate the entire process. The proposed procedure has been successfully applied by the authors in recent seismic loss estimate studies of modern cities with densely populated buildings in regions of moderate seismicity.

Park *et al.* (2009) carried out seismic analysis of low rise unreinforced masonry building. Develop fragility curve for two story URM in the region of southern US. They proposed structural modelling method that can be effectively used for fragility analysis without a significant increase in computational time, and maintains an acceptable level of accuracy in representing the nonlinear behaviour of the structures. Developed fragility curve is compared with the HAZUS methodology.

Rota *et al*. (2010) has proposed a new analytical method for the development of fragility curves for URM buildings. It is the probabilistic approach in which mechanical properties are considered as random variables. Since variation in masonry properties is also important for seismic performance. This method is based on nonlinear stochastic analyses of building prototypes. The mechanical properties of the prototypes are considered as random variables, assumed to vary within appropriate ranges of values. Monte Carlo simulations are then used to generate input variables from the mean and coefficient of variance. The model created and nonlinear analyses are performed. In particular, nonlinear static (pushover) analyses are used to define the probability distributions of each damage state whilst nonlinear dynamic analyses allow to determine the probability density function of the displacement demand corresponding to different levels of ground motion. By using complex convolution process cumulative distribution of demand and the probability density function, for different damage states allows deriving fragility curves.

Lagomarsino *et al*. (2013) carry out nonlinear analysis of unreinforced masonry building by using equivalent frame modelling method in TREMURI program. They found that equivalent frame method easy and simple because it permits the user-friendly analysis of complete 3D URM structure with less computational efforts and this method is also suitable for engineering practical use. He presents the solutions adopted for the implementation of the equivalent frame model in the TREMURI program for the nonlinear analysis of masonry building.

A paper by Sonekar and Bakre (2015) presents a comparative study on the non-linear behaviour of masonry frame structures when subjected to earthquake excitation under different lateral loading pattern. Equivalent Frame Model (EFM) is being used for modelling the non-linear behaviour of masonry by providing flexural and shear hinges in the model. Higher strength estimates are obtained for uniform load pattern along the height of the structure out of three lateral load pattern while mode and parabolic lateral load patterns are found to be always equivalent (i.e. around 15% higher). Failure due to shear is found to be main criteria for failure of URM frame structures. Spectral displacement is seen to be more in the weak direction as compared to the strong direction (i.e. around 64 % less), stating, stronger and stiffer construction displaces less than weaker.

Bhosale *et al.* (2016) carry out the sensitivity analysis of structure with masonry infill. The variation in material properties greatly affects the seismic performance of the structure. They found out that how much lateral behaviour is sensitive to various properties of masonry. The main reason to carry out the sensitivity analysis is to find out the most sensitive parameter that affects the lateral response of the building. In this paper sensitivity analysis is carried out by considering 5% mean and 95% probability value based on mean and coefficient of variance of a random variable in the in-fills characteristics, to find a sufficient range of results representing a wide number of possible situations that can be met in practice. They used pushover curve and base shear at yield considered as sensitivity parameter. The results obtained shows that all other mechanical strength-related properties of masonry and concrete have shown a significant effect on the lateral structural performance except the tensile strength of concrete. Tornado diagram used to represent the sensitivity analysis result.

Hazus – MH 2.1 (2003) is the technical and user's manual developed by department of homeland Security Federal Emergency Management Agency (FEMA). It describes the procedure for the development of building specific damage and loss function with advanced engineering building module. This code also gives the probabilistic method for the development of fragility curve which is based on the several variables for different damage state. It gives uncertainty associated with different damage state. In order to avoid the complex convolution process, Hazus has given pre-calculated values for total variability used for the development of fragility for different damage states.

2.4 Summary

This chapter gives the overall idea about work done by various researchers in the field of URM. Review of literature is divided into two section: (a) Experimental Research and (b) Analytical Research. The idea about different types of failure, which occurs during an earthquake, variation in the properties of masonry, the effect of lateral load pattern on the behaviour of URM based on experimental and analytical research work are presented in this chapter. Codal provisions for the development of fragility are also given.

Chapter 3

Pushover Analysis

3.1 Overview

In 1970, though, the use of the nonlinear static analysis (pushover analysis) came in to practice but for last 10-15 years, its importance has been recognized. The use of this method is mainly found in estimating the drift capacity and strength of existing structure and the seismic demand for this structure subjected to selected earthquake. Further, its application can be fruitful in checking the adequacy of newly designed structures Owing to the ease in computation and effectiveness of static pushover analysis (SPO), in the last few years it has brought several seismic guidelines like FEMA 356, ASCE/SEI 41-06 and ATC 40 and design codes like PCM 3274 and Eurocode 8 into practice.

Pushover analysis is defined as a nonlinear static method of analysis where a mathematical model directly incorporates the nonlinear load-deformation characteristics of individual components and elements of the structure which are subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a ‘target displacement’ is exceeded. Although it is a nonlinear static method, it is stepwise linear because lateral load increases monotonically at the same time stiffness matrix get modified for the reduced stiffness in between this two steps it behaves as linear.

The structure is pushed up to target displacement which is the maximum displacement i.e. elastic plus inelastic deformation of the building at control node which is generally considered at the roof expected under selected earthquake ground motion at which, yielding of members takes place. It is an important tool to assess the structural performance by estimating the force and deformation curve and seismic demand can be estimated by a nonlinear static analysis algorithm. By knowing the storey forces, storey drifts, component deformation and component forces and sequence of yielding one can easily predict, how the structure will perform when the earthquake comes. It is an approximate method because the earthquake is uncertain and performance of structure in SPO analysis is based on several factors such as lateral load pattern and uncertainty in masonry properties influence the output of SPO analysis. There are several outputs obtained from pushover analysis listed below:

a) Estimate force at which yielding of member takes place and ultimate force at which failure of structure takes place.
b) Estimate yield displacement at which fine cracks develop and ultimate displacement at which failure takes place.
c) To ascertain the sequential yielding of the members and the progress of the overall capacity curve.
d) By knowing the sequence of member yielding one can identify the critical regions, where the inelastic deformations are expected to be high and identification of strength irregularities (in the plan or in elevation) of the building.

Pushover analysis delivers all these benefits for an additional computational effort (modelling nonlinearity and change in analysis algorithm) over the linear static analysis. This is an important tool to check the performance of existing structure when the earthquake comes for retrofitting purpose and for new construction for strengthening purpose. Step by step procedure of pushover analysis is discussed in next content.

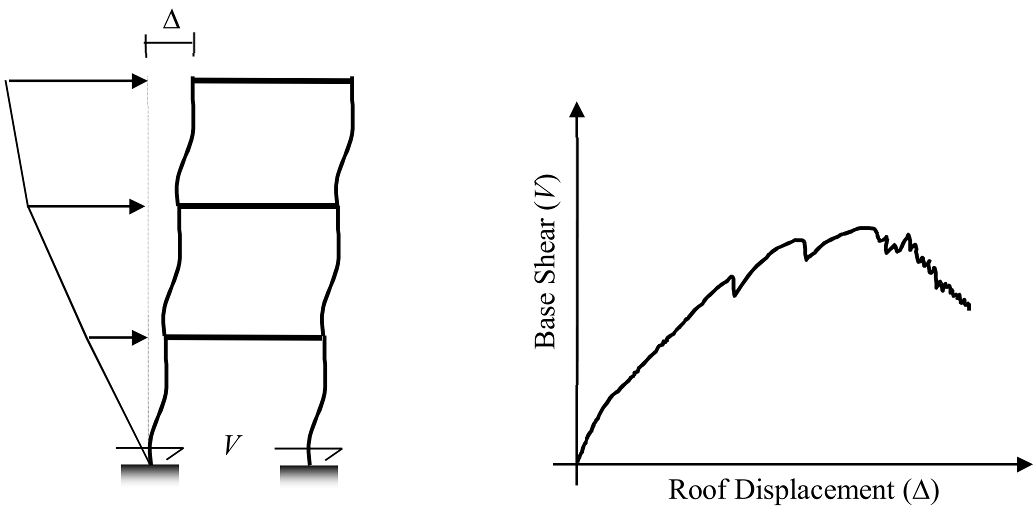
3.2 Pushover Analysis Procedure

Simply pushover analysis is nothing but the pushing the structure with predefined load pattern till the building collapses. Pushover analysis is an approximate method of nonlinear static analysis in which the predefined load pattern as per the codal provision is increased monotonically but while doing so the distribution of load pattern does not change, as shown in Fig. 3.1. The building is displaced till the 'control node' generally considered at roof up to yielding or building collapses. The stepwise procedure is as follows:

a) Creating a model as per the geometry of structure
b) Defining the load patterns i.e. various loads acting on the structure and a nonlinear static load pattern for SPO analysis
c) Assigning the hinges to vertical and horizontal members, for RCC and Steel members hinge properties are already defined in SAP2000 but in case of URM we have to define user defined hinges as per the cross section and mechanical properties and
d) Distributing the lateral load on each storey as per the considered distribution pattern.

After this one can run SPO analysis two times for first time up to the failure of structure in order to know the target displacement and after that running analysis up to target displacement in order to know the seismic demand of the structure. It is to be noted that for RCC and steel members predefined hinges are available in SAP2000 but for URM members we have to define the hinges based on a cross section of the member, properties of masonry and location of this hinges are based on the failure pattern observed from experimental research.

The lateral performance of the building is much sensitive to applied load pattern and selection of control node. Generally, by default software takes the topmost left side node as a control node. For the selection lateral load pattern in SPO analysis, various guidelines are given in FEMA 356 is explained in Section 3.2.1.



(a) Structural model (b) Pushover curve

Fig. 3.1: Simplified representation of pushover analysis procedure

3.2.1 Lateral Load Profile

The lateral behaviour of the structure is much sensitive to load pattern applied during analysis because of results obtained from analysis *i.e.*, base shear versus roof displacement, the yielding sequence of members, are very sensitive to the load pattern. Different codes specified different load pattern to carry out SPO analysis of the structure. These load pattern based on various factors like the magnitude of ground motion and type of earthquake and yield, stiffness characteristics change during earthquake response.

Several investigations are done by Gupta and Kunnath (2000) and Mwafy and Elnashai (2000) concluded that trapezoidal and triangular shape lateral loads are best suited for dynamic analysis but the results are accurate up to the elastic range, whereas for large deformations the results obtained from uniform distribution are in close agreement with dynamic response of the structure. It can be concluded that single load pattern fails to capture variation in structural behaviour under earthquake loading. At least two different load pattern should consider for SPO analysis is recommended by FEMA 356 code. The reason behind to use of two different lateral load patterns is to get the overall idea of response of structure *i.e.,* maximum and minimum response during actual dynamic. Since earthquake is uncertain, during analysis two different load pattern are considered. One is the inverted triangular and another one is a uniform distribution so that the response of structure lies in between. FEMA 356 specified to select one load pattern from each of the following two groups:

Group – I:

a) Code-based vertical distribution of lateral forces used in the equivalent static analysis (permitted only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration).
b) When more than 75% of the total mass participates in a mode a vertical distribution which is proportional to the shape of the fundamental mode in the direction under consideration should be used.
c) Considering a sufficient number of modes which capture minimum 90% of the total building mass and distributing proportional to story shear distribution which is calculated by a combination of modal masses as per response spectrum analysis. When the fundamental period of the mode of vibration exceeds 1.0 second this distribution should be used.

Group – II:

a) Distribution of force proportional to mass at each story also known as uniform distribution.
b) An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution shall be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

Instead of using the uniform distribution to bind the solution, FEMA 356 also allows adaptive lateral load patterns to be used but it does not elaborate the procedure. Although the adaptive procedure may yield results that are more consistent with the characteristics of the building under consideration it requires considerably more analysis effort. Fig. 3.2 shows the various lateral load pattern used in SPO analysis.

