STRENGTHENING MASONRY FOR SEISMIC ACTIONS IN DEVELOPING COUNTRIES

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α – Angle of internal friction (BS-EN)

α_k – Characteristic angle of internal friction (BS-EN)

γ – Shear strain

δ – Separation

Δv – Vertical shortening

Δh – Horizontal shortening

δ_m^{max} – Maximum separation

δ_m° – Effective separation at damage initiation

δ_m^{f} – Effective separations at failure

ε – Strain

σ – Normal Stress

σ° – Maximum nominal traction stress

σ_n – Stress in normal direction

σ_s – Stress in shear-1 direction

σ_t – Stress in shear-2 direction

μ – Coefficient of friction

μ_k – Characteristic coefficient of friction

u – Displacement amplitude

ν – Poisson's ratio

ξ – Damping ratio

ρ – Density

τ_0 – Shear strength (ASTM)

ω_n – Natural frequency

ω_g – Frequency of input wave

a – Acceleration

A_i – Area of cross section of specimen parallel to bed joint (BS-EN)

A_n – Net Area (ASTM)

a_{rms} – Root Mean Square Acceleration

A_s – Wave amplitude

COV – Coefficient of variation

D – Degradation factor

E – Modulus of Elasticity

E_i – Modulus of Elasticity of individual sample

f_c – Compressive strength

f_f – Flexural strength

f_t – Tensile strength

f_{ti} – Tensile strength of individual sample

f_{vo} – Initial shear strength of masonry (BS-EN)

f_{vok} – Characteristic shear strength of masonry (BS-EN)

f_{voi} – Shear strength of the individual sample (BS-EN)

f_{pi} – Pre-compressive stress on an individual sample (BS-EN)

$F_{i,max}$ – Maximum shear force (BS-EN)

F_{pi} – Pre-compressive force (BS-EN)

f_w – Tensile/bond strength of masonry joints (BS-EN)

f_{wi} – Tensile/bond strength of masonry joint of an individual sample (BS-EN)

f_{wk} – Characteristic tensile/bond strength of masonry joints (BS-EN)

g – Acceleration due to gravity

G – Shear Modulus

h – Height

h_{ef} – Effective height (BS-EN)

I_a – Arias Intensity

K_{nn} – Joint stiffness in normal direction

K_{ss} – Joint stiffness in shear-1 direction

K_{tt} – Joint stiffness in shear-2 direction

l – Length

l_{ef} – Effective length (BS-EN)

P – Applied Load/Force

PP – Polypropylene

PGA – Peak Ground Acceleration

R – Reaction force

R_x – Lateral Reaction force

R_y – Vertical Reaction force

t – Time

T_d – Time history duration

U – Displacement

U_x – Lateral Displacement

U_y – Vertical Displacement

U^o – Base displacement at damage initiation

U^f – Base displacement at failure

w – Width

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Abstract

The study presented aims to provide the most viable seismic retrofit solution for rural masonry. Muzffarabad is one such region where excess of unreinforced masonry structures claimed thousands of lives during 2005 earthquake. Field study was conducted in the region to familiarize with the dynamics of local construction industry before suggesting a suitable retrofit solution. Polypropylene (PP-) band retrofit has been selected as the most viable solution for retrofitting existing masonry structures in terms of cost, material availability and ease of application. To prove the efficiency of PP-band retrofit, numerical simulations and laboratory tests were conducted to assess the seismic efficiency of PP-band retrofit. Material tests were conducted in accordance with BS-EN to familiarize with the mechanical properties of locally available materials in Kashmir region and to provide material data for numerical analysis. Tests revealed lower strength and elasticity for bricks in comparison to materials found in developed countries, due to the unregulated and non-standardized manufacturing of masonry units and high water content in mortars. Shake table tests were conducted to test the effectiveness of PP-band retrofit masonry under dynamic vibrations. Results show that PP-band retrofit can enhance the post peak performance by at least 7 times in comparison to non-retrofit specimen. Real-scale structure retrofit with PP-band survived accelerations of up to 2g without any life-threatening damage, thus, proving to be an economic and efficient strengthening solution for rural communities. Following the shortcomings observed in Room-1, connection detail for PP-bands in Room-2 was revised to achieve a 100% performance enhancement. Numerical models were developed to predict cracks in masonry and analyse diagonal compression test models, in accordance with ASTM standards. The results showed 30% higher residual strength after cracking for PP-band retrofit masonry and the wall integrity was maintained for higher deformations.

Declaration

No portion of the work referred to in the dissertation has been submitted in support of an application for another degree or qualification of this or any other university or other institute of learning.

ATHER ALI

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Copyright Statement

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"The difference between being in a earthquake and being in a disaster is the level of preparation." Christopher Schmachtel [1]

Chapter 1 : Introduction

1.1 Background

Design for shelter is primarily governed by the need for protection against the forces of nature and the threats of other beings. The first ever instance of an engineered house dates back around 44,000 years ago by Neanderthals [2], which is believed to be a milestone for structural engineering. The need for an efficient and safe load transfer mechanism is important to sustain adverse structural loads (that is, external load due to a natural event like earthquake on a building) throughout the structure's intended life cycle. Structures that fail to sustain dynamically induced earthquake loads are the major cause for human casualties during earthquakes. Building a stronger structure to withstand earthquakes is not that simple especially when there are other aspects such as economy, occupancy, usage, material availability, ease of application and sustainability that needs to be taken care of simultaneously.

In order to meet the above-mentioned criteria it is important to understand the characteristics of ground motion during earthquakes and the subsequent response of the structure. Engineers and geologists continue to characterise ground vibration and the subsequent structural response to help improve seismic designs. This continued research in the field of earthquake engineering led to the proposition of seismic design codes to provide regulations and guidance for designing structures in seismic regions. Seismic design and provisions vary based on the material and techniques used for construction.

Steel and Reinforced Concrete structures due to their ability to handle stresses have better chances of surviving an earthquake as compared to masonry structures. Masonry being the most ancient construction technique used to-date due to its economy and ease of construction has major implications in rural communities where access to better materials is unaffordable. Apart from being a cheaper construction material, masonry has many inherent advantages, such as, effective sound and heat insulation, aesthetics, easy construction and fire resistance. This is why masonry is also a preferred choice in urban areas for low-rise single-family dwellings.

Masonry construction uses different types of materials such as stones, clay-brick, mud-bricks and concrete blocks. Masonry construction can be categorised in to two broad types; unreinforced and reinforced. Unreinforced masonry consists of masonry units laid using ordinary cement, lime or mud mortar without any reinforcement to handle tensile stresses in the masonry especially during earthquakes [3, 4]. On the other hand reinforced masonry is where some form of confinement or strengthening measure is provided to enable the structure to sustain tensile stresses. Masonry can efficiently resist collapse due to earthquake-induced stresses if constructed with the proper understanding of its seismic behaviour and complemented with suitable strengthening measures to resist the tensile stresses and prevent the disintegration of masonry. Most of the heritage structures are constructed of unreinforced masonry due to their historical importance [5]. These age-old buildings have proved the worth of masonry under extreme loadings, but, with the constant wear and tear of time, they need regular maintenance and suitable retrofitting [6].

Seismic strengthening techniques for masonry structures developed by researchers over the course of time [7, 8] may have high reliability in terms of earthquake performance but often fail to meet the applicability or feasibility requirements. This promotes the need to research the applicability of strengthening techniques associated with masonry in context to a specific community in terms of reliability, cost, material availability and ease of application.

The research presented focuses typically on a site in Pakistan, which has witnessed a number of devastating earthquakes in the past. Pakistan is a developing country where most of the buildings are non-engineered masonry construction especially in rural areas, which require suitable seismic retrofitting to minimize the risk of casualties during earthquake [9]. Retrofitting would not only save human lives but also save the cost for rehabilitation thus minimising the setback on economy.

1.2 Problem Statement

Masonry construction is still very common in rural and urban Pakistan. According to the data provided by “The World Bank” [10], 62% of the total country’s population live in rural areas. Due to poor transit, the availability of modern construction materials in rural areas is scarce and expensive. Locals have to resort to locally available materials for their construction needs. Structures in these areas are generally masonry construction using adobe bricks or rammed earth, stones, clay bricks, or concrete block units [9]. Due to the non-availability of engineers in rural areas, houses are built without considering any design guides or by-laws. Brick masonry is also common for single or two storey house constructions in most urban areas due to its economy, aesthetics and easy construction. Engineer involvement in the design is avoided to save cost and the subsequent performance efficiency rests on contractor's knowledge and experience. These structures fall in the category of non-engineered buildings because of the contractors' lack of knowledge about structural design especially for seismic resistance [11, 12].

Earthquake Engineering is a modern age development in the fields of geology and Civil Engineering to help understand the nature of ground vibrations and predict the subsequent behaviour of structures. This study of structural behaviour helps engineers identify the weak zones within the structure and implement suitable strengthening measures. Likewise, masonry is a very durable form of construction that requires proper strengthening to perform well under seismic actions based on the knowledge of seismic behaviour for masonry structures [13]. There are plenty of old and new masonry structures in both rural and urban areas that require performance assessment under earthquake loadings for the following reasons:

- Non-adherence to modern design codes or local construction regulations, (for example, in case of historic and rural constructions) can considerably increase the chances of structural collapse during earthquakes.

- Buildings designed and constructed in a seismic zone without incorporating earthquake resistant design and detailing principles such as, joint detailing, architectural layout, opening sizes and locations, etc. produces complex structural behaviour under seismic actions. This result in high stress concentrations in the structure rendering it more susceptible to collapse during earthquake.

- Negligence from engineers or contractors in deciphering the seismic provisions of the code can pose serious threat to life during earthquakes.

- Modification in seismic zoning is another factor that require performance assessment on the existing structures. For instance the seismic zone map of Pakistan was redefined following the 2005 Muzaffarabad earthquake, placing some of the weak seismic regions into high seismic zones.

Hence, there can be many reasons why a building or parts of it can be highly vulnerable to potential earthquake in a region. Historical evidences have shown that most of the casualties and damages during earthquakes are attributed to masonry structures due to the failure in understanding their seismic behaviour. Among the most striking examples is the 2005 earthquake in Muzaffarabad, 2010 earthquake in Haiti and the 1976 earthquake in Tangshan. The heavy death toll in these earthquakes was due to the extensive use of unreinforced masonry, which collapsed under the intensity of earthquake vibrations as elaborated further in Chapter-2.

1.3 Aim and Objectives

The research aims to *provide viable and effective seismic retrofitting solution for masonry structures in the rural communities of northern Pakistan especially Kashmir.*

The objectives for this research are:

1. To familiarize with the practices and problems faced by the construction industry of Kashmir for suggesting a viable retrofitting solution applicable in the region based on economy, material availability, ease of application and sustainability.
2. To assess the properties of construction material used in Kashmir for comparison with the findings of literature from developed regions.
3. To assess the performance enhancement of suggested retrofit solution on local masonry through shaking table test capable of simulating earthquake time history.
4. To assess the performance of retrofit masonry through numerical analysis of masonry models capable of simulating cracks.

1.4 Methodology

Initially literature research is conducted to familiarize with the seismic behaviour of masonry and the key features affecting it. A review of state of the art seismic retrofitting techniques for masonry structures is conducted to provide to the advantages and disadvantages of each with respect to economy, material availability, ease of application, sustainability and aesthetics. Seismic history of Kashmir is researched and documented along with the current problems and construction practices of the local community.

To test the knowledge of masonry behaviour under lateral loads obtained through literature, small-scale experiments of freestanding masonry shear walls were conducted. For the sake of simplicity the application of load was static and in-plane. The behaviour for shear walls with and without opening and retrofit was studied through image processing software, *Digimizer* [14].

In the next stage site study of Kashmir, high seismicity rural region selected for the research, was carried out to familiarize with the practices and problems faced by the local construction industry. Investigation proceeded by conducting interviews of people involved in the local construction industry such as building officials, contractors and labours. The study gave an insight into the changes in construction practices following the 2005 Muzaffarabad Earthquake. The knowledge acquired during field survey helped to determine the most suitable seismic retrofit solution for local masonry.

Material tests were conducted to evaluate the quality of different types of masonry used in the region. Various types of masonry units and mortar mixes were tested in combination to determine their effect on the shear and bond strength of masonry. These tests were based on the BS-EN standards for masonry testing [15-20]. Tests provided data for the numerical analysis of masonry to compare the behaviour of retrofit against non-retrofit masonry. Numerical models were verified using the test method prescribed by ASTM [21] for shear strength of masonry wallettes against the shear strength obtained from the lab tests using BS-EN.

Shaking table tests were conducted on masonry wallettes to dynamically test the efficiency of the selected retrofitting method. For this purpose, retrofit and non-retrofit wallettes were tested concurrently on shaking table to compare their behaviour and failure under increasing amplitudes of ground vibration. The results helped quantify the efficiency or the delay in collapse of a structure due to the applied retrofitting. Shaking table tests were extended to assess the performance of selected retrofit solution on a real-scale structure under seismic vibrations. Two rooms using different connections details, to apply PP-band on the structure, were tested to study the effect of connection detail in delaying the collapse of the structure.

Finally the performance of retrofit masonry against non-retrofit masonry is analysed using *Abaqus*, finite element solver [22]. The numerical model generated for the research is capable of predicting cracks in masonry. The model is verified against the small-scale tests of Chapter-3. The numerical analysis utilizes material properties determine through material tests in Chapter-5 and provides comparison between retrofit and non-retrofit masonry

1.5 Thesis Outline

The thesis consists of 'nine' chapters. Chapter-1 provides an introduction to earthquakes and masonry along with seismic related problems faced by construction industries in poor communities. This chapter points the aim of this research and lists the objectives that are to be accomplished to achieve the aim.

Chapter-2 provides a review of state of the art literature on seismic behaviour and performance of masonry. It describes the key features of masonry that affects the seismic performance of masonry buildings. It also provides a review of seismic retrofitting techniques to compare the advantages and disadvantages associated with each retrofitting method. An introduction to the geography and demography of Kashmir is also presented along with the summary of major past earthquakes and common construction practices of the region found in literature.

Chapter-3 describes the experiments carried out on small-scale wall models, constructed using MDF (Medium Density Fibreboard) bricks, to understand the behaviour of masonry under lateral in-plane loads. It also discusses the effects of retrofitting and openings on the wall specimen. The findings of the experiments are used to develop a numerical model.

Chapter-4 discusses the findings of the survey carried out in a high seismic region of Muzaffarabad (Kashmir), which was heavily devastated during the 2005 Muzaffarabad Earthquake. These findings detail the shift in the construction practices of the region after the earthquake of 2005. The survey also identifies the problems faced by the locals with regards to house construction and the role of authorities in that matter. Main objective of the study was to assess the viability and feasibility of the suggested retrofitting technique presented in this thesis.

Chapter-5 details experiments carried out to determine the material properties of masonry to provide data for numerical models. These material tests were carried out in Pakistan to assess the properties of the locally available construction material.

Chapter-6 reports the experimental work carried out on masonry wallettes under dynamic shear loading. The wallettes were tested to examine the performance of the selected strengthening technique under shear loading.

Chapter-7 discusses shake table test carried out on a prototype masonry room with the suggested strengthening technique. The aim for the tests was to assess the improvement in seismic resistance of masonry structure by employing the suggested retrofitting technique of this research. Furthermore, two different connection details, used for fixing PP-bands on the structure, were tested on two different rooms to determine their effect on the seismic performance of the retrofit structure.

Chapter-8 documents the development of numerical model for predicting crack propagation in masonry under in-plane shear loading. The accuracy of the model in predicting cracks is verified against the results of Chapter-3. Material properties from Chapter-5 are used to simulate diagonal shear tests to obtain masonry shear strength, modulus of elasticity and shear modulus for masonry and compare the results of retrofit versus non-retrofit masonry.

Chapter-9 summarizes the findings of the report to outline the contribution of this research and provide scope for future research.

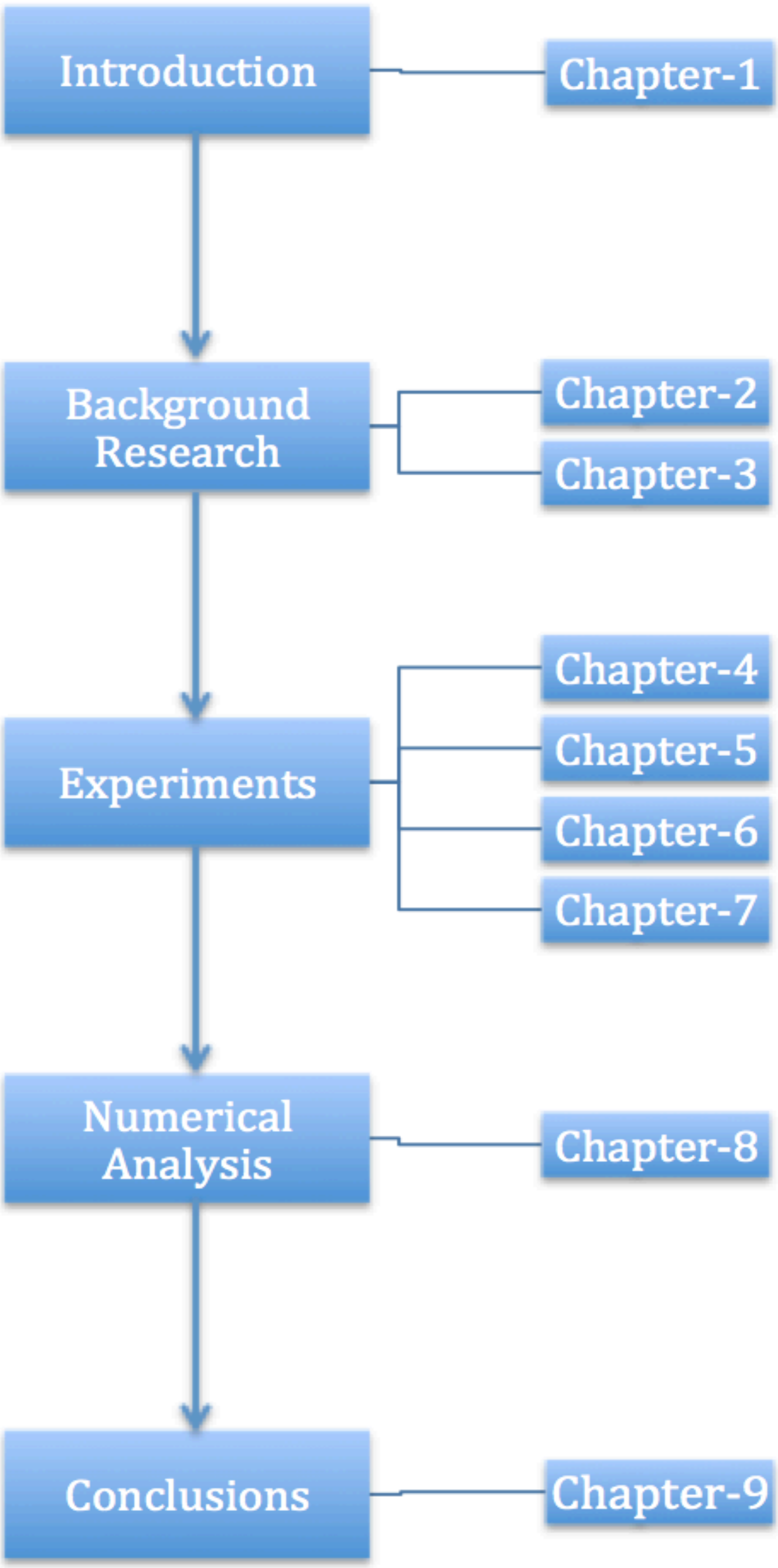


Figure 1.1: Flow chart for Report outline

Chapter 2 : Literature Review

2.1 Introduction

Masonry structures are more vulnerable to earthquakes than concrete or steel structures. This chapter provide accounts of some historic earthquakes where great damage to property and lives were caused by masonry structures. Literature research is carried out to provide an understanding of structural behaviour during seismic actions, and the key features affecting the seismic resistance of masonry structures are addressed herein. The chapter further provides the findings of the investigation carried out on state of the art retrofitting for enhancing seismic performance of masonry structures, which helps to determine the most viable retrofit solution for rural masonry constructions.

Kashmir is selected as the case study under the funds provided for this research to help development in the region. Introduction to the region and the reasons for selecting it as a case study region are documented to justify the choice. A review of state of the art retrofitting is carried out to assess the most viable retrofit solution for Kashmir region. Finally, the advantages and limitations of two different numerical modelling techniques for masonry are documented. These two numerical modelling techniques are later used in Chapter-8 for carrying out analysis on masonry to predict cracks and compare the performance of retrofit against non-retrofit masonry.

2.2 Seismic Performance of Masonry

Masonry, which is primarily used for walls, is responsible for transferring the roof load to the ground. Although masonry construction has undergone many developments over the course of time with regards to construction style and material, but it still poses great concerns when used in earthquake prone areas. Masonry performs well under gravity loads where the walls are subjected to compressive stresses only, but in case of earthquakes, lateral forces are induced in the structure that in turn is responsible for the generation of tensile stresses in the walls. As masonry is weak in tension, therefore, these tensile stresses can cause damage if not properly considered and catered for in the design of the structure.

Masonry is a very common construction style with its inherent advantages, as discussed in 'Background' section of Chapter-1, especially in rural communities and cannot simply be avoided on the basis of its poor seismic performance. Masonry type construction is commonly used for single-family dwellings and in case of rural communities the homeowners themselves supervise the construction, without any understanding of the structural behaviour especially under seismic effects. Many earthquakes in the history have shown that unreinforced masonry structures are most vulnerable to earthquakes especially the unreinforced masonry where no horizontal or vertical reinforcement/confinement is provided [23-25]. Most devastating earthquakes with highest number of death tolls are from regions where high numbers of unreinforced and non-engineered masonry structures were present.

Following are the well-documented earthquakes where the high death toll is attributed to the collapse of non-engineered masonry structures in rural communities:

a) *Tangshan Earthquake 1976*

The great Tangshan earthquake of 1976 is one of the deadliest earthquakes in modern times with a death toll of approximately 242,000. It occurred on 28th of July at 3:42 am local time with a magnitude of 7.5 and an epicentral intensity of XI [26] on Modified Mercalli Intensity Scale [27]. Most of the residential structures in the city were low-rise unreinforced brick masonry construction with no consideration for seismic design. These low-rise unreinforced masonry buildings collapsed during the earthquake due to poor connection between walls and roof, along with many reinforced concrete and masonry industrial buildings with heavy roofs [23].

Figure 2.1: Destruction in Tangshan Earthquake 1976 [28]

b) *Bam Earthquake 2003*

A fierce earthquake of magnitude 6.6 struck the city of Bam in Kerman province of Iran on 26th December 2003 at 01:56:52 UTC (Coordinated Universal Time). At least 30,000 people lost their lives and similar number faced injuries. 85% of the structures were destroyed in Bam area where the earthquake intensity was IX with a ground acceleration of 0.98g [29]. Most of the houses in the area were either non-engineered (i.e. with no consideration of design guidelines [30]) adobe structures or unreinforced brick masonry houses. Poor performance of such structures resulted in a high death toll. Reinforced masonry buildings, built with horizontal and vertical reinforced members, showed better performance and in many cases saved lives [24].

Figure 2.2: Destruction in Bam Earthquake 2003 [31]

c) *Muzaffarabad Earthquake 2005*

On Saturday, October 8th 2005 at 03:50:40 UTC (Coordinated Universal Time) an earthquake of magnitude 7.6 hits the northern regions of Pakistan claiming 86,000 lives and leaving around 70,000 injured. In India the death toll reached around 1,400 and the number of injured persons reported was 6,266. Most damage occurred in the city of Muzaffarabad and Balakot in the state of Kashmir, with an earthquake intensity of VIII [32]. 70% of Muzaffarabad city was levelled and many villages in the region were completely destroyed. This vast scale damage was due to the prevailing construction practices in the region, where most of the structures were unreinforced masonry without any consideration for seismic design and detailing [25].