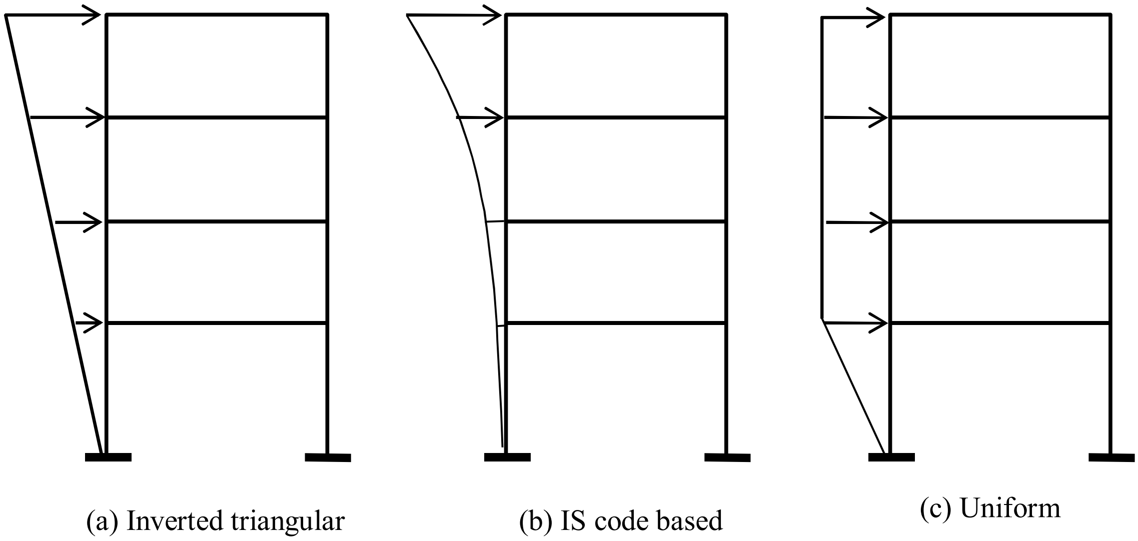


Fig. 3.2: Different load pattern for SPO analysis as per FEMA 356 (considering uniform mass distribution)

3.2.2 Target Displacement

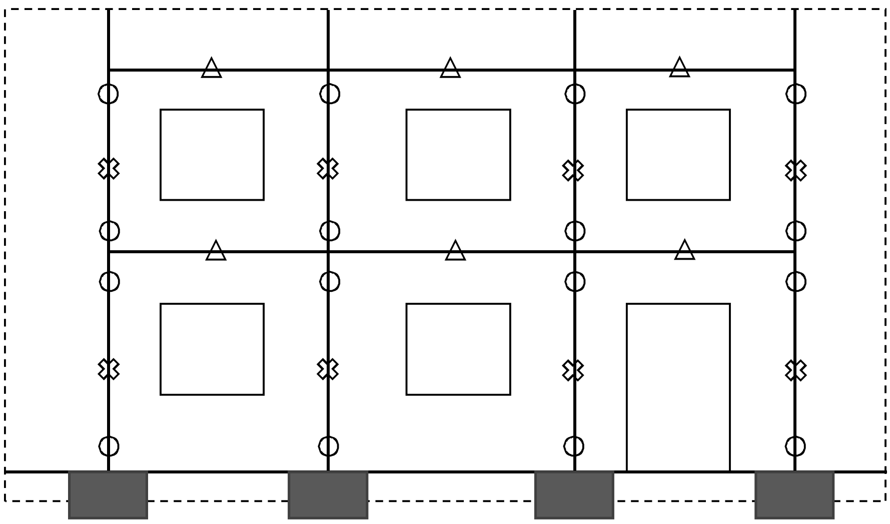
Target displacement can be defined as maximum deflection at a point considered generally known as control node with respect to the response to the earthquake. Different performance level such as immediate occupancy, life safety and complete collapse are obtained from the pushover curve i.e. load verses top displacement. Therefore, the good knowledge of the target displacement results in good achievement in pushover curve. Various methods to find out target displacement are as follow:

a) CSM: Capacity Spectrum Method and
b) DCM: Displacement Coefficient Method

These methods use pushover curve to calculate global displacement demand on the building from the response of an equivalent single-degree-of-freedom (SDOF) system. The technique used is the only difference in these two methods.

3.3 Equivalent Frame Modelling

EFM is nothing but the modelling the wall as a combination of vertical and horizontal members just like column and beam in RCC building. Where vertical member is known as the pier and horizontal member as a spandrel. Walls with an opening can be divided into horizontal and vertical members which combine to represent the complete wall depicted in Fig. 3.3. This modelling method is commonly known as equivalent frame modelling (EFM). Provided hinges allow the structure to undergo inelastic shear and flexural deformation to predict the actual behaviour of the structure during an earthquake. In the present study, EFM is used to model the wall. The plastic hinges were used in SPO analysis are as per given in Pasticier *et al.* (2008) since it allows the user to accurately follow the structural behaviour not only up to the elastic limit but in inelastic limit up to failure. The hinges were modelled based on failure mechanism as shown in Fig. 1.3 and various experimental results obtained for URM to actually represent the nonlinear behaviour (Magenese *et al.* 1995). The followed modelling is given in section 3.3.1.



○ Rocking hinge ⯐ Pier shear hinge △ Spandrel shear hinge

Fig.3.3: EFM with hinges

3.3.1 Non-Linear Hinge Modelling for the SPO Analysis

The standard load–deformation curve, which can be used in the SAP2000 plastic hinges, is shown in Fig. 3.4 (a). The masonry piers were modelled as elasto-plastic with final brittle failure as shown in Fig. 3.4 (b) and (c) by providing one ‘shear hinge’ at mid-height and two ‘rocking hinges’ at the end of the deformable parts and. In case of spandrel on shear hinge at centre with hinge properties shown in Fig. 3.4 (d) and (e). For all plastic hinges rigid-perfectly plastic behaviour is assumed with final brittle failure.

To define the hinge properties in terms of ultimate moment M_u and ultimate shear V_u can be calculated by using the following equations. Ultimate moment capacity defined by Equation 3.1. For shear strength capacity, two equations were given by Magnese *et al.*, 1997 based on the experimental results. Equation 3.2 is used for existing buildings given by Turnsek and Cacocic, 1971 based on failure with diagonal cracking. For new buildings equation given in The Italian Official Gazette, 2003 is used to find ultimate shear strength based on failure occurs due to sliding referring Equation 3.3. Although formulated differently, such a criterion is also recommended by the Eurocode.

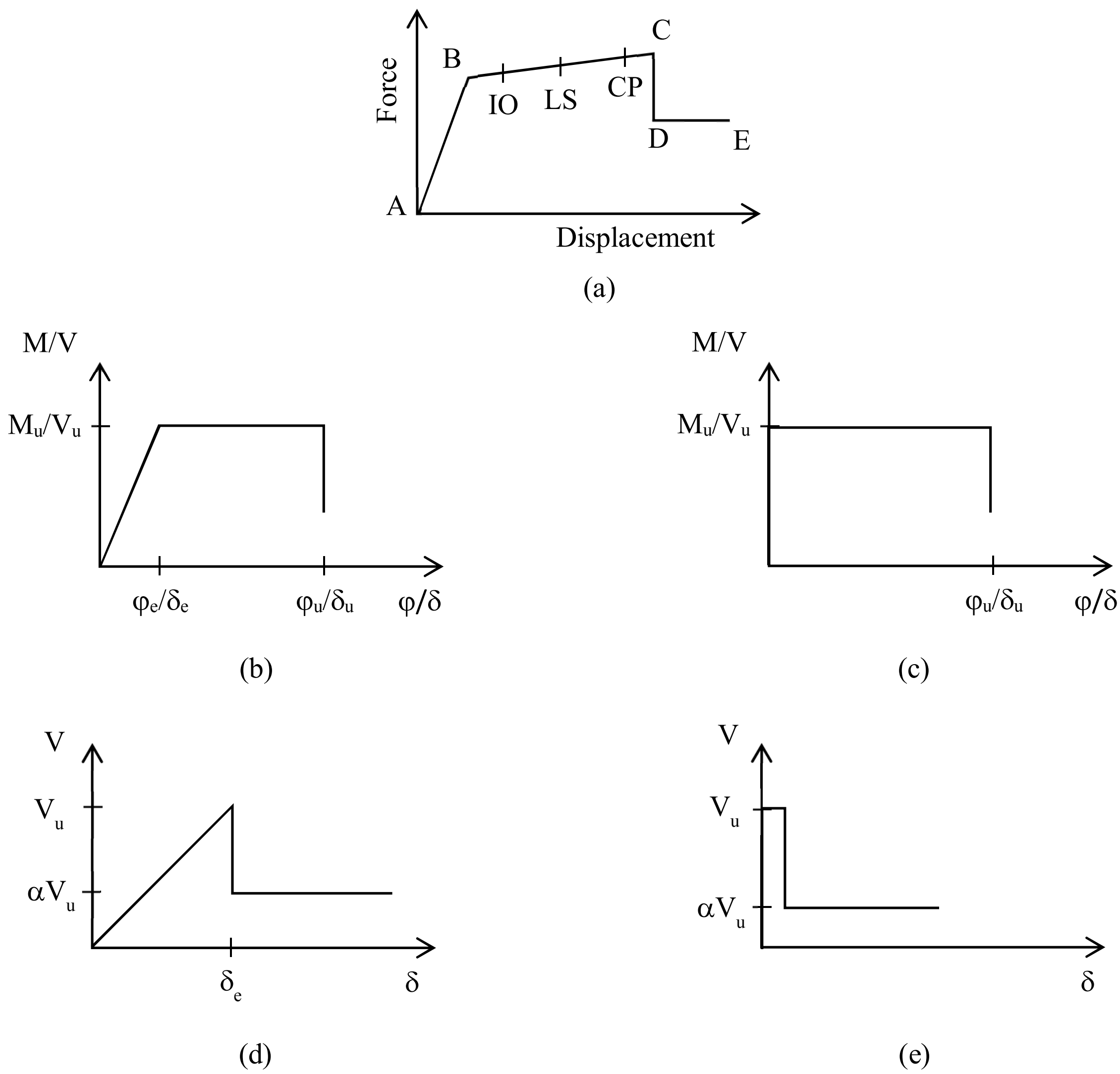


Fig. 3.4: (a) Standard load versus deformation curve in SAP2000 for the plastic hinge; (b) and (c) assumed behaviour for the entire pier and the corresponding plastic hinge respectively; (d) and (e) assumed behaviour for the entire spandrel beam and the corresponding plastic hinge.

Equation (3.1) gives ultimate moment capacity for rocking hinge and Equations (3.2) and (3.3) gives ultimate shear capacity for the shear hinge, minimum of this two value should be considered for ultimate shear hinge capacity.

$$M_u = \frac{\sigma_0 D^2 t}{2}\left(1 - \frac{\sigma_0}{k f_m}\right) \qquad (3.1)$$

$$V_u^f = \frac{1.5 f_{v0d} D t}{\varepsilon}\sqrt{1 + \frac{\sigma_0}{1.5 f_{v0d}}} \qquad (3.2)$$

$$V_u^s = \frac{1.5 f_{v0d} + \mu_f \dfrac{\sigma_0}{\gamma_m}}{1 + \dfrac{3H_0}{D\sigma_0} f_{v0d}} Dt \qquad (3.3)$$

where:

Mu is the ultimate moment for rocking hinges and V_u^f and V_u^s are the shear strength considering failure with diagonal cracking and failure with sliding respectively,

σ_0 = mean vertical stress,

D = pier width,

t = pier thickness,

k = coefficient taking into account the vertical stress distribution at the compressed toe (a common assumption is an equivalent rectangular stress block with k= 0.85),

f_m = design compression strength,

f_{v0d} = design shear strength with no axial force,

μ_f = friction coefficient,

ε = coefficient related to the pier geometrical ratio

ε =1 when H_0/D< 1

=H_0/D when 1 <H_0/D<1.5

=1.5 when H_0/D>1.5

H_0 = effective pier height (distance between two rocking hinges), and

γ_m = safety factor (assumed to be equal to 2).

To define hinge property, the maximum rotation ϕ_u corresponds to a maximum lateral displacement δ_u is given in Equation 3.4 and for shear hinge maximum shear displacement can be calculated as per Equation 3.5 as recommended in Pasticier *et al.* (2008). The behaviour assumed for entire pier and corresponding plastic hinge as shown in Fig. 3.5 (b) and Fig. 3.5 (c).

$$\phi_u = \frac{0.8}{100} h_d - \delta_e \quad \text{(3.4)}$$

$$\delta_u = \frac{0.4}{100} h_d - \delta_e \quad \text{(3.5)}$$

where:

ϕ_u = Ultimate rotation,

h_d = Deformable height of pier,

δ_e = Lateral elastic deflection and

δ_u = Ultimate shear displacement.