Figure 2.3: Destruction in Kashmir Earthquake [33]

d) *Haiti Earthquake 2010*

A magnitude 7.0 earthquake hit Haiti on Tuesday, January 12th, 2010 at 21:53:10 UTC. The epicentre of the earthquake was near the town of Léogâne, where the earthquake intensity felt was VIII. It is estimated that around 320,000 people died and 300,000 were injured by the earthquake [34]. Haiti is considered as the poorest country in the western hemisphere with around 80% of its population living a life below the poverty line and almost 54% in abject poverty [35]. Due to such poor economic conditions prevalent in the region, the construction techniques cannot employ modern materials with proper design guides. People mostly used locally available material in their construction practices; hence most of the houses in the area were unreinforced stone or brick masonry structures and only a few had timber panels (colombage) or horizontal steel ties to provide tensile resistance [36].

Figure 2.4: Destruction in Haiti Earthquake 2010 [37]

2.3 Seismic Load Transfer in Masonry

Modern day materials like concrete and steel have well established design procedure for seismic actions. Section sizes and reinforcement details, for an anticipated level of seismicity, can be worked out with confidence using available design guides. However, in the case of masonry the design codes necessitate the use of confined or reinforced masonry for structures constructed in high seismic zones [38]. For unreinforced masonry there are certain key aspect of masonry construction (discussed in Section 2.4) that needs consideration for better seismic performance. Efficient earthquake resistant masonry design can only be realized after clear understanding of its structural behaviour under seismic vibrations and the resulting stress pattern.

Masonry is mostly used for wall construction whereas roofs and floors are constructed out of timber, concrete or steel. The entire load transfer from roof to the foundation takes place through walls, which are responsible for holding up the structure. During an earthquake the lump of mass concentrated at the floor or roof level tries to oppose the lateral drift due to inertia. With ground moving in one direction the roof mass at top generates a resisting inertial force in the opposite direction. This inertial force is directly proportional to the mass concentrated at roof level and the magnitude of ground acceleration. Between these two opposing forces wall is the only connecting member responsible for transferring roof loads to the foundation and keeping the structure intact (Figure 2.5) [39].

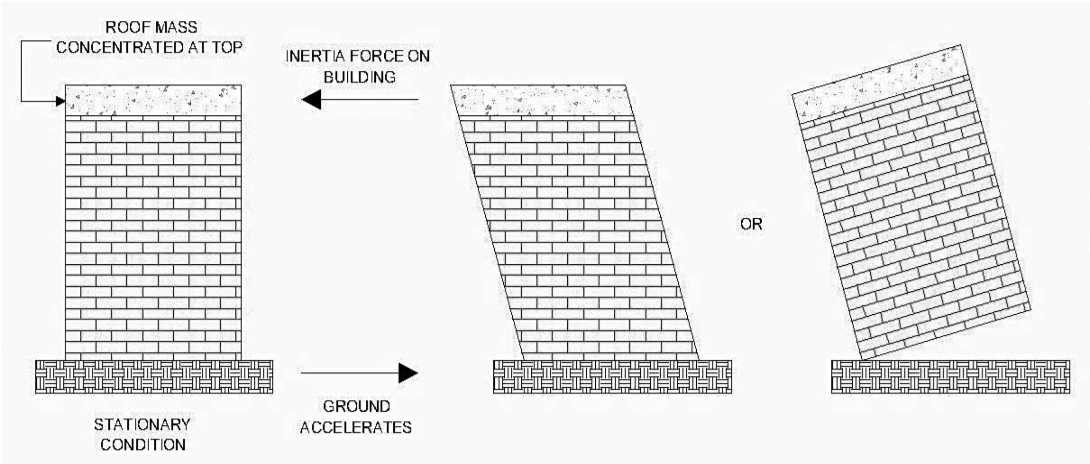


Figure 2.5: Seismic Behaviour of Structures

It is favourable to keep the mass of the structure to a minimum to reduce the inertial forces acting on the structure during earthquake-induced motion. To successfully transfer the roof loads to the foundation with sustainable damage, walls should have sufficient ductility to withstand the cyclic loading induced during an earthquake [40].

2.4 Key Features of Masonry

This topic outlines the key features of masonry construction that affects the strength and performance of masonry structures under seismic actions.

a) Joints

For a structure to behave as a box under lateral loads it is imperative to have good connection among the walls in two orthogonal directions. It not only helps in successfully transferring the load to the foundation without structural disintegration, but, also provides a support to walls that are experiencing thrust in the weaker out-of-plane direction (Figure 2.6) [41]

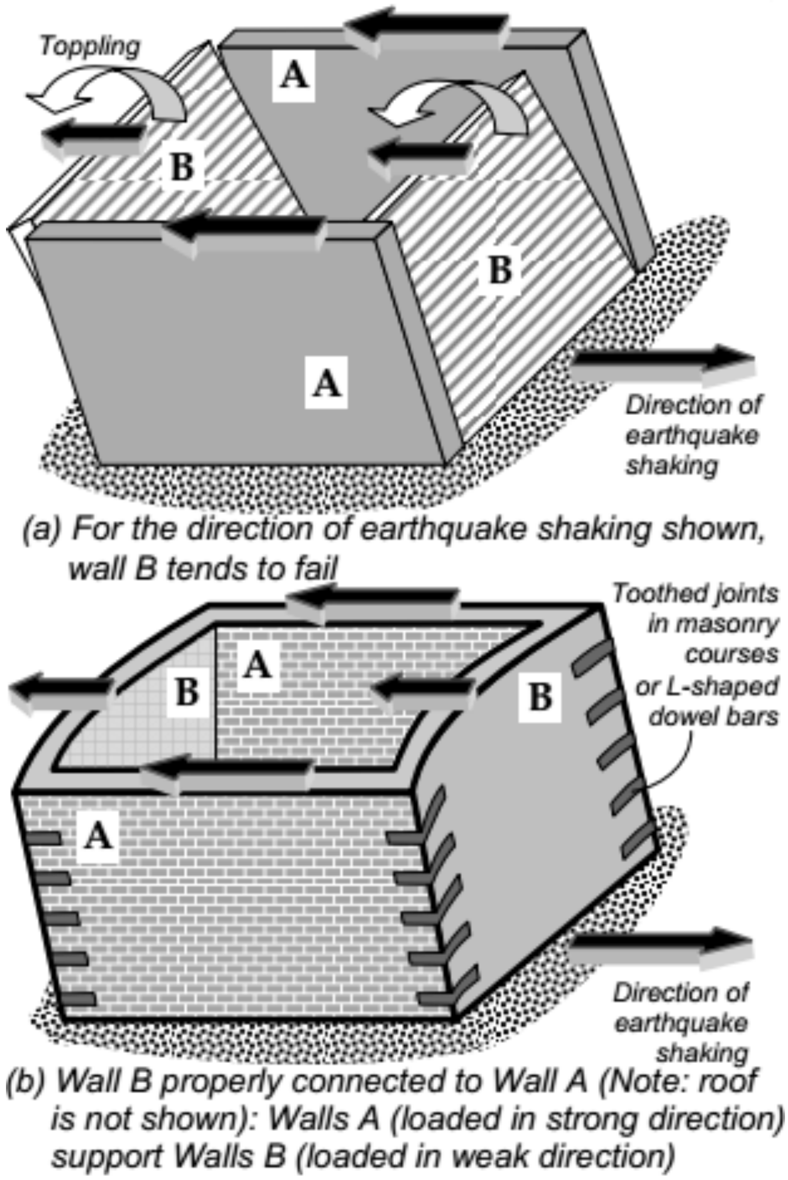


Figure 2.6: Advantages of Joints [41]

Joints for wall corners should essentially be toothed joints with end brick of the alternate layers in two walls penetrating into one another as shown in Figure 2.7. Dowels may also be provided at wall corners to further ensure the intactness of joints during an earthquake. Similarly, the joints between masonry and vertical confining elements of concrete should have the tooth joint arrangement (Figure 2.7).

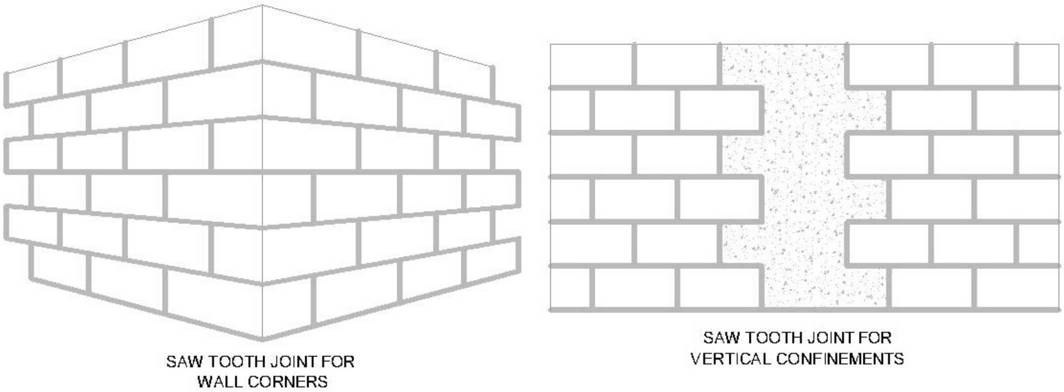


Figure 2.7: Saw Tooth Joint Arrangement

Rigid connection between roof slab and the walls also helps in maintaining the structural integrity by keeping the walls tied together at roof level [41]. This ensures a monolithic behaviour of the structure in resisting lateral loads and help transfer the loads from walls with plane perpendicular to the direction of motion to the walls having plane parallel to the direction of motion.

b) Aspect Ratio and Thickness

To increase wall resistance against out of plane forces the length to width and height to width ratio should be kept to a minimum and the limiting values are normally stated in the building design codes for the region e.g. Table 9.14 to Table 9.16 of 'Building Code of Pakistan - Seismic Provisions 2007' [38] and Table 9.2 of 'Eurocode 8: Design of Structures for Earthquake Resistance - Part 1: General rules, seismic actions and rules for buildings' [42].

Shear walls should conform to certain geometric requirements, namely *(Clause 9.5.1; BS EN 1998-1:2004) [42]*:

a) The effective thickness of shear walls (see EN 1996-1-1:2004 for determining t_{ef}), t_{ef}, may not be less than a minimum value, $t_{ef,min}$ (Table 2.1);

b) The ratio h_{ef}/t_{ef} of the effective wall height (see EN 1996-1-1:2004 for determining h_{ef}) to its effective thickness may not exceed a maximum value, $(h_{ef}/t_{ef})_{max}$ (Table 2.1); and

c) The ratio of the length of the wall, *l*, to the greater clear height, *h*, of the openings adjacent to the wall, may not be less than a minimum value, $(l/h)_{min}$ (Table 2.1).