Pier will fail when the first of maximum rotation or displacement occurs based on this failure can be classified as a rocking failure or shear failure. Due to the formation of hinges in model one can describe which type of failure is occurring in the pier. Since in SAP2000 it is not possible to automatically control the total deflection of an entire macro-element if more than one of its plastic hinges exceed the elastic limit, such a quantity was manually checked on every macro-element at the end of each load step.

To model the spandrel *i.e.*, horizontal member one shear hinge is provided at the centre of spandrel whose ultimate shear strength can be calculated by the equation given below:

$$V_u = htf_{v0d} \quad \text{(3.6)}$$

where:

h = spandrel depth,

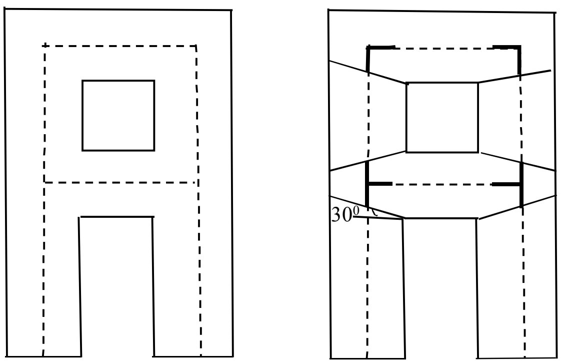
t = spandrel thickness,

f_{v0d} = design shear strength with no axial force.

A brittle–elastic behaviour with residual strength after cracking equal to one fourth of the maximum strength was assumed for the entire element, with no limit in deflection.

3.3.2 Determination of Effective Height of Masonry Piers

After modelling wall as a combination of piers and spandrels, it is important to find out the effective height of pier in order to know the aspect ratio. Effective height is simply the distance between two rocking hinges. There are two approaches to locate the rocking hinge. One approach is rigid offset (RO) which considered spandrel and pier interaction as fully rigid and hinges should be provided a junction of pier and spandrel. The second method given by Dolce, 1989 is to take the portion of pier-spandrel interaction as rigid and hinge should be provided at the intersection of 30^0 inclined line from openings and pier centre line as shown in Fig. 3.6 (b) and (c). Results shows that rigid offset results in higher strength estimation hence in the present study Dolce offset is used to calculate the effective height of pier.



(a) (b) (c)

Fig. 3.5: (a) EFM, (b) EFM with Dolce RO, (c) EFM with full RO

3.4 Validation of Equivalent Frame Model

To understand the concept of EFM and to check reliability of the EFM, model was first investigated by carrying out SPO analysis of a wall, which is already analysed by other researchers using advanced programs. The plan of the ground floor is displayed in Fig. 3.6 for the analysed building. This is a stone masonry house typical of the north-east of Italy (Pasticier *et al*. 2008). Having wall thickness 0.5m all dimensions are in a meter.

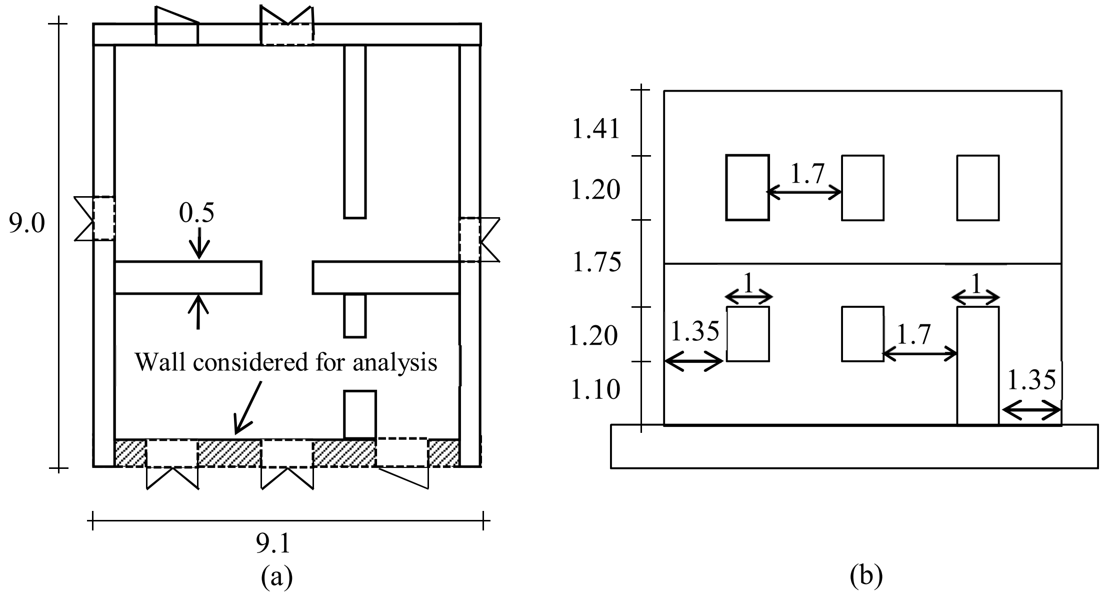


Fig. 3.6: (a) Plan and (b) Elevation of selected stone masonry building (Pasticier *et al*. 2008)

The design values for the mechanical properties are based on the mean values measured in the situ on a number of similar buildings located in the same area as shown in Table 3.1 (Pasticier *et al.* 2008).

Table 3.1: Material properties of selected wall for validation

Properties	Values
E (Young's modulus)	1600 N/mm^2
G (Shear modulus)	640N /mm^2
ϒ (unit weight)	1900 kg/m^3
f_m (design compression strength)	0.8 N/mm^2
f_{v0d}(design shear strength)	0.042 N/mm^2
μ_f (friction coefficient)	0.5

The EFM of the above masonry structure in SAP2000 is shown in the Fig. 3.8, where the vertical members are called as piers and horizontal members are spandrels.

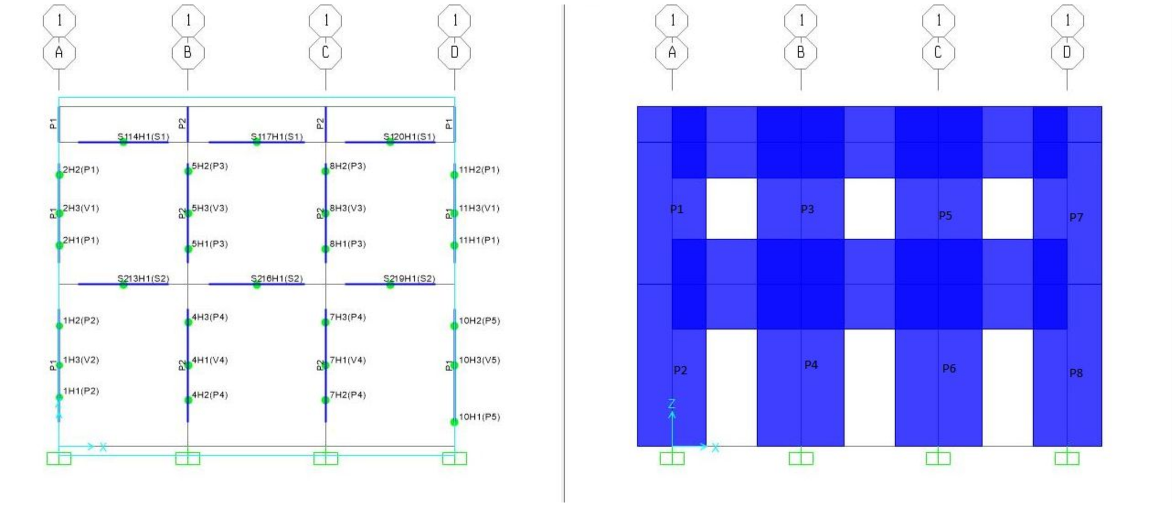


Fig. 3.7: EFM in SAP2000

3.4.1 SPO Analysis for Validation

The selected wall described in above section was analysed by EFM and results were compared with the results from the literature. The EFM with hinges assigned to it is shown in Fig. 3.8. An initial linear analysis of a model for dead loads is done to get an axial load on each pier and vertical pressure coming on them. The cross section properties of each piers are determined. The calculated values of rocking and shear hinges were found out using Equations (3.1), (3.2) and (3.3) are given in Table 3.2. P1, P2, P3, P4, P5, P6, P7 and P8 are names given to piers i.e. vertical members.

Table 3.2: Flexural and shear hinge properties

Pier name	Aspect ratio $\varepsilon = \frac{H}{D}$	Axial stress (σ_0) (kN/sq.m)	Ultimate moment (*Mu*) (kN-m)	Ultimate shear(V_u^f) (kN)	Ultimate shear (V_u^s) (kN)	Ultimate rotation ((ϕ_u) (radian)	Ultimate lateral deflection δu (mm)
P1	1.465	92.711	36.482	37.861	17.844	0.016	7.912
P2	1.465	186.563	61.682	48.883	32.832	0.016	7.912
P3	1.282	109.765	66.504	57.682	28.027	0.017	8.720
P4	1.282	216.118	106.519	74.633	49.400	0.017	8.720
P5	1.282	109.765	66.504	57.682	28.027	0.017	8.720
P6	1.282	216.118	106.519	74.633	49.400	0.017	8.720
P7	1.465	92.711	36.482	37.861	17.844	0.016	7.912
P8	1.992	186.563	61.682	35.958	28.438	0.022	10.756

The pushover curves were obtained for the wall for two different lateral loadings (a) inverted triangular distribution (SPO1), (b) uniform distribution (SPO2), as recommended by recent codes of practice and regulations. Table 3.3 represents the lateral load distribution at each floor. The outcomes of the numerical comparisons are displayed in Figs. 3.8 and 3.9 for analysed wall. Both pier and spandrels were modelled as described earlier.

Table 3.3: Seismic weight and distribution of the lateral forces

Floor	Seismic weight, W_i (kN)	Seismic force/base shear ratio at each floor, $F_i/\sum F_i$	
		Inverted triangular distribution	Uniform Distribution
1^{st}	278.7	0.67	0.47
Ground	281.5	0.33	0.33

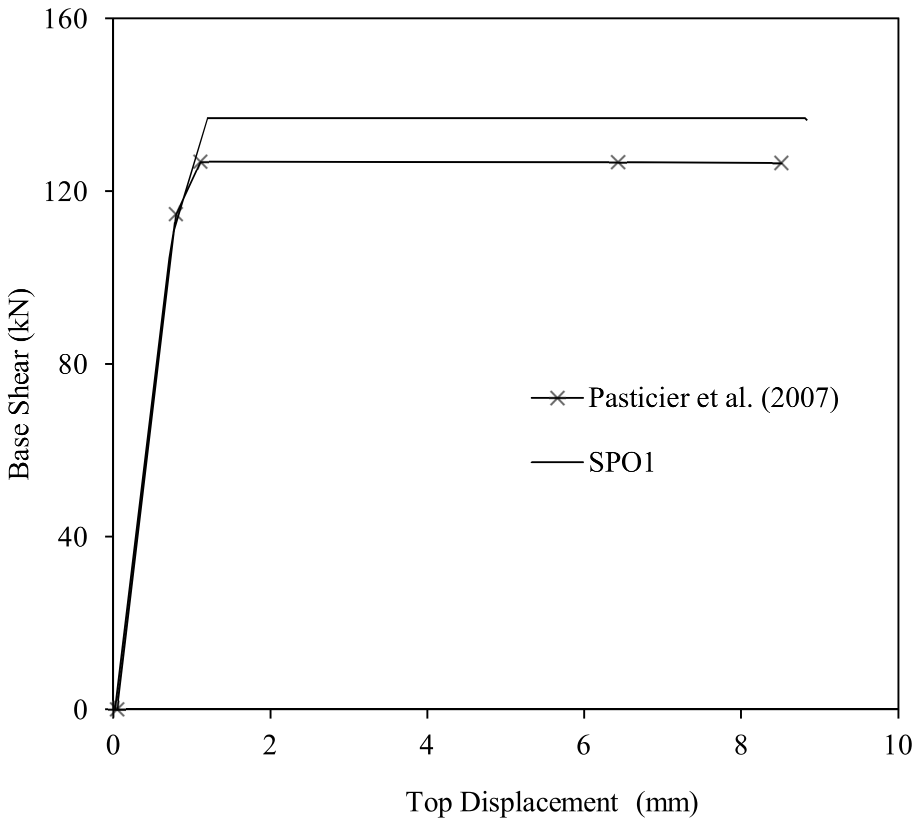


Fig. 3.8: Pushover curve for inverted triangular distribution

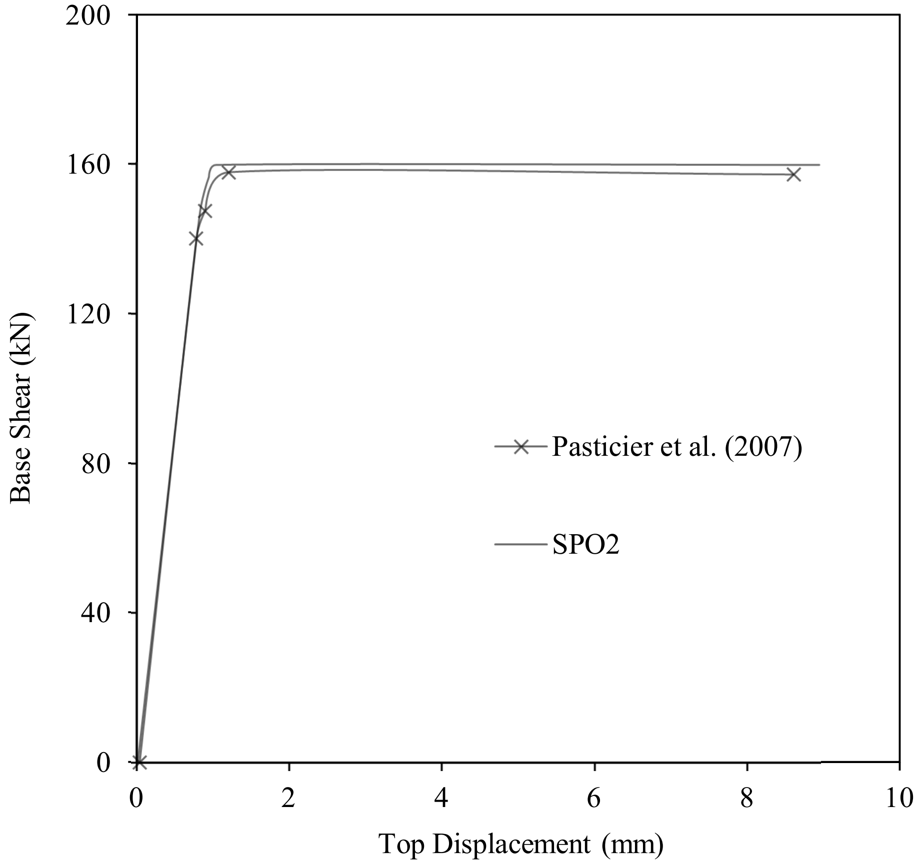


Fig. 3.9: Pushover curve for uniform distribution

3.4.2 Validation Results

Result from Pasticier *et al*., (2007) and obtained from the analysis are shown in Table 3.4. The top displacement was almost the same that detected by Pasticier *et al*., (2007) with maximum 7.8% of error in the base shear result for inverted triangular distribution. Failure pattern for different load pattern is as shown in 3.10.

Table 3.4: Comparison of base shear

Base shear (kN)	Lateral load pattern	
	Inverted Triangular	Uniform
Pasticier *et al.,* (2007)	126.54	157.15
Present study	136.51	159.65
Error in Base shear	7.8%	1.5%

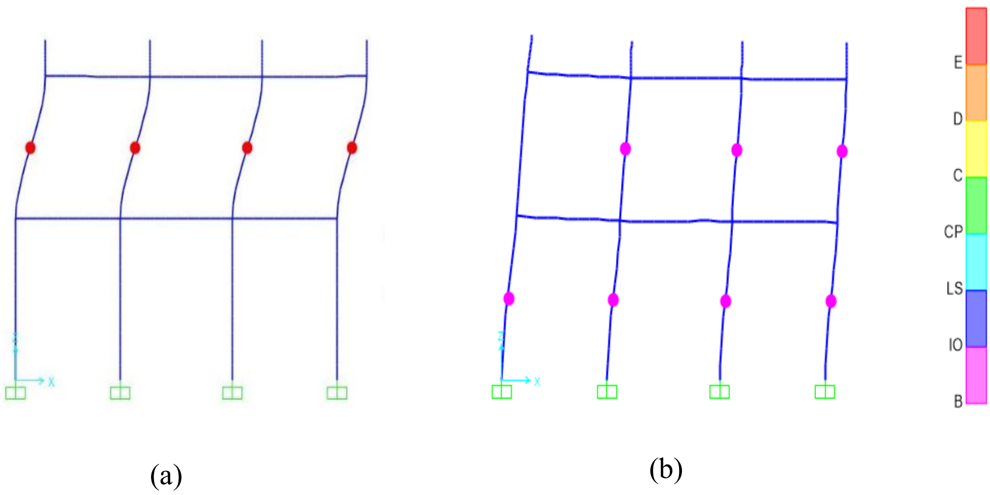


Fig 3.10: Deformed shape and hinge formation (a) SPO1 analysis with inverted triangular distribution and (b) SPO2 analysis with uniform distribution

3.5 Summary

This chapter describes the SPO analysis procedure and various technical terms used in SPO. EFM is simple, easy method to carry out SPO analysis of URM masonry. In order to validate the EFM, the analysis is done and results are compared with the results of Pasticier *et al.*, 2007. Present equivalent frame model presents the strength and displacement in close agreement with literature. Therefore, the present model can be considered as valid.

Chapter 4

Structural Modelling

4.1 Introduction

This chapter begins with the geometric details of the selected wall and masonry properties used in the study. The second part of this chapter presents the SPO analyses for AAC and CLC and comparative study of the lateral behaviour of the wall for different masonry which is analysed by EFM. SPO analysis is carried out considering dead load only.

4.1.1 Geometric Modelling of Masonry Wall

A detailed pushover analysis of the two story unreinforced masonry having door and window openings is carried out, by using equivalent frame modelling. Modelling of the wall is done as per described in Chapter 3. The plan and elevation of the wall is as shown in Fig. 4.1. All windows are of the same size and having a wall thickness equal to 0.25m.

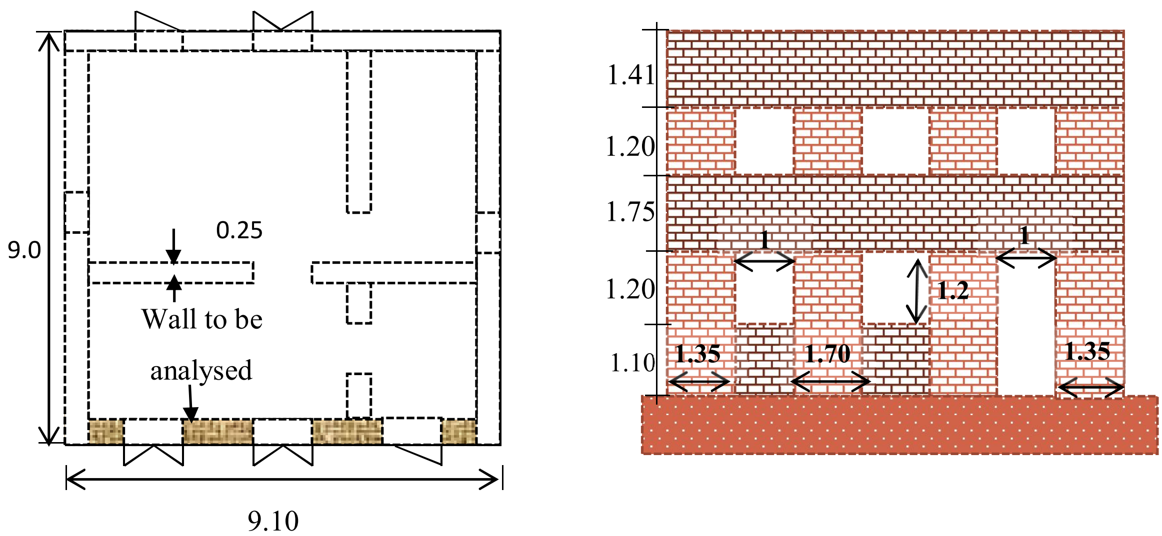


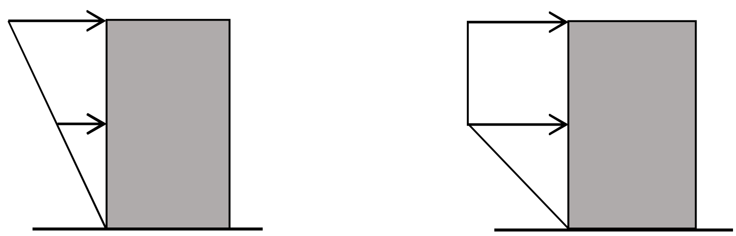
Fig. 4.1: Plan and elevation of masonry wall

(All dimensions are in metres.)

4.1.2 Modelling in SAP2000

Three hinges are provided for each pier i.e. one shear hinge at centre and two rocking hinges at the end of the pier. In case of spandrel one shear hinge is provided at the centre. Perfectly rigid plastic behaviour with final brittle failure was assumed for all these plastic hinges. The hinge properties in terms of the ultimate moment and ultimate rotation or ultimate shear and ultimate shear displacement were calculated as per equations 3.1, 3.2 and 3.3 as described in Chapter 3.

Cross-section of each pier and spandrel were found out based on the geometry of the structure and modelled in SAP2000 by using equivalent frame modelling concept. Wall is shown in Fig. 4.1 were analysed considering different masonries like clay masonry, Fly ash masonry, AAC and CLC masonry. Analysis is done for two different lateral load pattern, one is inverted triangular i.e. proportional to the product of the masses by the floor heights and another one is uniform distribution i.e. proportional to the floor masses used in the present study, as shown in Fig. 4.2.



(a) SPO1 (b) SPO2

Fig. 4.2: Load pattern considered in the present study (a) inverted triangular and (b) uniform

4.2 Material Properties

The mechanical and physical properties for masonry like density, modulus of elasticity, Poisson's ratio, shear modulus, compressive strength and design shear strength are taken from the literature review. Coefficient of friction is taken as 0.5 for all masonry since no standard value for coefficient of friction for masonry is available

4.2.1 Clay Masonry Properties

The material properties used in the present study for clay masonry are presented in Table 4.1. These properties are based on test conducted in lab, it can be used for new construction but for old masonry structure some laboratory and in-situ test should be performed to find these mechanical properties.

Table 4.1: Clay masonry properties

Property	Variable	Mean	Source
Density (kN/m^3)	γ	18.84	Park *et al.* (2009)
Masonry compressive strength (MPa)	f_m	5	Bakshi and Kamini (2006)
Masonry shear strength (MPa)	f_{v0d}	0.18	Park *et al.* (2009)
Elastic modulus (MPa)	E_m	4200	Park *et al.* (2009)
Poisson's ratio	μ	0.07	Bosiljkov *et al.* (2005)

4.2.2 Fly Ash Masonry Properties

For fly ash, masonry properties were taken from Teja (2015). He has given compressive strength and shear strength of fly ash masonry considering different cement-mortar ratio given in Table 4.2 and 4.3 and gives the relationship between modulus of elasticity with compressive strength and shear modulus with design shear strength as given in equation below. The density of fly ash (γ) = 17.31 kN/m^3.

$$E_m = 600 f_m \quad (4.1)$$

$$G_m = 6226 f_{v0d} \quad (4.2)$$

Poisson's ratio is calculated from the basic relation between Young's modulus, shear modulus and Poisson's ratio as given in Equation 4.3.

$$E_m = 2G_m(1+\mu) \quad (4.3)$$

Table 4.2: Designation and mix proportions of different grades of mortar for fly ash

Designation (Cement: Sand)	Mix proportion	Characteristic
CM1	1:6	Weak Mortar
CM2	1:4.5	Intermediate Mortar
CM3	1:3	Strong Mortar

Table 4.3: Fly ash masonry properties (Teja, 2015)

Designation	f_m (MPa)	E_m (MPa)	f_{v0d} (MPa)	G_m (MPa)	μ
CM1	1.86	1116	0.112	697.312	0.200
CM2	2.87	1722	0.113	703.538	0.224
CM3	3.76	2256	0.171	1064.646	0.060

4.3 SPO Analysis

4.3.1 SPO Analysis for Clay and Fly Ash Masonry

The SPO curves are obtained for the wall shown in Fig. 4.1 using the same procedure explained in Chapter 3. The analysis is done by considering dead load only. First dead load analysis is run to find out mean vertical stress (σ_0). Two SPO analyses, designated 'SPO1' and 'SPO2' are correspond to two different load pattern inverted triangular and uniform distribution respectively. Analysis is carried out on the same wall for two different masonries one is clay masonry (CLM) and another one is considering as Fly ash masonry (FAM), therefore total four analyses were carried out as shown in Fig.4.3. Both piers and spandrels were modelled as described in Chapter 3.