Table 2.1: Recommended geometric requirements for shear walls – Eurocode 8 (Table 9.2) [42]

Masonry type	$t_{ef,min}$ (mm)	$(h_{ef}/t_{ef})_{max}$	$(l / h)_{min}$
Unreinforced, with natural stone units	350	9	0.5
Unreinforced, with any other type of units	240	12	0.4
Unreinforced, with any other type of units, in case of low seismicity	170	15	0.35
Confined masonry	240	15	0.3
Reinforced masonry	240	25	No restriction
Symbols used have following meaning: t_{ef} - thickness of the wall (see EN 1996-1-1:2004) h_{ef} - effective of the wall (see EN 1996-1-1:2004) h - greater clear height of the openings adjacent to the wall l - length of the wall			

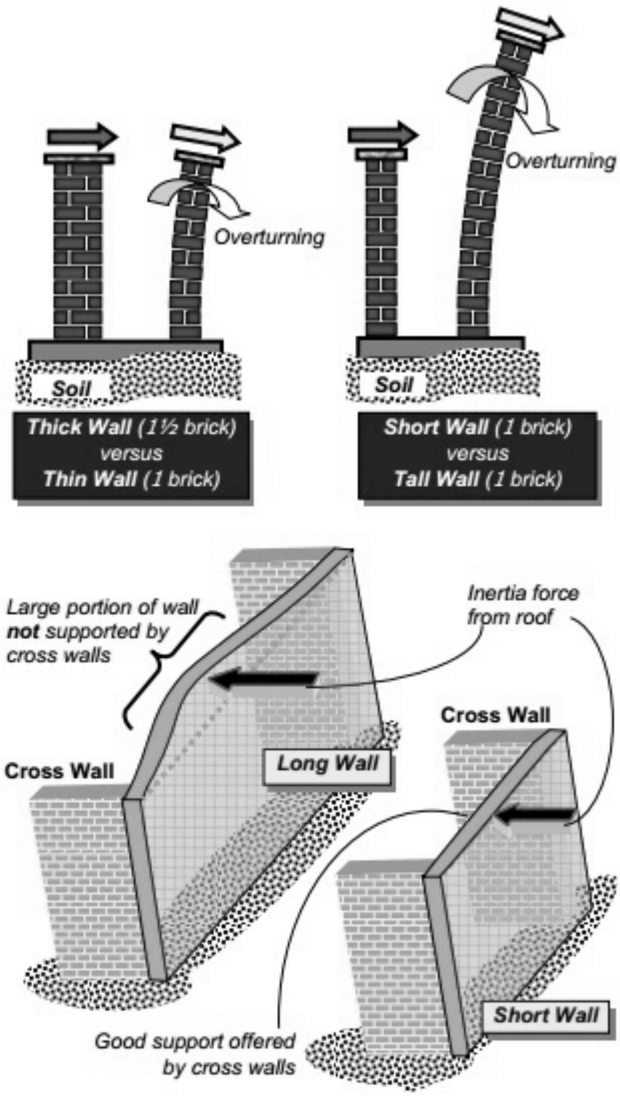


Figure 2.8: Effect of aspect ratio and thickness on stability of wall [41]

As illustrated in Figure 2.8, walls having their length or height dimension too large as compared to wall thickness are more prone to failure in their weaker (out-of-plane) direction (limiting values given in Table 2.1. To reduce the chances of failure due to overturning, horizontal bands should be provided at intermediate levels along the entire length of the building. These horizontal bands can either be reinforced cement concrete or wooden beams, or embedded steel bars in mortar layer. Similarly, to reduce the length of the wall vertical bands of RCC, wood, steel bars (in hollow masonry units) or a supporting wall in perpendicular direction should be provided. Error! Reference source not found. outlines the recommended geometric requirement for shear walls according to Eurocode-8 [42].

c) Openings

Openings in a shear wall form weak zones with stress concentrations at its corners. Greater the size of the opening weaker would be the wall; therefore, the size of the opening should be restricted and kept to a minimum as prescribed by the seismic provisions. No opening should be provided at the corners of the walls and a minimum distance requirement from the centreline of buttressing wall or pier should be provided according to design code specifications [43]. It is recommended to keep the height of all the openings at the same level so that continuous horizontal reinforcing band should be provided at the lintel level of all the walls to make the structure behave monolithically. If the size of the opening exceeds the limiting values of the code then reinforcing concrete members should be provided to strengthen the opening.

Arches are perform satisfactorily under compression but fail to sustain the tensile stresses developing due to the lateral actions of seismic vibrations and thus should be avoided or strengthened. For instance, the use of arches over the openings is identified as a source of weakness in the "Building Code of Pakistan – Seismic Provisions 2007 (Clause 9.9.1.10)" and shall be avoided, otherwise, steel or reinforced concrete ties should be provided [44].

d) Architectural Form

Complex architectural forms such as projections or re-entrant corners produce special modes of oscillation in the structure and subsequently result in high stress concentrations in the structure [40]. Keeping the geometry simple would also help in providing a simple load transfer path for seismically induced forces and would consequently be easier to analyse and strengthen in comparison to a structure with complex architectural features.

Structures build in seismic zones should have a simple rectangular plan rather than L or T shaped plan or any other complex plan [45]. To avoid such irregular plans it is ideal to divide the layout into smaller rectangular plans as illustrated in Figure 2.9.

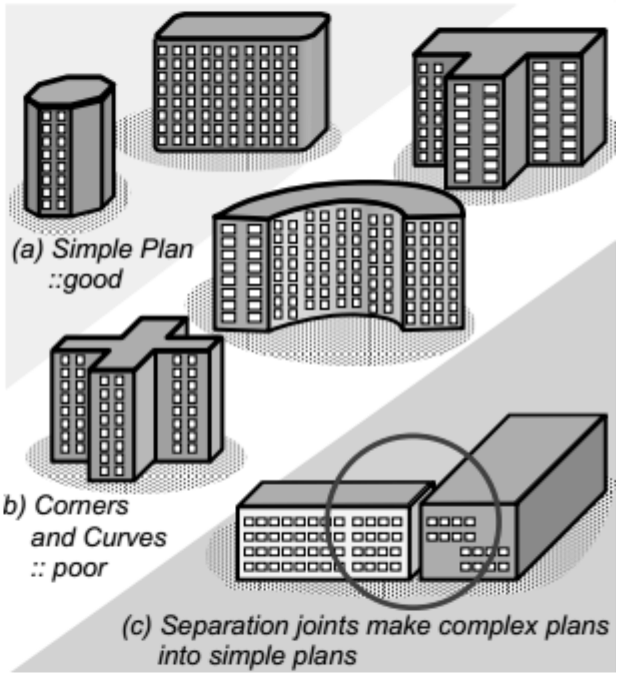


Figure 2.9: Building with simple plan perform well during earthquakes [45]

Providing separation in buildings to simplify the layout and to avoid re-entrant corners can result in pounding between the adjacent buildings during earthquake. To avoid this, considerable distance should be allowed between the buildings and their height and mass should be kept fairly similar to one another to produce similar natural frequencies and mode shapes during ground motion. This would result in two buildings moving in harmony with one another, thus avoiding collisions [45].

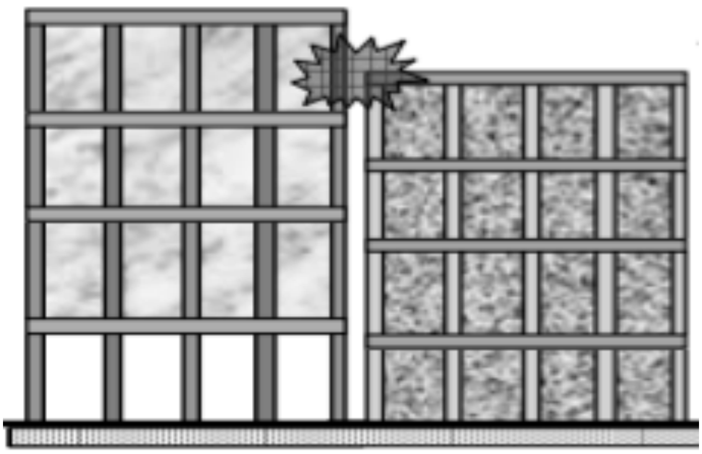


Figure 2.10: Buildings constructed close to one another may collide during earthquake [45]

Furthermore, buildings constructed in earthquake prone areas should be well proportioned in terms of height and length. To avoid torsion and complex stress concentrations, building layout should not be too long or too tall as compared to its other two dimensions [45]. A building moving entirely in one direction poses lesser threat as compared to differential movements, which produces torsion in the walls. Torsion in buildings usually occurs when the centre of mass of the building does not coincide with the centre of rigidity of the building. This happens due to the fact that inertial forces induced in the structure due to ground vibrations act at its centre of mass (CM) and the resisting force which is in an opposite direction acts at the centre of rigidity (CR) and if not coincident will produce torsion as shown in Figure 2.11 [46]. To avoid this complexity it is better to distribute the structural member throughout the structure uniformly along with the non-structural elements [47]. This would result in keeping the centre of mass and the centre of rigidity close to one another and would subsequently reduce torsion. Eurocode-8 in *Clause 9.7* specifies area, sizes, distribution and spacing of shear walls throughout a simple masonry structure in two orthogonal directions to help achieve lateral resistance by proper distribution of rigid elements [42].

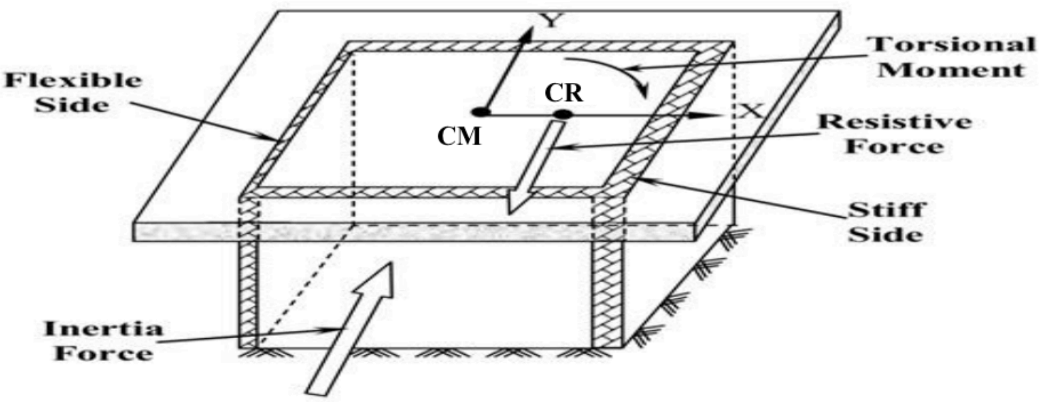


Figure 2.11: Effect of Centre of mass and rigidity on building behaviour [46]

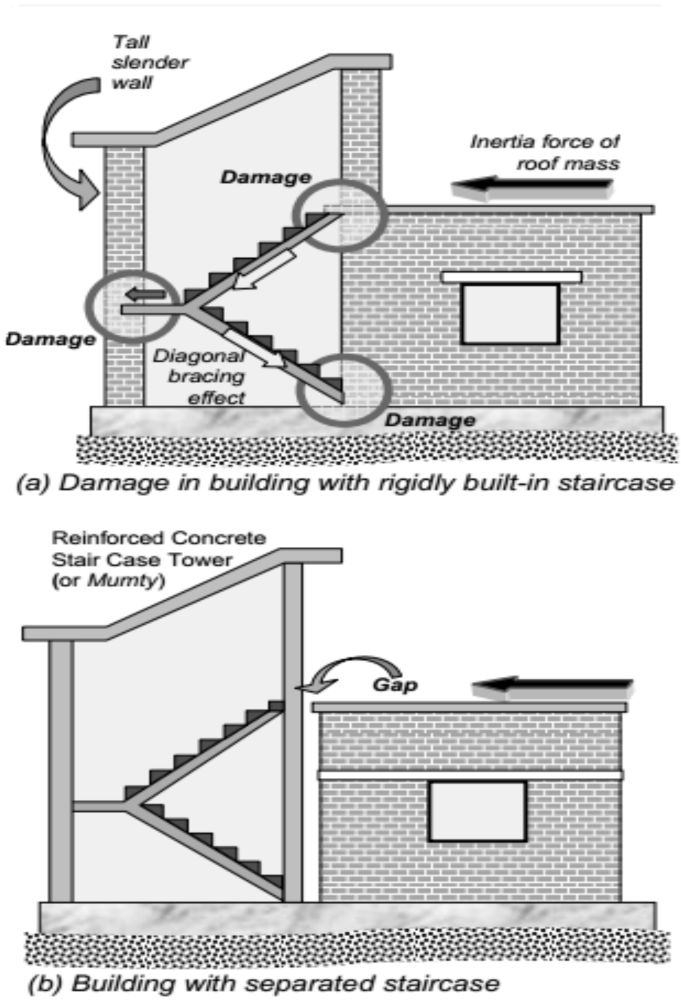


Figure 2.12: Seismic Detailing of staircase in masonry building [43].

Similarly, staircases should be constructed with a gap from the main building and the gap should be sufficient to avoid any collision between the structures (Figure 2.12). This separation is made because stair towers are usually a projection in the plan and would induce complexities in the structure. The extra rigidity of the stair tower would tend to shift the centre of rigidity away from the centre of mass and thus would complicate the design [43].

2.5 Failure Modes for Walls under Seismic Loads

Earthquake induces lateral forces are more concentrated at roof or floor levels due to the concentration of mass and their subsequent inertias, as explained in Figure 2.5. In case of masonry constructions, walls are the main structural elements supporting the structure and responsible for the transfer of load from roof/floor level to the foundation, therefore, it is imperative to understand the failure modes associated with masonry walls. There are two types of failure modes for masonry walls during earthquakes [48]:

- In-plane stresses (due to forces parallel to wall)
- Out-of-plane stresses (due to forces perpendicular to wall)

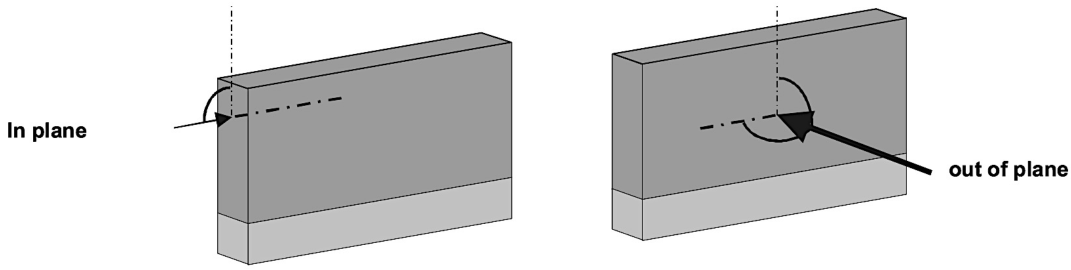


Figure 2.13: In-plane and out-of-plane directions for walls [49]

Walls are mostly designed to resist lateral loads in their in-plane direction where walls stiffness is greater in comparison to out-of-plane direction. In unreinforced masonry shear walls are the main members responsible of providing lateral strength to resist horizontal earthquake forces [50]. This also research focuses on studying the crack behaviour of masonry for in-plane stresses. Failure modes for masonry with lateral forces acting along the plane of the wall are as follows:

a) <u>Shear Crack</u>

Shear cracks appear as a result of the opposite lateral forces acting at the top and bottom of the wall. This is the most common mode of failure and it follows the path of load transfer from roof to the foundation i.e. a diagonal crack appears through the wall (Figure 2.14). Usually occurs in wall loaded with considerable vertical as well as horizontal loads and with an aspect ratio of (1:1) to (2:1) in case of bigger vertical loads [51].

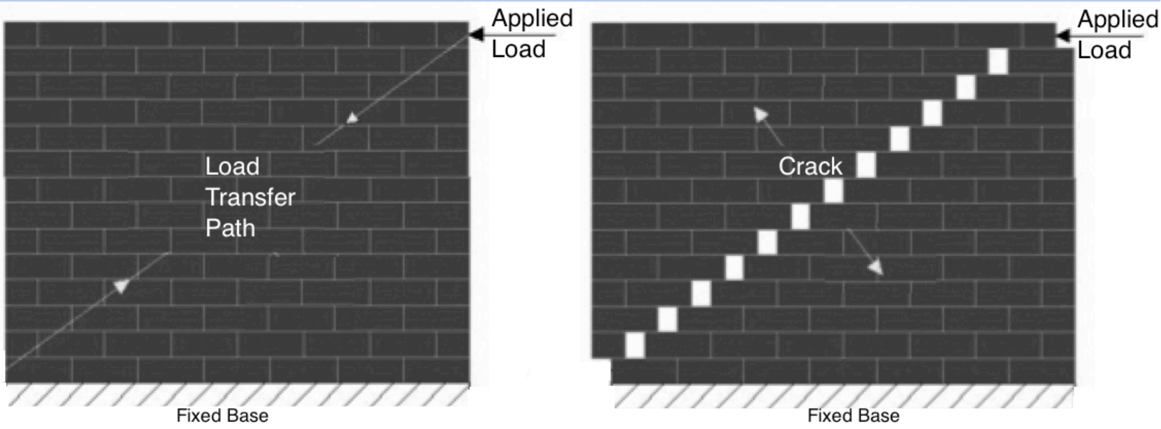


Figure 2.14: Diagonal shear crack in wall follow load transfer path

b) <u>Shear Slip</u>

In shear slip failure mode the wall cracks appear horizontally in between the brick layers as shown in Figure 2.15. Cracks rather than opening along the diagonal load path finds a weaker plane along the brick layer and creates a horizontal shear slip. Walls that are predominantly loaded with horizontal forces can exhibit this type of failure and the aspect ratio for such walls is usually (1:1) [51].

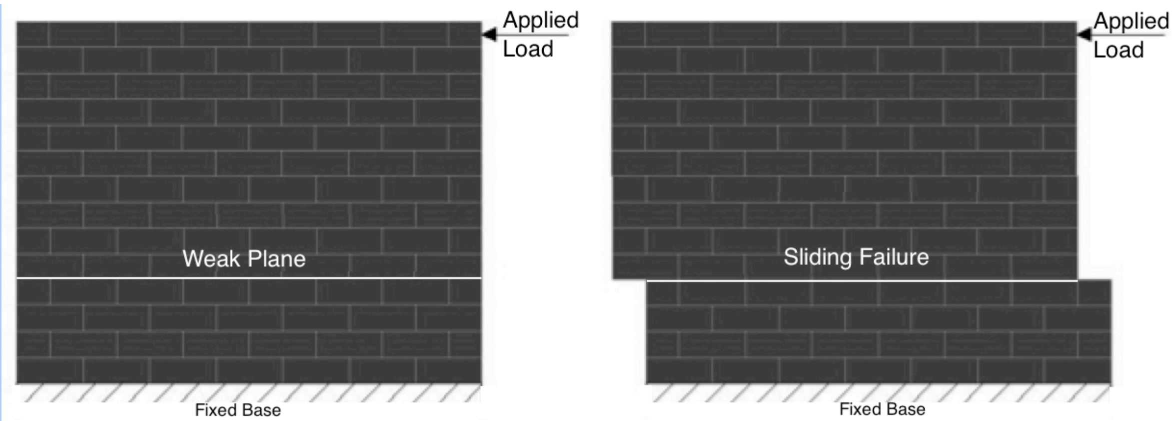


Figure 2.15: Shear Slip due to a weak plane between masonry layers

c) <u>Bending</u>

Bending failure mode occurs when the wall is sufficiently tall as compared to its width i.e. with aspect ratio of (2:1) or higher. Due to lesser length dimension of the wall in comparison to the height, the wall is strong against shear failure or slip but eventually fails due to flexure [51]. Cracks in bending failure open on one side of the wall where tensile forces are generated with crushing on the other side of the wall where compressive forces govern (Figure 2.16).

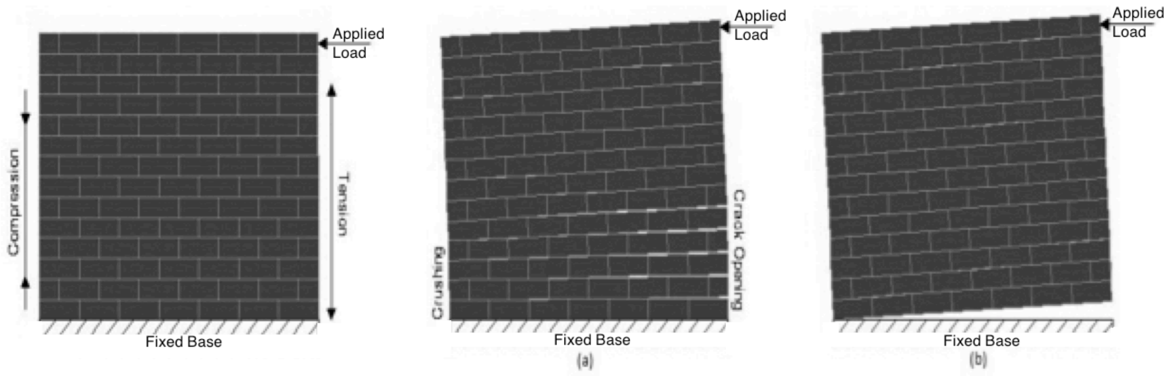


Figure 2.16: Bending failure in shear walls: (a) flexure crack opening, (b) overturning.

2.6 Review of Seismic Retrofitting Techniques

Rural communities in developing countries have extensive use of masonry structures that require retrofitting to enable them to sustain earthquake actions with no life threatening damage. Researchers have developed number of seismic retrofitting techniques for masonry structures. This section provides a review of state of the art on masonry seismic retrofitting to list their advantages and disadvantages with respect to a rural community. The assessment would help to pick the best viable solution for the region under study i.e. Kashmir.

i. Shotcrete

Application

Concrete or sometimes mortar is sprayed on to the wall under high velocity using pressure pump and hose. Before shotcreting steel reinforcement mesh is applied to the wall and the entire process should be carried out in the presence of experienced worker [8].

Advantages and Disadvantages

Increases the lateral load resisting capacity of wall by a factor of 3.6 and also improves the ductility of wall. As this method employs high tech machinery, modern construction materials and skilled labour, therefore, it cannot be applied in rural communities. Also concrete used in smaller quantities is not quite sustainable. Excessive dust and noise is produced during the application which could be very unpleasing for the neighbours [8].

ii. Stitching & Grout/Epoxy Injection

Application

Stitching and grout/epoxy injection is used to hold the cracks appearing in a wall. Firstly, horizontal slots are made into masonry across the crack line 25-30 mm deep and 10 mm wide. Slots are then cleaned with water and grout is placed into the groove using grout applicator gun. Stitching steel bars are pushed into the groove and covered on top with Cement based grout. These bars should extend approximately 500 mm beyond the crack line, on both ends. If the crack is at the corner then the bar is bent around the corner to satisfy the required length [52, 53].

Advantages and Disadvantages

It is a non-disruptive method that does not cause extensive damage to the existing wall and gets the structure back to its original outlook. Not quite expensive as the products used are readily available and have easy application especially in small quantities. This technique restores the initial stiffness of the wall and minimises the likelihood of further cracking [8].

iii. Re-Pointing

Application

Re-pointing is used where the bricks are in perfectly good shape and only the mortar has been deteriorated. In such case the old mortar is cut out carefully using thin diamond blade without causing any damage to the brick units. The mortar still stuck inside the joint is removed using jointing chisel. The joints are then cleaned using vacuum and low-pressure water to get rid of dust and old mortar. The new stronger mortar is then laid in the joints using pointing trowel (Figure 2.17).

Advantages and Disadvantages

This method is cost effective, as it requires products that are readily available. Application is relatively easy and require little technical knowledge, however, some form of temporary works may be required to support the structure [8]. The original aesthetics of the building is preserved or even more enhanced.

Figure 2.17: Re-pointing on brick masonry structure [54]

iv. Seismic Wallpaper

Application

Seismic wallpapers are basically fibre-reinforced polymers (FRPs), which typically contain carbon or glass fibres for providing strength and stiffness. These fibres are held together and protected by a matrix, commonly made of polyester, nylon or epoxy [55]. FRPs may be applied like wallpaper on the entire wall or it may be used in the form of strips applied at certain locations. These strips can be placed horizontally, vertically or diagonally.

Advantages and Disadvantages

FRPs can be applied easily without causing damage to the existing structure and, hence quite suitable for repair of cracked walls. FRP-repaired wall offers the same deformation integrity and resistance as that of undamaged FRP-strengthened wall [56].

FRP improves ductility and shear capacity of masonry. In case of repair, the shear capacity of retrofitted wall is more than that of the original wall. However, FRP composite drawbacks in recovering the initial stiffness of the wall [57].

Manufacture of glass-reinforced fibre uses a lot of energy and if burned in fire it produces harmful fumes. Application requires skilled labour with easy application and difficult removal [8].

v. Post Tensioning

Application

Post tensioning of masonry uses the same basic principal as that of Pre-stressed concrete where additional compressive forces are induced to counteract the tensile forces that might later generate. Post tensioning is generally carried out using tendons made of either high strength threaded steel bars or steel mono-strands [58]. The arrangement (Figure 2.18) shows that it requires a strong concrete foundation in which the anchorage plate can be embedded. Also, a concrete cap at the top of the wall is provided for supporting the spreader plate and stressing the tendon.