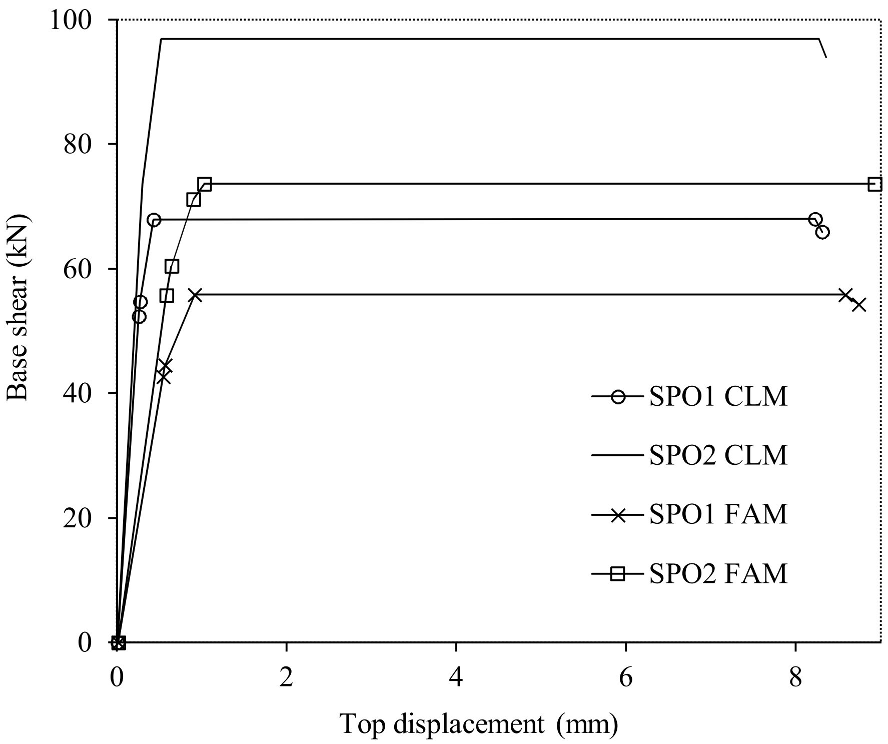
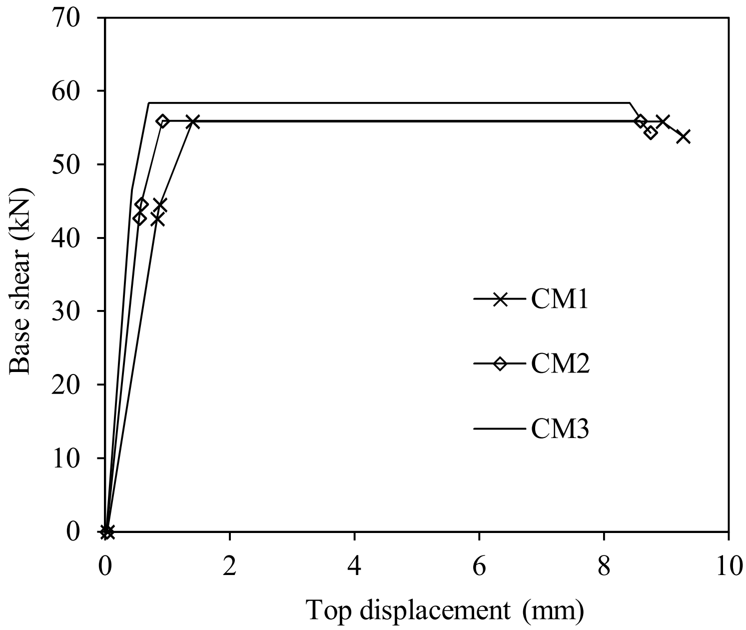


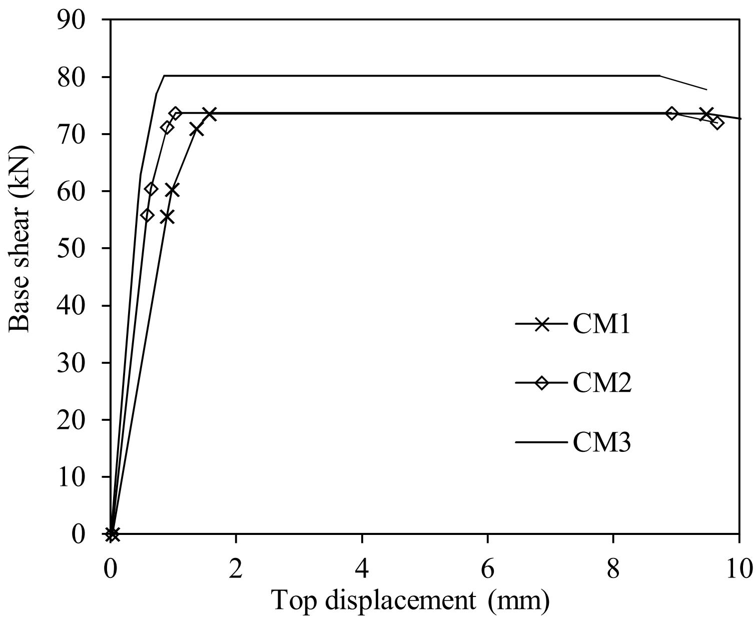
Fig. 4.3: SPO curves for clay and fly ash masonry

4.3.2 SPO Analysis for Fly Ash Masonry for Different Grades of Mortar

To see the effect of mortar grades on pushover curve, three mortar grades designated as CM1, CM2 and CM3 given in Table 4.3 are used to develop a model and hinge properties. SPO analyses was carried out by modelling as per the properties given in Table 4.3 and pushover curves were developed for two different lateral loads SPO1 and SPO2 as described in Section 4.3.1. SPO curves are as shown in Fig. 4.4 (a) and (b).



(a) SPO1



(b) SPO2

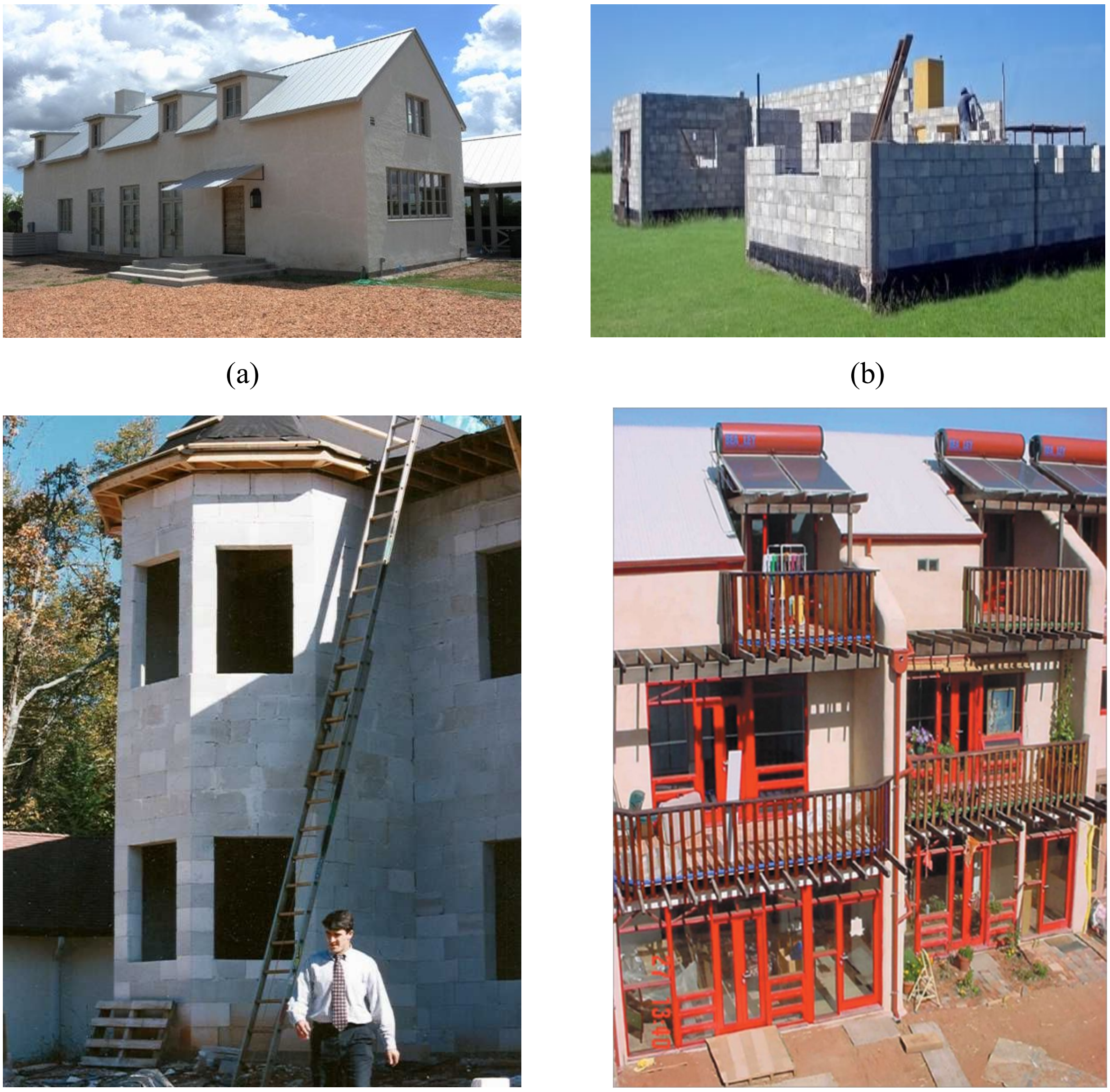
Fig. 4.4: SPO curves for different mortar grades

4.4 AAC and CLC Masonry

AAC and CLC stands for Autoclave Aerated Concrete (AAC) and Cellular Lightweight Concrete (CLC). These are the popular types of lightweight concrete brick currently used in construction industry. These lightweight concrete bricks are used to replace the traditional bricks like clay brick and fly ash brick as infill material for the wall in frame buildings. The advantage of using lightweight material to replace the traditional brick is to reduce the dead load of the building, hence reduces the size and increase the capacity of the structural member. In India, these bricks are now using for infill but in foreign these bricks are used as infill also to construct load-bearing structures as shown in Fig. 4.5.

AAC and CLC block have good compressive strength. Although it's density is about one-third of the of normal clay brick it still has half the bearing strength, and load-bearing structures up to two storeys high can be safely constructed with AAC blockwork. To a greater extent, in Australia, AAC is used as cladding system instead of load bearing wall. Construction of complete low-rise building is possible with AAC masonry. In market pre-casted slab, roofing panels, floors and roofing with reinforced lintels are available. AAC and CLC both were made from a similar material such as cement, sand and other materials with the inclusion of air voids. The difference was in the method of manufacture. Because of several advantages over traditional brick, these bricks are now rapidly using in construction practices. The rapid rate of construction, cost effective easy to handle and not much supervision is required for the use of these bricks.

This bricks also have good thermal insulation, fire protection and good appearance. It has uniform shape and flat surface. Due to all this advantages this bricks are becoming more popular.



(a) (b)

(c) (d)

Fig 4.5: Unreinforced AAC structures

(Source: http://www.yourhome.gov .au/materials/autoclaved-aerated-concrete,

Last Accessed: 26 March 2017)

4.4.1 SPO Analysis for AAC and CLC Masonry

Up to now CLC is not using for the construction of load bearing structures but for comparative study SPO analysis is carried out considering both AAC and CLC unreinforced masonry. Wall shown in Fig. 4.1 were modelled in SAP2000 considering AAC and CLC property. Properties required for AAC and CLC masonry to develop EFM are taken from the Bhosale (2017) are shown in Table 4.4.