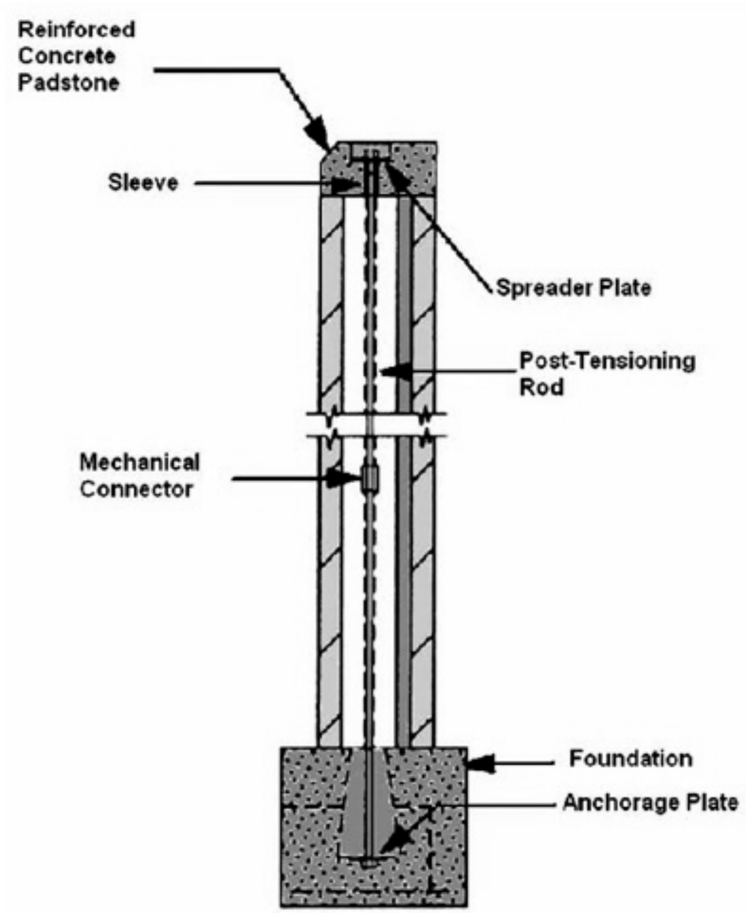


Figure 2.18: Post tensioning of masonry using steel tendons [58]

To avoid the corrosion problem of the reinforcing tendons, fibre reinforced plastic (FRP) tendons have been used in place of steel tendons. Post tensioning method in case of FRP tendons is same as that of steel tendons except that FRP tendons are placed inside a steel tube and grout is injected to ensure that tendons are fully restrained [7].

Another unconventional post tensioning method proposed by Turer et al. (2009) uses rubber tyres [59]. In this case the post tensioning material is not embedded in the wall but applied on the exterior as depicted in Figure 2.19. In this method the sides of the tyre are cut off and only the middle portion is used. A circular chain of tyres is formed by passing steel pipes through the tyres and connecting with the pipe of the adjacent tyre by two steel bolts on each end of the pipes as shown in Figure 2.19(b) [59]. Once circular chain of connecting tyres is wrapped around the wall the bolts are tightened to apply the post tensioning stress. After installation there might be stress losses due to stretching of rubber, therefore the bolts are tightened again after a few days. To prevent the damage of wall corners half cylindrical wooden logs are used at wall corners and edges as shown in Figure 2.19(c, d and e) [59].

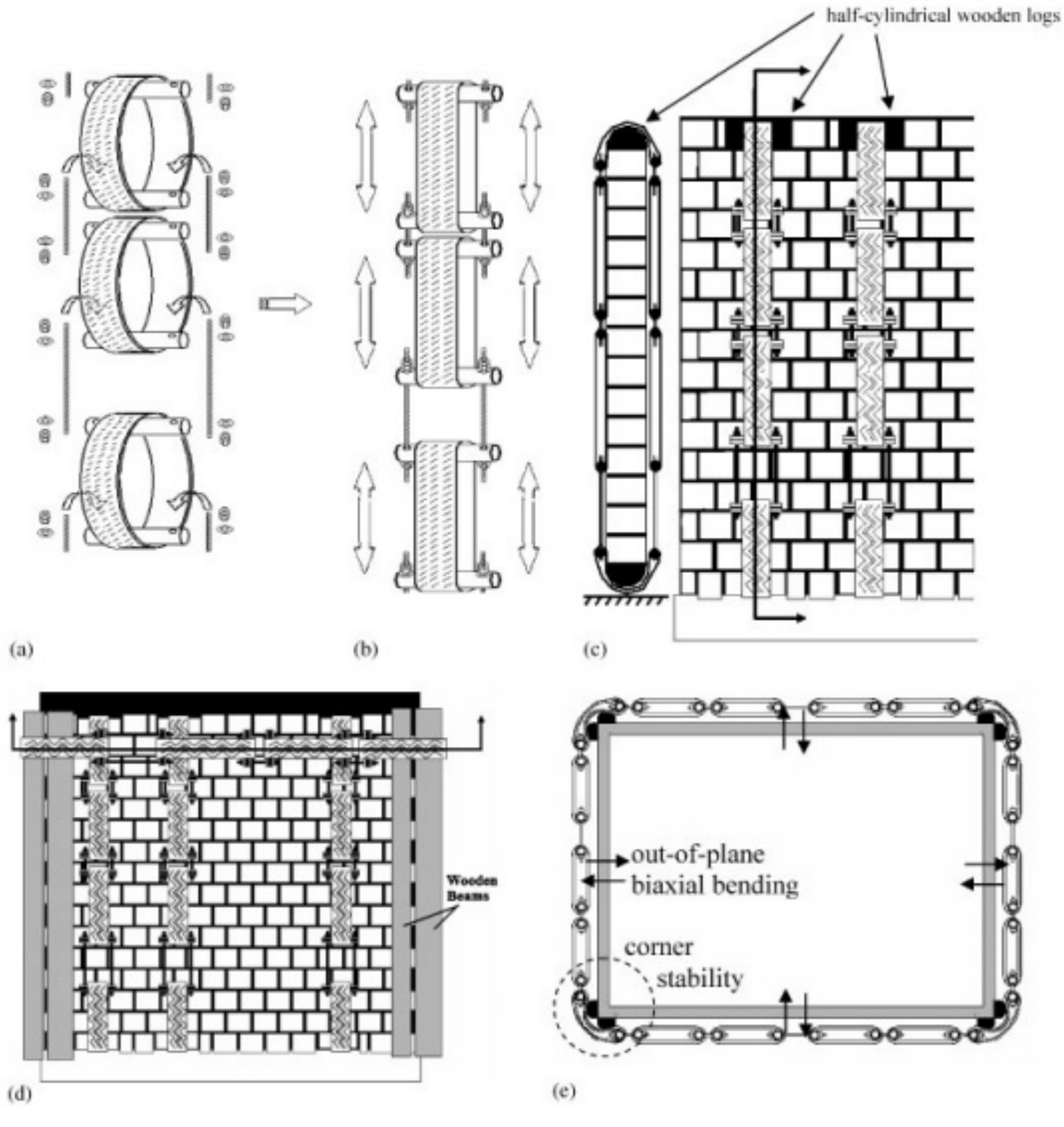


Figure 2.19: Post tensioning of masonry using rubber tyres [59]

<u>*Advantages and Disadvantages*</u>

The conventional post tensioning, using mono-strands, greatly improves the seismic performance of masonry with increased shear resistance and out-of-plane resistance of wall. However, the method is expensive and complicated to be employed in rural areas because it requires highly skilled labour and high tech machinery especially in case of existing buildings.

The post tensioning method using rubber tyres is highly sustainable and cheap as compared to the ones using tendons. The seismic resistance of the masonry models having vertical and horizontal reinforcements increases by 100% [8]. Major problem with this method is the compromise on aesthetics, as the rubber tyre reinforcement is too protruding and cannot be concealed with plaster. Rubber tyres are highly flammable material and will produce toxic fumes on burning.

vi. Polymer Mesh Reinforcement

Application

This method uses two types of material, industrial geo-grid and soft-mesh. The later one is weaker and cheaper [8]. This mesh can either be applied on the entire wall or critical sections of the wall where high stress concentrations are expected such as around openings or re-entrant corners. After the application of mesh to the wall surface, the mesh is covered with plaster finish.

Figure 2.20: Application of Reinforcing mesh [60]

Advantages and Disadvantages

The method improves the seismic behaviour of wall by preventing masonry disintegration under continued earthquake vibrations. The method offers a gain in the strength of wall and deformation capacity. It has easy application and can be applied to existing structures without causing any damage. However, the material used in polymer mesh reinforcement is not easily available in rural areas and uneconomical especially in case of industrial geo-grid, which has a higher application cost as well. The material is not sustainable unless obtained from some recycled or reused units [8].

vii. Centre Core

Application

In this method a reinforced grouted core is placed in the centre of existing masonry walls. A continuous vertical hole is drilled from the top of the wall to the basement of the wall. Size of the hole depends on the degree of strengthening aimed in the process. The method uses oil-drilling technique and can be drilled through a 2 or 3-storeyed masonry wall. The dust and debris is vacuumed out and reinforcement is placed in the hole, finally, the grout material is pumped into the hole under pressure. The grout is composed of a binder material (e.g. epoxy, cement, and polyester) and a filler material (e.g. sand) [7].

Advantages and Disadvantages

Centre core technique improves the in-plane and out-of-plane load resistance of the wall. There is no compromise on the aesthetics as the core is hidden and no compromise on the occupancy or architecture of the building.

The method requires high tech machinery for drilling, cleaning the hole and pouring of grout, thus making it uneconomical and unfeasible for rural communities. The reinforced centre core creates zone of varying stiffness and strength properties within the structure [7].

viii. PP-band Mesh Reinforcement

Application

This method uses polypropylene (PP) packaging strip intertwined to form a mesh, which is attached to the wall by drilling holes through the wall and passing steel wire to tie the mesh. A parametric study on the size of mesh and the spacing of connectors revealed that coarser mesh adequately connected may perform as well as finer mesh partially anchored [61]. Therefore, decreasing the size of mesh is not helpful if the mesh is not properly attached to the wall.

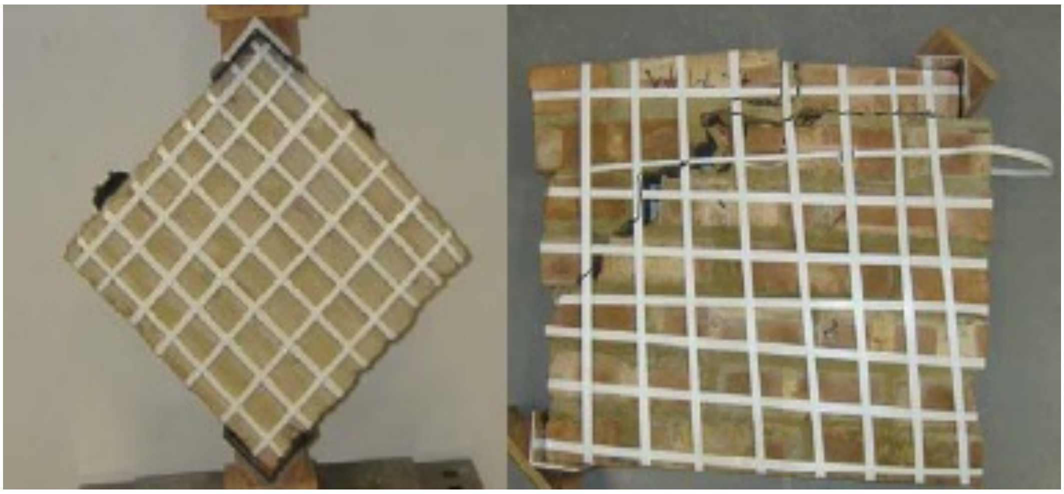


Figure 2.21: PP-Band Mesh Reinforced Specimen Before and After Test [62].

Past Research

This method was proposed by Paola Mayaorca and Kimiro Meguro of University of Tokyo in a paper submitted to JSCE Journal of Earthquake Engineering on 10th October, 2003 [63], and later published a conference paper on PP-band retrofit in 2004 at 13th World Conference on Earthquake Engineering [61]. Tests mentioned in these articles conclude that walls reinforced with PP-band mesh had:

- No considerable change in crack pattern;
- Higher stiffness and slower speed of crack propagation;
- No effect on wall peak strength which implies that mesh influence occurs only after cracking;
- Higher post-peak strength (approx. 60%), whereas for unreinforced it dropped to 10 - 40%;
- 25% residual strength after initial cracking, in one of the walls, which was due to the poor mortar overlay and broken steel wires which means that mesh was not properly connected to wall.

For out-of-plane bending tests, the retrofitted wallettes achieved strengths twice greater and deformations 60 times larger than the non-retrofitted wallettes [64]. A study on the orientation of mesh carried out by Macabuag et al. in 2008 showed that horizontal bands prevent the separation of bricks within the row, whereas vertical bands increase the frictional resistance between brick layers and contributes in resisting sliding [62]. Shaking table test on miniature PP-band retrofit masonry models with and without slab retrofit revealed 16 and 4 times higher seismic capacity, respectively in comparison to non-retrofit masonry in terms of Arias Intensity [65]. For ¼ scale two-storey stone masonry structure produced at least 4 times better seismic performance for retrofit model in terms of arias intensity [66].

Shaking table tests on miniature models showed [67]:

- Non-retrofitted walls showed sudden brittle failure and were unable to maintain further load.
- The complete mesh effectively prevents loss of material and maintains wall integrity for large deformations.
- PP-band retrofitting enhanced the safety of single-storey stone masonry buildings even in worst earthquake scenarios.

The method was proposed for masonry structures in Nepal and Pakistan (particularly, Kashmir), and local masons were trained to use PP-band mesh for strengthening their masonry structures. The training course concluded with a low-tech shake-table demonstration on small-scale masonry structure to demonstrate the effects of PP-band retrofit. A full scale retrofitting of a real-scale masonry structure was also carried out with the help of local masons to give them experience of PP-band retrofit [68, 69].

Advantages and Disadvantages

This method is claimed to have an easy application without the need of skilled labour with prior knowledge of seismic construction. Polypropylene packaging material is cheap, readily available and sustainable if the scrap PP-bands from packaging is re-used as retrofitting material. Study shows PP-band retrofit carried out by engineer or by the home owner himself have the same collapse potential thus avoiding the need for expert's supervision [70].

As the PP-band mesh is not protruding therefore it can easily be concealed with plaster. Plaster is necessary to protect PP-bands from fire and from the deteriorating effects of UV radiation from sun [67, 71]. Plastering can however affect the true aesthetics of masonry construction and add cost to the retrofitting if plaster was not intended initially.

PP-band mesh application costs only 5-15% of the total construction cost [68, 72] and can be applied to an existing structure causing minimal damage to it. It improves the in-plane and out-of-plane resistance against static and dynamic loading.

The method although being claimed as a cheap and readily available material with easy application is struggling to make way into the local construction practices. The main reason for its very limited application till date is due to sophistication in terms of band orientation, anchorage and fixing of wire mesh into the wall [25]. These issues with PP-band application need to be addressed to provide more confidence and knowledge for people who wish to strengthening their structures.

ix. Reinforced concrete confinement

Application

The method uses vertical and horizontal RCC bands to seismically retrofit the existing and new masonry structures. Horizontal reinforced concrete bands are provided at intermediate levels (i.e. plinth and sill levels) and at lintel level, while the vertical bands such as tie columns are provided around window and wall edges for bringing a monolithic behaviour to the structure [38, 73]. This arrangement of reinforced concrete (RC) tie columns and RC tie beams to confine masonry is called *Confined Masonry Construction.* Tie beams and columns are responsible for holding masonry together and prevent disintegration of the walls under seismic actions. Masonry walls are the major load resisting members for gravity and lateral loads. Confining elements resist a small proportion of gravity loads, and their main objective is to hold together masonry walls during earthquake vibrations and prevent the subsequent collapse of structure [73].

Code Specifications

Specifications for confined masonry as given in the Building Code of Pakistan (section 9.9.4) [44]:

- Horizontal Confining elements should be provided:
  - Over all walls at floors/roof level, if the floors/roof is not reinforced concrete.
  - At lintel level over all walls.
  - Over Gable walls.
  - At every floor level and in any case, vertical spacing between the confining elements should not be more than 3 m (10 ft).
- Vertical confining elements shall be placed:
  - At the free edges of each structural wall element;
  - At both sides of any wall opening with an area of more than:
    a) 1.40 m^2 (15 ft^2) for Seismic Zones 3 and 4,
    b) 1.86 m^2 (20 ft^2) for Seismic Zone 2, and
    c) 2.32 m^2 (25 ft^2) for Seismic Zone 1.
  - Within the wall if necessary in order not to exceed a spacing of 4.5 m (15 ft) between the confining elements;
  - At the intersections of structural walls, wherever the confining elements imposed by the above provisions are at a distance larger than 1.5 m (5 ft) for Seismic Zones 2, 3 and 4 and 2 m (7 ft) for Seismic Zone 1.
- Concrete for confining elements should be poured after masonry walls have been built with proper toothing in place, to obtain perfect bonding between masonry and confining elements.
- Minimum cross-sectional dimension for confining elements limited to 150 mm (6 in).

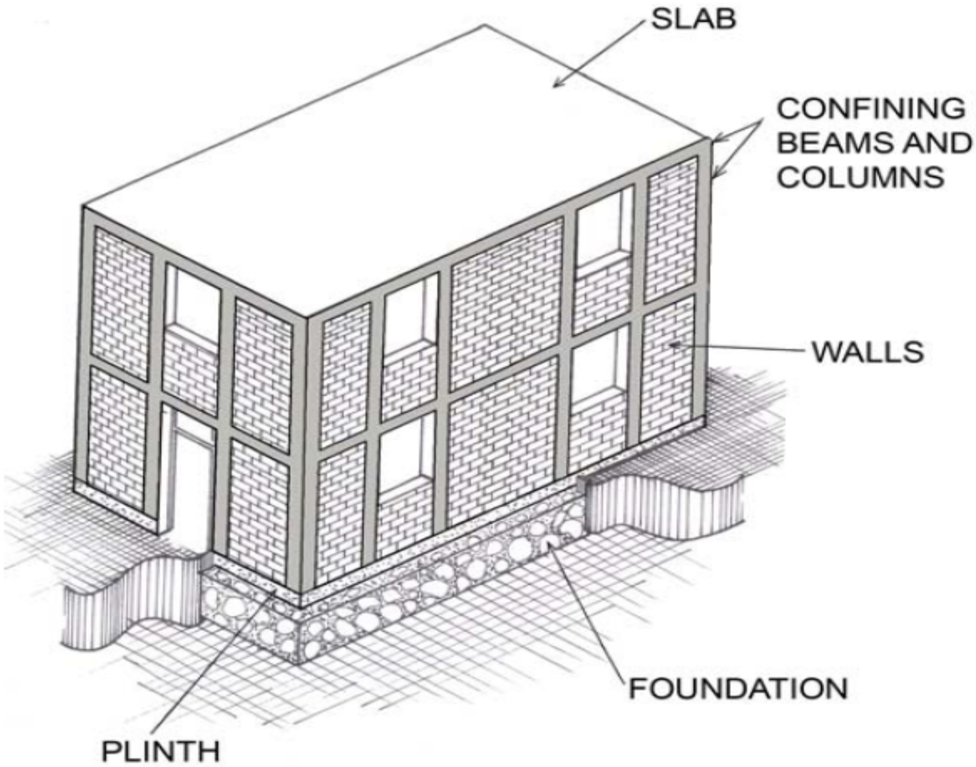


Figure 2.22: Confined Masonry Construction [73]

Advantages and Disadvantages

Past earthquakes demonstrated that a properly constructed confined masonry structure develops lesser damage as compared to unconfined masonry structures. Confined masonry has improved seismic behaviour with increased ductility and shear strength. Confining members prevents disintegration of masonry under cyclic loading and the structure behaves monolithically [73].

Confinement method is only cost effective if used for new constructions and with regards to rural construction this method is quite expensive, as it requires skilled labour for steel fixing and concreting. In case of existing structures this method would be uneconomical and unsustainable, as it requires heavy demolition and reconstruction of wall sections. Labour required for confined masonry needs to have a higher level of skill than those required in unreinforced masonry.

x. Plastic Carrier Bag Mesh

Application

Tetley and Madabhushi first tested the method in 2007 and used ordinary plastic carrier bags cut into strips of 20 mm and the plaited together to make ropes. These plaited ropes were knotted together in to mesh of sizes 50 mm x 50 mm. The mesh was then wrapped around the wall and fixed at the ends using tacks to mimic pegging. Plaster could be applied to the wall to hide the bags and protect them from deterioration [74, 75].

Figure 2.23: Plastic Carrier bag retrofit for masonry [75]

Advantages and Disadvantages

Addition of carrier bags increased the collapse time for the masonry wall and showed ductile failure. This means sufficient advance warning of collapse to minimize causalities [75]. The material is light and flexible to use, costs very cheap and reusing plastic bags for retrofit is considered a sustainable solution [74]. The method is easy to apply and does not require any special work force or supervision.

Disadvantage for this method involves the time taken to construct the mesh, which serves as a barrier for this method unless the mesh manufacturing can be industrialized/streamlined in some way [74].

xi. Bamboo Mesh

Application

The method uses bamboo reinforcement as vertical and horizontal strips tied together to form a mesh. The mesh is applied on outside and inside of the house and tied together by PP-strings passed through the holes drilled in the wall [75].

Figure 2.24: Bamboo mesh retrofit for masonry [75]

Advantages and Disadvantages

Shake table tests carried out by Meguro et al. show that adobes masonry structures retrofitted with bamboo mesh could withstand twice larger input energy in terms of the arias intensity in comparison to the non-retrofit structure [76].

The method is simple and straightforward to undertake and requires no special work force. Bamboo is a cheap material and universally available, thus making it a feasible alternative for seismic strengthening in developing countries. However precaution is needed against insect attack [77].

2.7 Why Study Kashmir?

The region of Kashmir in Pakistan was chosen as the site for this study due to its frequent seismic activity and prevalent construction practices that are typical in many geographically similar areas around the world. It is a region situated in Himalayan ranges and has poor accessibility network thus setting it back in terms of economy and technology. Structures in the region are mostly masonry constructions built using locally available materials. The region was badly affected during the 2005 earthquake and thousands of lives were lost due to poor construction practices and material quality [9].

a) Geography and Demography

Kashmir is located in the northwest of Indian subcontinent in the great Himalayan ranges and comprises of Jammu and Kashmir region in India, states of Gilgit-Baltistan and Azad Kashmir in Pakistan, and Chinese region of Aksai-Chin. The region has breath-taking beauty with majestic mountains of Great Himalayas and Pir Panjal. The vast extents of glacial peaks in Himalayan Ranges are responsible for the formation of many perennial rivers that irrigate the regions of Pakistan, India and China before terminating in the ocean (Figure 2.25). The famous rivers of Indus, Jhelum, Chenab, Ravi, Beas, Sutlej, Ganges, Yamuna, Brahmaputra and Spiti all originate from Kashmir region which makes it a disputed territory between the adjoining countries, all trying to control its bountiful rivers. This conflict has affected the region's overall development in terms of economy and infrastructure.

Figure 2.25: Map of Rivers originating from Himalayan Ranges [78]

Figure 2.26: Map of Kashmir Region [79].

Pakistan controlled Kashmir region comes under the province of Khyber Pukhtunkhwa, which is the second most densely populated province of Pakistan with population density per square kilometre of 238 according to the 1998 census conducted by the Pakistan Bureau of Statistics [80]. The population of Kashmir region under Pakistan's control have a population density per square kilometre of 320 which is more than the average population density of Pakistan which is 232 [81]. This concentration of population in cities and valleys constituted to the high death toll during 2005 Muzaffarabad earthquake. Indian controlled Kashmir region however has a population density of 56 people per square kilometre as per the census conducted by the in 2011 [82].

b) *Seismic History*

Around 225 million years ago India was a large island located 6400 km south of Asian continent. The Indian plate started moving north around 200 million years ago towards Asian Plate [83]. As a result of this tectonic movement over millions of years the two plates came in contact with another and were locked in a giant struggle for supremacy. Under immense pressure the crust began to buckle and move upward rising to altitudes of 8000 m above sea level. This marks the formation of Himalayan ranges, which stretches for 2.5 km and have some of the tallest peaks of the world. This tectonic movement still carries on today but at a very slower rate, where Indian plate moves into Eurasian plate at 4 mm per year. As a result of this relative movement the pressure in the joints keeps building up over decades when finally this pressure is released in the form of an earthquake and resulting in the formation of high-rise mountains in the great Himalayan range. The ongoing energy storage due to subduction of Indian plate gives a high probability of future earthquakes in the region with magnitude greater than 8 [84].