Modelling of the wall is done as per described in Chapter 3. SPO analysis is carried out for inverted triangular (SPO1) and uniform distribution (SPO2) for AAC and CLC masonry. The response of the structure is shown in Fig. 4.6.

Table 4.4: AAC and CLC masonry properties (Bhosale, 2017)

Property	Variable	Mean	
		AAC	CLC
Density (kN/m^3)	γ	5.58	9.7
Masonry compressive strength (MPa)	f_m	2.23	2.42
Masonry shear strength (MPa)	f_{v0d}	0.22	0.23
Elastic modulus (MPa)	E_m	1610	2418
Shear modulus (MPa)	G_m	643	964

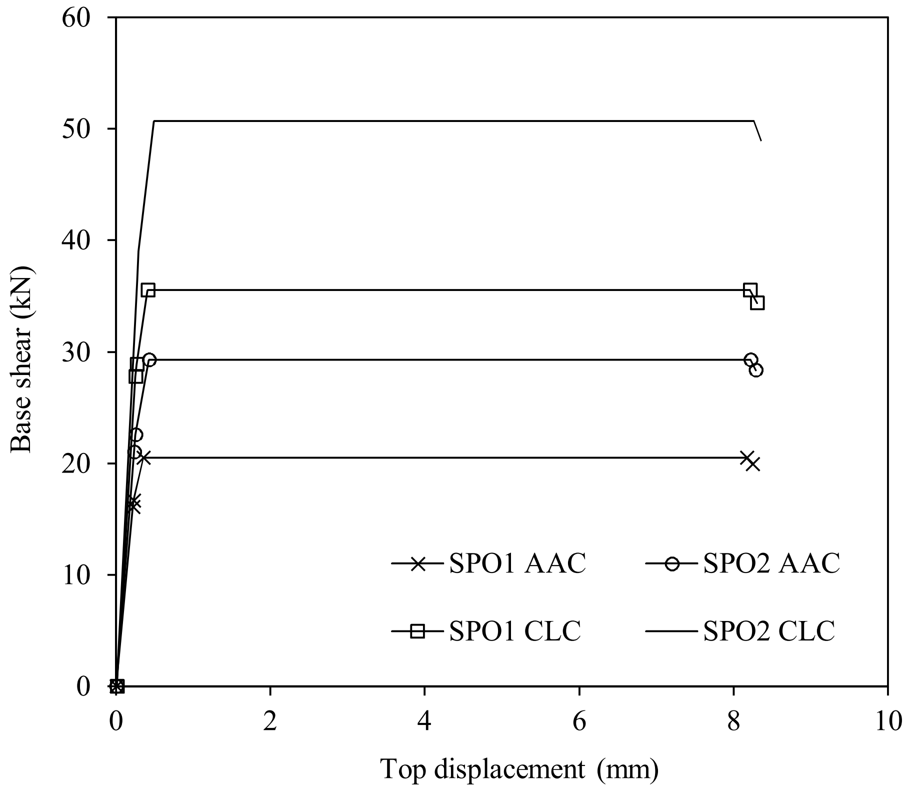


Fig. 4.6: SPO curve for AAC and CLC masonry

4.5 Summary

This chapter begins with description of wall which is to be analysed. The same wall is analysed considering different masonry through-out the study. Details of all masonry properties required for the analysis are given in this chapter along with the source. SPO analysis is carried out for two different loading conditions. Uniform lateral distribution always shows higher base shear strength estimation compare to inverted triangular distribution. In terms of top displacement, both distribution of seismic forces lead to nearly the same value. Obtained SPO curves shows that clay masonry will perform well compare to Fly ash, CLC and AAC masonry as shown in Table. 4.5. Also the effect of cement: mortar ratio on lateral behaviour shows that, for grade CM1 and CM2 there is not much variation in base shear whereas for grade CM3 shear strength is about 20% more compare to CM1 and CM3.

Table 4.5: Variation of base shear and top displacement

Lateral Load Pattern	Base Shear (kN)		Top Displacement (mm)	
	SPO1	SPO2	SPO1	SPO2
Clay Masonry	67.963	96.884	8.22	8.27
Fly Ash Masonry	55.908	73.69	8.57	8.92
CLC Masonry	35.56	50.692	8.21	8.25
AAC Masonry	20.54	29.281	8.16	8.3

Chapter 5

Sensitivity Analysis

5.1 Introduction

As the name itself indicate finding out which input parameter is sensitive for the output behaviour of a structure known as sensitivity analysis. The parameter may be physical or mechanical properties just like density, compressive strength, Young's modulus etc. By changing one property and keeping all other properties as mean finding the change in response of the structure. Sensitivity analysis is the study to know how the input parameters affecting the output parameters. In this study masonry properties like compression strength, shear strength, Young's modulus, shear modulus and density of URM are considered as input parameters in order to know the effect of it, on the lateral behaviour of URM when the earthquake comes and base shear at yield and ultimate base shear are considered as sensitivity parameter. Simply, the study of uncertainty in output with respect to uncertainty in input known as sensitivity analysis. There are several advantages of sensitivity analysis which are listed below:

a) To reduce the uncertainty in the model by knowing the parameters (input) that results in significant change in output.
b) By knowing the sensitive parameters one can focus on these parameters results in less computational effort and time-saving.
c) In order to know the relationship between input and output variables.
d) In the presence of uncertainty to test the reliability of the model.
e) Reduction in uncertainty, through the identification of model inputs that cause significant uncertainty in the output and should, therefore, be the focus of attention in order to increase reliability.
f) By detecting the abrupt relationship between output and input errors in the model can be predicted.
g) To simplify the model by knowing the non-sensitive parameters so that one can fix that model inputs.

In the case of calibrating models with a large number of parameters, a primary sensitivity test can facilitate the calibration stage by focusing on the sensitive parameters. Not knowing the sensitivity of parameters can result in wasting time on non-sensitive ones.

5.2 Sensitivity Analysis of URM Wall

Variation in the material properties like physical and mechanical properties affect the lateral behaviour of building. In order to find out which parameter of URM is sensitive to the earthquake response sensitivity analysis is carried out. In the present work sensitivity analysis is carried out by considering 5% and 95% probability values of input properties of masonry. By knowing the abrupt changes in output due to change in input errors in the model can be predicted. This chapter presents a sensitivity analysis carried out to obtain a reasonable range of results representing a wide number of possible situations that can be met in practice by using pushover analysis.

5.2.1 Selected URM Wall

In the present study a single two story URM wall considered for the sensitivity analysis shown in Fig.4.1, having wall thickness 0.25 m. Sensitivity analysis was carried out on the same wall considering different masonry (Clay, AAC and CLC). The compressive strength of masonry, density, shear strength, modulus of elasticity, shear modulus or Poisson's ratio of masonry are considered as random variables for sensitivity analysis which are given in Table 5.1 and Table 5.2.

Table 5.1. Details of random variables of clay masonry used in analysis

Property	Variable	Mean	COV	Distribution	Source
Density (kN/m^3)	γ	18.84	0.05	Lognormal	Park. *et al.* (2009)
Masonry compressive strength (MPa)	f_m	5	0.13	Normal	Bakshi and Kamini (2006)
Masonry shear strength (MPa)	f_{v0d}	0.18	0.2	Lognormal	Park *et al.* (2009)
Elastic modulus (MPa)	E_m	4200	0.38	Normal	Park *et al.* (2009)
Poisson's ratio	μ	0.07	0.43	Normal	Bosiljkov *et al.* (2005)

Table 5.2. Details of random variables of AAC and CLC masonry used in analysis (Bhosale, 2017)

Property	Variable	Mean		COV		Distribution
		AAC	CLC	AAC	CLC	
Density (kN/m^3)	γ	5.58	9.7	0.26	0.21	Normal
Masonry compressive strength (MPa)	f_m	2.23	2.42	0.26	0.21	Normal
Masonry shear strength (MPa)	f_{v0d}	0.22	0.23	0.28	0.29	Normal
Elastic modulus (MPa)	E_m	1610	2418	0.25	0.19	Normal
Shear modulus (MPa)	G_m	643	964	0.25	0.19	Normal

5.3 Tornado Diagram

Pushover analysis by equivalent frame modelling is used in the present study to examine the most sensitive parameter. Pushover analysis is carried out by using SAP2000 software. To examine the change in the response of structure made up of clay, CLC and AAC 5%, mean and 95% probability values of random variables are considered. The pushover curve for clay masonry wall for random variables is shown in Fig. 5.1. The sensitivity results are represented by using Tornado diagram (TD) for various variables involved. Tornado diagrams are useful for deterministic sensitivity analysis – comparing the relative importance of variables. For each uncertainty i.e. input property, we have to find out what would be the mean, low and high response. While carrying sensitivity analysis we have to keep all other parameters at mean except considered sensitive parameter i.e. we have to change only single parameter at a time and find out what is change in the response of structure considered. The sensitivity of all parameters that affect the lateral behaviour of URM wall is plotted as shown in Figs. 5.2, 5.3 and 5.4.

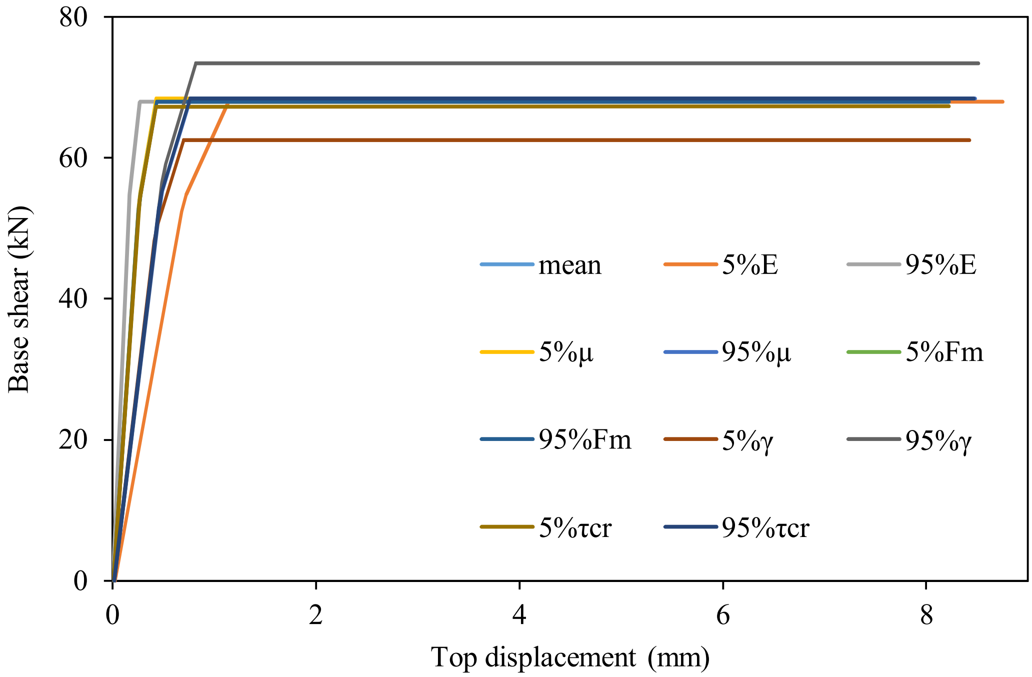


Fig. 5.1: Pushover curve for 5%, mean and 95% random variables considered for clay masonry

Base shear at yield and ultimate base shear are considered as a response for sensitivity analysis. It can be observed from TDs that f_m i.e. compressive strength of masonry does not affect the lateral behaviour of URM wall whereas f_{v0d} and γ (density) greatly influence the response of structures.

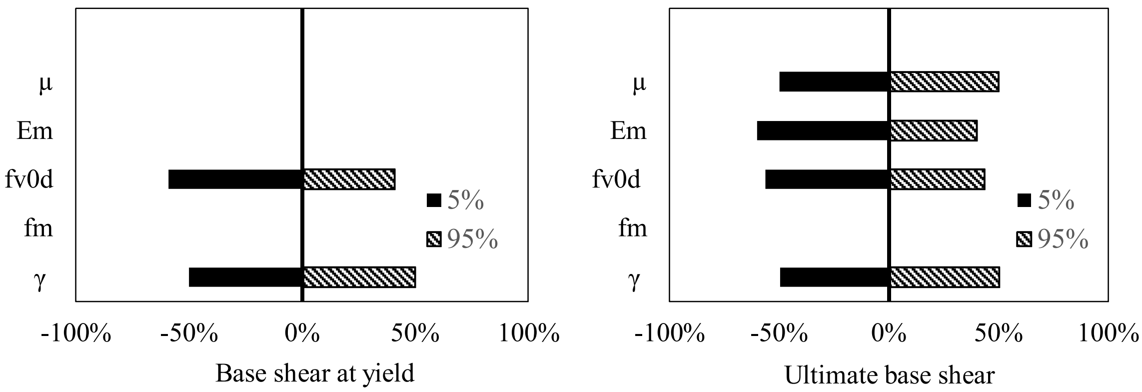


Fig. 5.2: TD for clay masonry wall

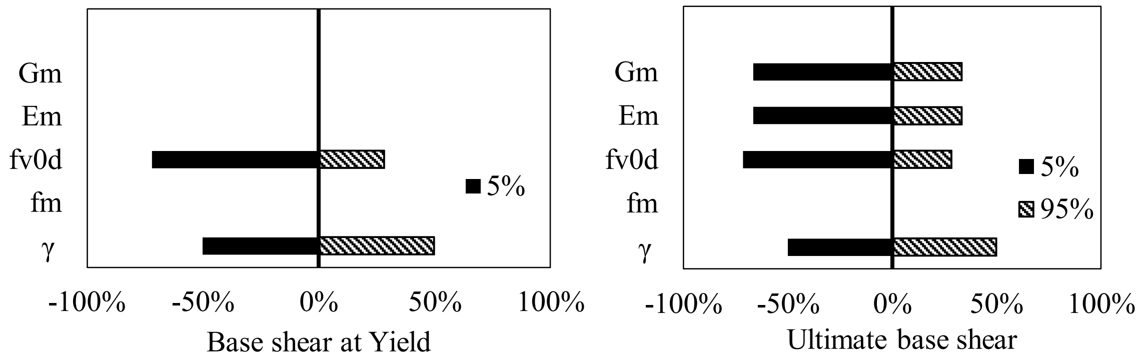


Fig. 5.3: TD for AAC masonry wall

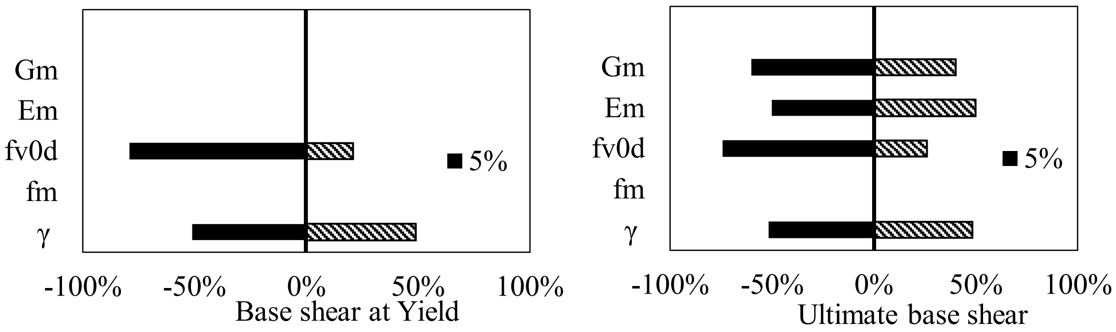


Fig. 5.4: TD for CLC masonry wall

The results show that the strength-related variation values of masonry, with the exception of compressive strength of the masonry, have shown a significant effect on the structural performance and that this effect increases with the progress of damage condition for the concrete. At yield stage lateral behaviour greatly affected by shear strength and density of masonry, whereas at ultimate stage except for compressive strength all other parameters affect the lateral behaviour.

5.4 Summary

This chapter gives the overall idea about what the sensitivity analysis means along with its advantages. Later on sensitivity analysis is carried out by considering 5% and 95% probability value of a random variable in the masonry properties. Result of sensitivity is represented in Tornado Diagrams. Results shows that base shear at yield level is sensitive to shear strength and density of masonry whereas ultimate base shear is sensitive to all properties with exception to the compressive strength of masonry.

Chapter 6

Performance Assessment Using Fragility Curve

6.1 Introduction

Fragility curve is useful to predict the possible level of damage when the earthquake comes. URM buildings are most sensitive to earthquake damages because of its high stiffness, heavy weight and low ductility. Although URM structures are common in the rural area in developing country like India. For URM catastrophic failure results in complete collapse of the structure as seen in Bhuj earthquake in 2001 in India shown in Fig. 6.1.

Most of the studies regarding performance-based seismic design are based on deterministic approach. But since lots of uncertainties are associated with material strength and earthquake loads so a probabilistic approach seems to be a more rational way for performance assessment of a structure. The HAZUS methodology has been widely used for estimating the potential losses of an existing building caused by earthquake ground shaking for the purpose of quantifying seismic risk in a region or an urban area. Often nonlinear pushover analysis of typical buildings is required for establishing building capacity and fragility curves. This chapter presents a procedure for establishing the required fragility curves for various damage states, in particular for the more severe damage states, based on nonlinear pushover analysis results.

The fragility of a structure is defined as its susceptibility to damage by the earthquake loading of a given intensity. The fragility curves can be regarded as one of the most useful tools for performance-based design of structures for the design of new buildings and also for assessing the performance of existing buildings situated in the earthquake prone area all over the world for retrofitting. Evaluation of damage state probability is very important in estimating earthquake losses. It is expressed as the probability of attaining or exceeding a certain damage state in terms of ground motion severity that may be PGA or spectral displacement. A number of approaches are available for developing the fragility curves for different types of the building considering either the empirical data from past earthquakes or using the data obtained from analytical simulations. In the present work to develop the fragility curve of two storey clay URM wall HAZUS method is used. ATC method specifies ground motion in terms of PGA whereas in HAZUS method spectral acceleration or displacement are considered as ground motion parameter. There are two components in HAZUS damage functions for ground shaking, one is based on engineering parameters like yield and ultimate strength from pushover curve known as a capacity curve and another one is fragility curves which describes the probability of damage to building for different four damage states.

Fig. 6.1: Bhuj earthquake damage to URM, 2001

6.2 The HAZUS Earthquake Loss Estimate Methodology for Buildings

The HAZUS earthquake loss estimate methodology for buildings is schematically illustrated in Fig. 6.1. It comprises a number of modules as briefly introduced and discussed in the following sections. Where Gr1, Gr2, Gr3 and Gr4 are different damage grades explained in following sections. The damage states defined by Barbat *et al*. (2006) based on yield and ultimate spectral displacements of a building are used in the present work.

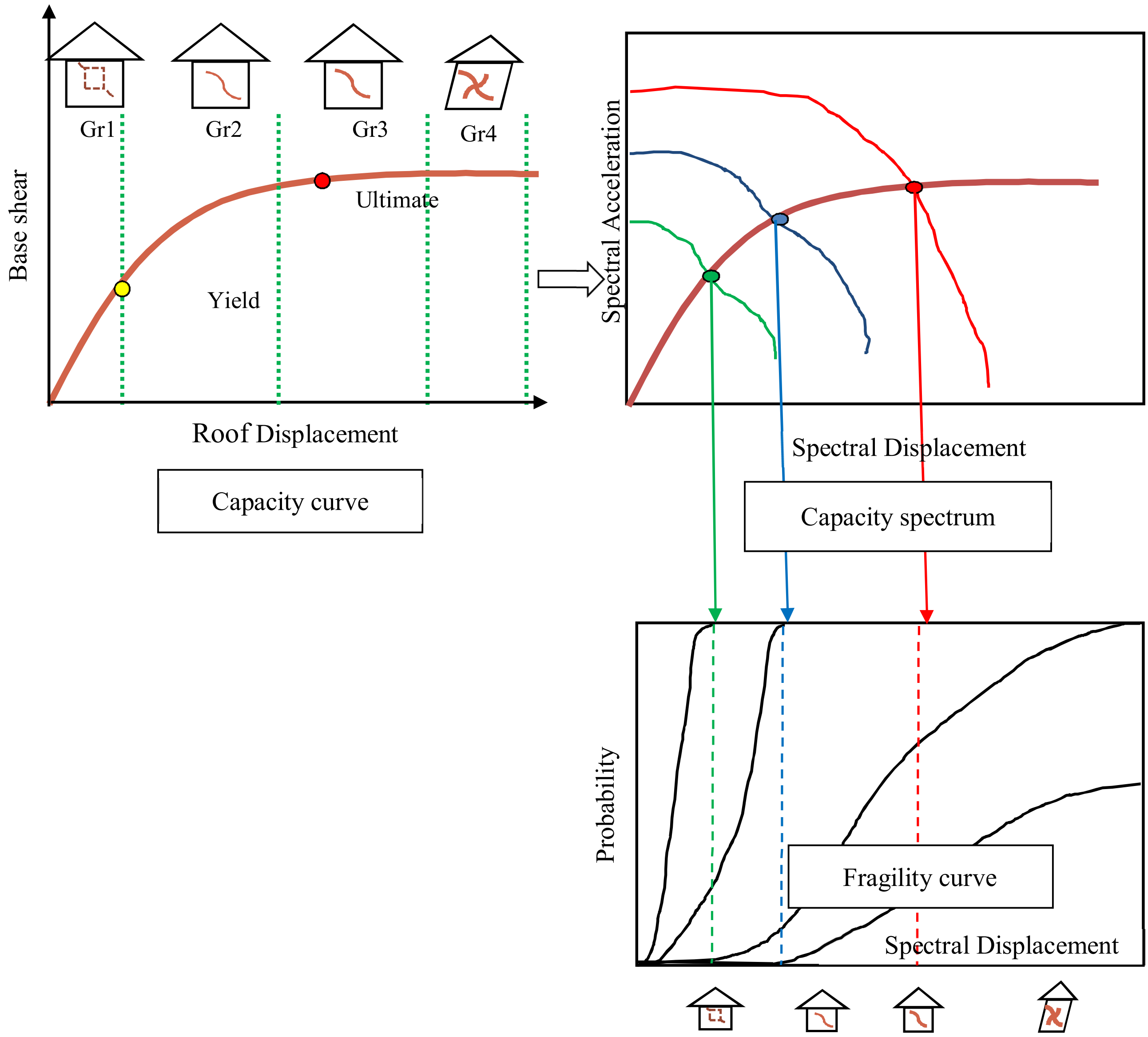


Fig.6.2: HAZUS earthquake loss estimate methodology for buildings (HAZUS 2003)

6.2.1 Development of Fragility Curves for URM

Fragility curves may be defined as the log-normal distributions representing the probability of attaining or exceeding a given structural or non-structural damage state with the median estimate of spectral response (spectral displacement in the present work) being known. This is mathematically expressed as shown in Equation 6.1.

$$p\left[\frac{d_s}{s_d}\right]=\phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{s_d}{\bar{s}_{d,ds}}\right)\right] \quad \text{.................................}(6.2)$$

where:

$\emptyset$ is known as a normal cumulative distribution function,

β_{ds} is the variability parameter obtained from standard deviation of natural logarithm of the spectral displacement for damage state *ds*,

$\bar{s}_{d,ds}$ is the median spectral displacement at which building reaches the threshold of damage state *ds*.

6.2.2 Variability Parameter (β_{ds})

$$\beta_{ds} = \left\{ \left(CONV\left[\beta_c, \beta_D, \bar{s}_{d,ds}\right]\right)^2 + \left(\beta_{M(ds)}\right)^2 \right\}^{\frac{1}{2}} \qquad (6.3)$$

where:

β_c is the lognormal standard deviation parameter showing variability in capacity properties of the building,

β_D is the variability in the demand spectrum due to spatial variability of the ground motion,

$\beta_{M(ds)}$ is the uncertainty in the estimation of the damage state threshold.

The total variability of structural damage can be taken as combining the three damage variability given in the above equation using the complex convolution process. HAZUS-MR1 has provided pre- computed values of βds to avoid complex numeric calculations. The variability values are shown in Table 6.1 for the parameters assumed in the study. However, HAZUS has defined uniform moderate variability for damage state threshold ($\beta_{M(ds)}$) as 0.4 and capacity curve variability (β_c) as 0.3. The variability due to post-yield degradation for Gr3 damage states considering minor degradation is 0.9 and for Gr4 damage states, considering major degradation is 0.5. So the total variability ($\beta_{M(ds)}$) for Gr2 damage state is 0.95, Gr3 1.05 and for Gr4 taken as 1.05.

Table 6.1: Variability consideration

Damage state	Kappa (k) factor	Damage variability ($\beta_{M(ds)}$)	Capacity curve variability (β_c)	Total variability (β_{ds})
Slight	Minor degradation(0.9)	Moderate (0.4)	Moderate (0.3)	0.8
Moderate	Major degradation(0.5)	Moderate (0.4)	Moderate (0.3)	0.95
Extensive	Extreme degradation(0.1)	Moderate (0.4)	Moderate (0.3)	1.05
Complete	Extreme degradation(0.1)	Moderate (0.4)	Moderate (0.3)	1.05

6.2.3 Damage States

Damage states give an idea of building physical conditions, which are related to various loss parameters, like an economic loss, functional loss etc. The damage states defined by Barbat *et al*. (2006) based on yield and ultimate spectral displacements of a building are used in the present work. This is shown in Table 6.2 given below.

Table 6.2: Damage state definition (Barbat *et al.* 2006)

Damage grade	Damage state	Spectral displacement
Gr1	Slight damage	$0.7S_{dy}$
Gr2	Moderate damage	S_{dy}
Gr3	Extensive damage	$S_{dy} + 0.25(S_{du} - S_{dy})$
Gr4	Complete damage	S_{du}

6.2.4 Sampling

There are many uncertainties associated with the material properties of masonry used in any construction. In the recent years, due to the absence of actually observed experimental data, many analytical methods have been adopted for generating the numerous data those are required for the development of fragility curves. These analytical techniques save time, reduces cost and controls a number of data. So to generate a number of data to incorporate these uncertainties, Latin hypercube sampling method is used in the present study. This method has the advantage that it requires a lesser number of simulations and has a smaller sample size in the analysis process. The capacity of any members or the system as a whole depends upon material strength which is inherently uncertain. This uncertainty can be modelled by using two statistical parameters like mean and standard deviation to define the central value and variability.

Mean and the covariance of the random variables considered in present study for LHS are shown in Table 5.1. The values presented for clay masonry referred from the different papers. MATLAB program is used to do the sampling.

6.2.5 Modelling and Analysis

20 different combination of properties were generated based on the mean and COV given in Table 5.1 using LHS sampling. Then 20 models were generated for the same wall and nonlinear static analysis (pushover) is carried out using SAP2000 for inverted triangular distribution. This pushover analysis method is mostly used to obtain quantitative limit state values. The critical points like yield and ultimate response and initiation of a collapse mechanism are obtained from the pushover curves (in the form of base shear versus roof displacement) using bi-linear idealization as shown in Fig. 6.3.

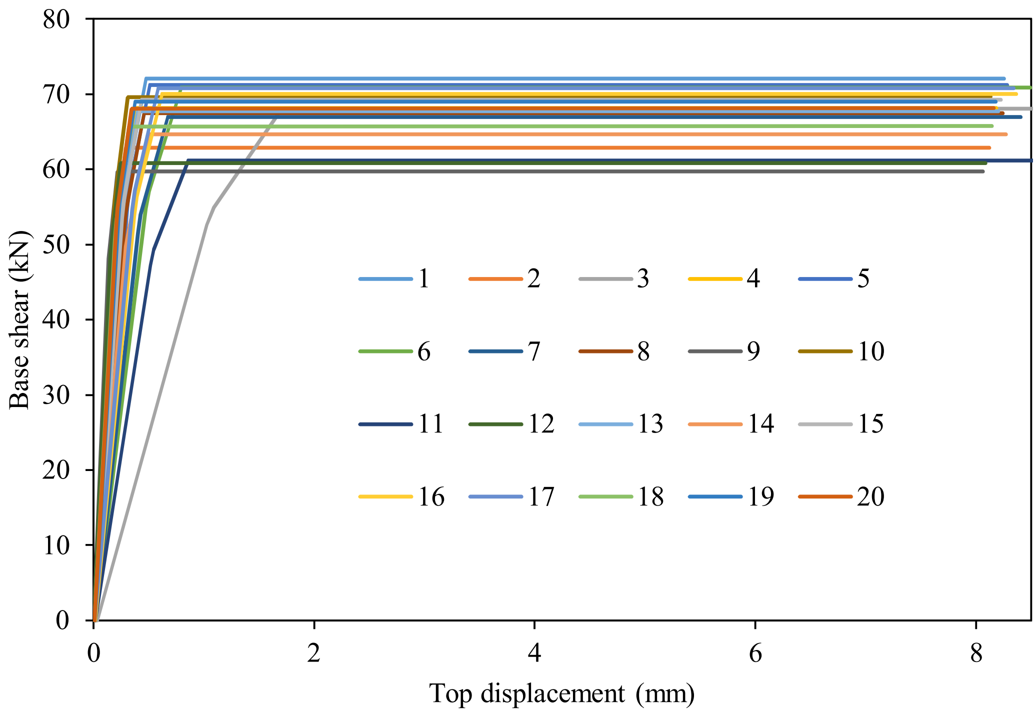


Fig. 6.3: Pushover curve for 20 different combinations of masonry properties

Using above pushover curve 20 values for yield and ultimate spectral displacement found out from the capacity spectrum. After getting this values, median yield and ultimate spectral displacement for different damage states are obtained. For the present wall median yield spectral and median ultimate spectral displacement was found to be 0.35mm and 8.23 mm respectively. Only damage state Gr2, Gr3 and Gr4 are considered in the present study for developing the fragility curves. From the spectral displacements obtained for 20 cases, median spectral displacement ($\bar{s}_{ds}$) are obtained. Median spectral response shows the threshold limit of a given damage state. Then using the normal distribution function probability of equal or exceeding a given damage state can be obtained.

6.3 Performance of URM Masonry Wall

Fragility curves for two-storey masonry wall is developed as per methods discussed above for three damage states Gr2, Gr3 and Gr4. The slope of fragility curve developed depends on the lognormal standard deviation value of β. A Smaller value of β indicates the lesser variability of damage state and hence steeper fragility curve is generated. So the Gr2 curves are stiffer than Gr3 curves (β of Gr2 = 0.95, Gr3 = 1.05 and for Gr4, it is 1.05).

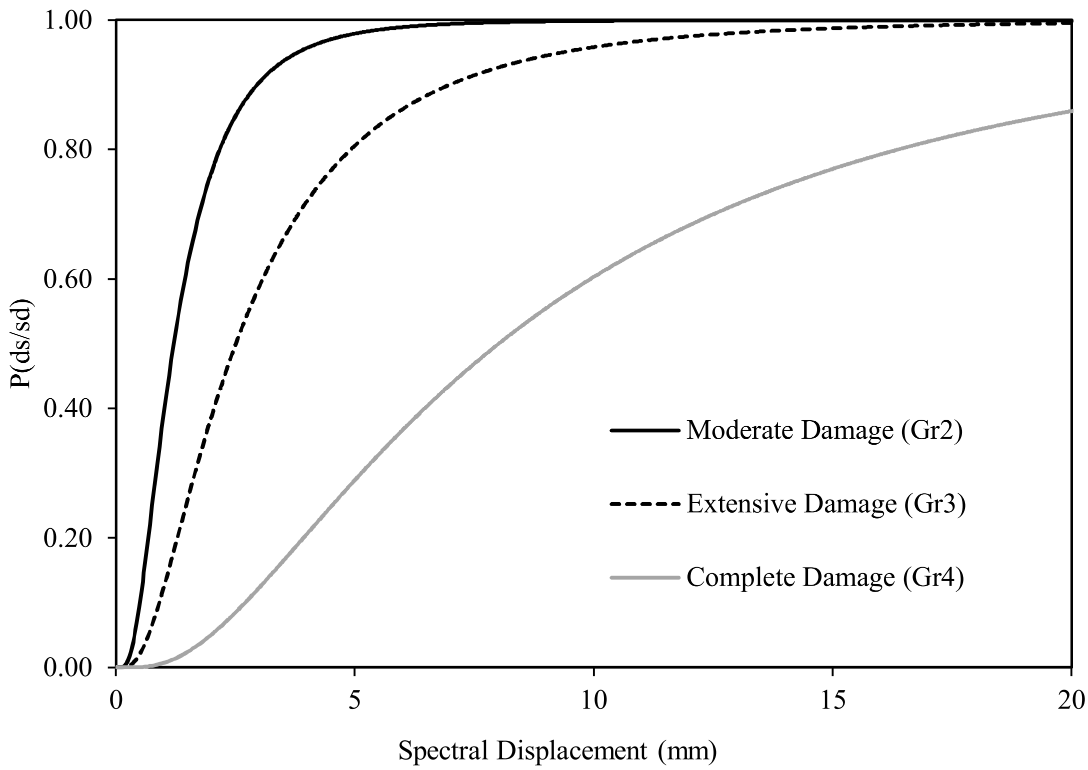


Fig. 6.4: Fragility curves for 2-storey clay masonry wall for different damage states

6.4 Summary

This chapter illustrate the step by step procedure for the development of fragility based on Hazus methodology for different damage states. Four damage state are considered in the present study defined by Barbat *et al*. (2006). Fragility curve developed for two storey clay masonry wall for 3 damage state. It is observed that the there is great probability of moderate damage compare to complete damage. Since for Gr3 and Gr4 damage state all other parameters being constant the probability of reaching or exceeding that state depends only on the median spectral displacement.

Chapter 7

Summary and Conclusions

7.1 Summary

Extensive literature review, were carried out in order to establish the objectives of the present research work. EFM method is used to understand the lateral behaviour of URM. First of all, to understand the concept of EFM and reliability of method, validation was done. In order to observe the lateral behaviour of URM, a wall with opening is selected and analysed throughout the present study. Same wall with different masonry properties were analysed for two different lateral loadings. Results of SPO analysis shows the higher strength estimation for uniform lateral load. Same wall was analysed for different cement mortar ratio. Higher grade of cement mortar results in higher strength estimation.

Considering 5%, mean and 95% of masonry properties (random variables) based on its mean and COV values, sensitivity analysis is carried out. Base shear at yield and ultimate base shear are considered as sensitivity parameter in present study. Results of sensitivity analysis are shown in Tornado Diagram for different masonry.

Seismic fragility curves are used for assessment of seismic losses for post-earthquake recovery programs as well as for pre-earthquake disaster planning. It provides the probability of structural response when subjected to earthquake load as function of ground displacement or ground motion intensity (PGA). In the present study HAZUS methodology used for the development of fragility curve. Fragility curve were developed for URM wall for three damage states. In the present study fragility curve is developed only for the clay masonry. Various conclusions obtained from present study, future scope of present study are given in this chapter.

7.2 Conclusions

Following are the major conclusions that are obtained from present study:

a) Pushover curve: Results obtained from SPO analysis it can be conclude that clay masonry will behave good as compare to Fly Ash, AAC and CLC masonry in case of earthquake. Higher grade of cement mortar will result in higher response of URM structure. Higher strength estimation is obtained for uniform lateral load distribution compare to inverted triangular distribution. Main reason for failure of URM was due to formation of shear hinges in the structure. For inverted triangular distribution story mechanism is occurring in top story whereas, story mechanism is occurring in ground story for uniform lateral load. For both the distribution, ultimate displacement is near about same.

b) Sensitivity analysis: Results obtained from sensitivity analysis shows that base shear at yield level is sensitive to shear strength and density of masonry whereas ultimate base shear is sensitive to all properties with exception to the compressive strength of masonry.

c) Fragility curve: In present study fragility curve is developed only for clay masonry wall for three damage states. It is observed that the there is great probability of moderate damage compare to complete damage. Probability of damage will decrease with increase of severity of damage.

7.3 Limitation and Future Scope of Present Study

In the present study single wall is analysed considering different masonry properties. The present work can be extended by considering different walls with different geometry, different orientations in openings. This work is limited for in-plane strength (2-D). For more accurate result the effect of out of plane strength (3-D) should be include in this modelling. Rigid wall without openings is kept out of this study. There is great variation in physical and mechanical properties of URM in different regions so in order to have more accurate results determining these properties precisely, is very important. Fragility curve is developed only for clay masonry.

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