There is no descriptive seismic history available and no repetition of earthquakes in a region has been recognized for the purpose of estimating recurrence interval. Although the history of the subcontinent extends beyond 1500 B.C. but there is no properly maintained and descriptive record available for earthquakes except for the last 500 years and nearly completed records are available only for the last 200 years [85]. Although some account of past seismic activities in the Kashmir region can be found in various ancient scriptures and texts [85] [86].

Table 2.2 lists some of the major earthquakes in and around Kashmir region up to the well-known devastating 2005 earthquake; note that the intensities given in the table are not quite accurate but only as interpreted through the devastation mentioned in ancient texts. Data for this table are derived from the works of Iynegar et al. (1999) [86], Vigne (1844) [87] , Bilal (2011) [88] and earthquake records available from U.S. Geological Survey (USGS) website.

Earliest recorded earthquake that we know today is of 1250 BC. The tradition mentions that the earthquake caused an entire town of Sindmat Nagar to reduce to rubbles. A crack appeared in the middle of the town form which water gushed out and formed present day Wular lake [86].

Table 2.2: History of Earthquakes in Kashmir Region

No.	Year	Intensity	Location
1	1250 B.C.	12	Wular Lake, Srinagar
2	883 A.D.	5	Srinagar
3	1123 A.D.	4	Srinagar
4	1501 A.D.	7	Srinagar
5	1555 A.D.	12	Srinagar, Hasanpur
6	1560-61 A.D.	3-4	Husainpur, Maru-Petgam
7	1569-77 A.D.	3-4	Srinagar
8	1669 A.D.	4-5	Srinagar
9	1678-79 A.D.	7	Srinagar
10	1736 A.D.	8	Srinagar and Neighbourhood
11	1779 A.D.	7	Srinagar and Neighbourhood
12	1784-85 A.D.	8	Srinagar
13	1828 A.D.	7-8	Kashmir
14	1885 A.D.	10	Baramulla
15	1974 A.D.	8	Pattan
16	2002 A.D.	4	Srinagar
17	2005 A.D.	8	Muzaffarabad

Although 1555 earthquake is widely mentioned in the ancient texts but the exact location of this earthquake or its epicentre is still not known. According to historians the major earthquake was accompanied by severe aftershocks spread over a course of a week or so, and caused a total mayhem in the Kashmir valley. Even the well-built and strong houses could not survive the earthquake. Holes and fissures started appearing in the ground and it was difficult for the travellers to keep their path. Many landmarks such as trees and houses shifted their positions during earthquake. According to one tradition narrated by many historians two villages Hasanpur and Husainpur, located on opposite sides of river Bihat (present day Jhelum), swapped positions with one another [86].

The 1885 earthquake occurred at 5:00 am of 30th May and had an intensity of 10 on Richter scale. Effects of earthquake were spread over an area of 1000 km^2 but the focus of earthquake was at Baramulla. Death toll from the earthquake estimated between 3000 to 3500 and around 400 houses were destroyed [88].

The most devastating earthquake in Kashmir region for which descriptive records are available is the 2005 Muzaffarabad Earthquake. The earthquake claimed around 90000 lives and many villages were completely destroyed rendering almost 2,000,000 people homeless. Due to mountainous terrain and poor accessibility relief work was difficult and slow. Cold weather conditions and spread of epidemic in the area further claimed many lives. Aftershocks were felt for many days following the main earthquake. An estimate of total reconstruction cost give figures in excess of 5 billion dollars [84].

Apart from the above-mentioned earthquakes, the region is still seismically active with shocks of 4-6 magnitude on Richter scale being recorded on monthly basis [89].

Figure 2.27: Aerial view of devastation caused by 2005 Muzaffarabad earthquake [25]

c) <u>Construction Practices</u>

Kashmir being a mountainous region with high peaks has major accessibility concerns especially during bad weather conditions such as heavy rainfall and snow.

Rail network could never be established in the region due to the mountainous terrain, therefore road network is the only option but that too is quite dangerous and poorly maintained. As a result the region could not develop in terms of economy and technology. The scarcity of modern construction material makes them unaffordable for local masses; therefore these people have no other option than to use the locally available and cheaper materials for construction [9]. One such material that is commonly used in the region is masonry. Most of the construction in the region prior to 2005 earthquake was *Unreinforced Brick Masonry* in city areas especially for single or double storey housing units and *Stone Masonry* in rural areas. Masonry construction had poor performance during 2005 earthquake and the consequences of unreinforced masonry construction contributed significantly to the death toll of around 90,000.

Owing to the mountainous terrain flat terraces are congested and acquiring land there is expensive. Houses are built mostly on sloping grounds, which require cut and fill to level the ground for construction. In most cases the soil is not properly compacted and in the event of an earthquake the soil gets destabilized and causes settlement. Houses are built against the natural slopes of hills which exert increased pressure in the event of an earthquake due to the loosening of soil [25].

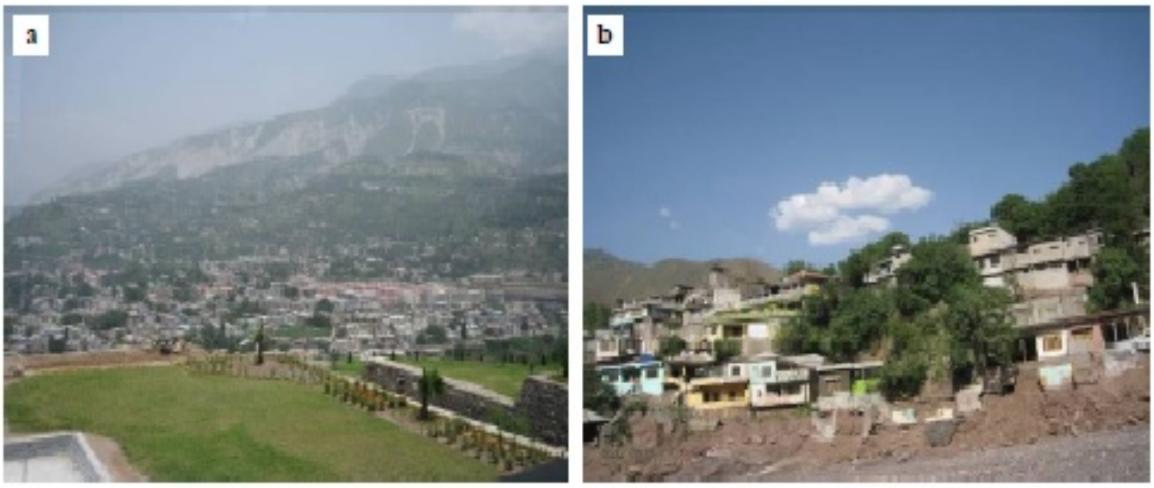


Figure 2.28: (a) Majority of population living along hill slopes, (b) Typical example of poor site selection for construction along hillside [25]

According to the housing report by Qaiser Ali in 2006 [11], houses in the region, after the 2005 earthquake, are still being built without any design and there is no involvement of engineer or architect in such constructions. These constructions do not follow any code of practice or local standards of the country. The construction is of poor quality with low-quality mortar and lack of integrity. The key features addressed in seismic code of Pakistan are still not followed as evident in Figure 2.29 where the figure on the right shows the opening size to be in excess of the prescribed limits for a seismic zone construction and no reinforcement is provided for window edges, along with a discontinuous lintel band shown in the left figure.

Figure 2.29: Discontinuous lintel band (left), Large window opening (right) [11]

Following the earthquake of 2005 several studies have been conducted to make recommendations for safer construction in the region using locally available material and construction techniques. Revised building code with seismic provisions has been introduced for the region in 2007, which details safer design and construction guidelines. Specifications for new masonry construction have been included along with reinforcement detailing. These specifications are for new construction and the code does not deal with old masonry structures and their strengthening measures. The code can only be interpreted by an engineer who has prior seismic design knowledge and in no way helps the masses to comprehend and apply them in their house constructions. As most of the constructions in Kashmir are non-engineered, therefore, several Non-Government Organizations have conducted workshops to train masons and people for safer earthquake construction practices.

Even after the occurrence of 2005 earthquake no seismic strengthening has been observed in the region [11]. Main reason for people still not employing strengthening methods in their construction practices is due to the high cost or the complexities of application procedure. These areas need retrofitting techniques that uses available material with easy application and low cost.

2.8 Numerical Modelling of Masonry

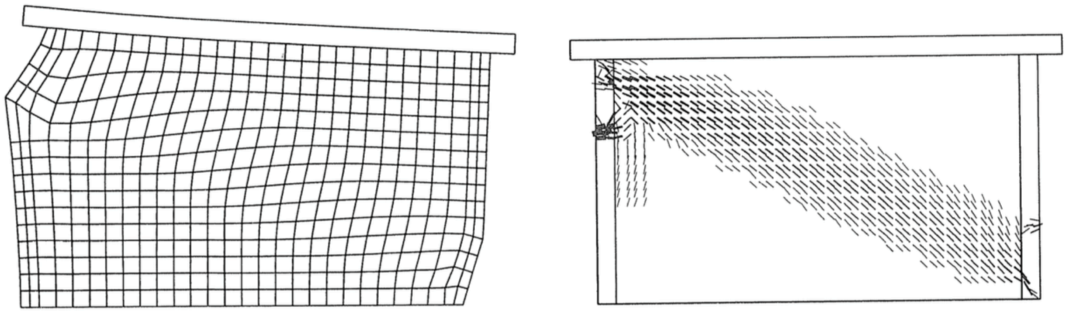
Researchers have carried out extensive numerical modelling for masonry and the modelling of masonry has been characterised in two main categories, *Macroscopic* and *Mesoscopic* modelling as defined by Kurt et al (2011) [90]. While Bigoni and Noselli (2010) calls them *Macro* and *Micro* modelling, respectively [91]. The two modelling techniques are also referred to as *Smeared* and *Discrete* crack models [92], and *Continuum* and *Discrete* models [93], respectively.

a) *Smeared crack model:*

Smeared crack approach is used in continuum or macro modelling technique where the masonry wall is modelled as a homogenous shell for 2D or solid continuum for 3D analysis. The idea behind smeared crack approach is that matrix aggregate composites such as concrete when subjected to tensile failure undergo micro-cracking (Figure 2.30), tortuous debonding and other series of internal damage, which eventually come together to form a geometric discontinuity in material called crack [92].

The characteristic of masonry in such type of modelling is the combined property of masonry unit and joint mortar. Smeared crack concept can be defined in terms of stress-strain relation with tensioning softening parameters included to allow the elements deterioration/damage. This idea of using stress-strain relation for continuum model fits the idea of finite element analysis, but however conflicts the realism of geometric discontinuity that separates the material and produces cracks [92].

Continuum or Macroscopic models are much easier and quicker in terms of modelling, and the failure pattern is smooth over a certain failure area [90]. The continuum models may provide satisfactory results but fail to provide a practical method for masonry analysis [94], where the geometric discontinuities or cracks have predefined orientation.



(a) (b)

Figure 2.30: Deformed shape (a) and cracks (b) in masonry through Discrete Modelling approach [95]

b) *Discrete crack model:*

Discrete crack approach is used in micro- or mesoscopic modelling technique, where the masonry wall is made up of discrete masonry units connected to adjacent units by using mortar definitions. The connection between interacting bricks can either be modelled as surface interaction, or as cohesive elements. In theory the discrete crack approach replicates the idea of cracking more closely because the crack is modelled as a displacement-discontinuity in the interface element that separates the two adjacent solid elements [92].

As the cracks in masonry normally occur due to the debonding of bricks or in other words failure of mortar joints (Figure 2.31), therefore, the location and orientation of cracks is predefined. This idea favours the use of discrete crack approach as it provides a better estimate of masonry cracks and its progressive collapse [93].

The disadvantage associated with the discrete modelling is large number of degrees of freedom and thus a higher computational cost [90]. The effort required to model masonry using discrete crack approach is much greater in comparison to smeared crack approach; therefore, this time taking process should be avoided by the use of an algorithm that automatically places each brick in appropriate position and assign appropriate mortar contact definition between the interacting surfaces of masonry units.

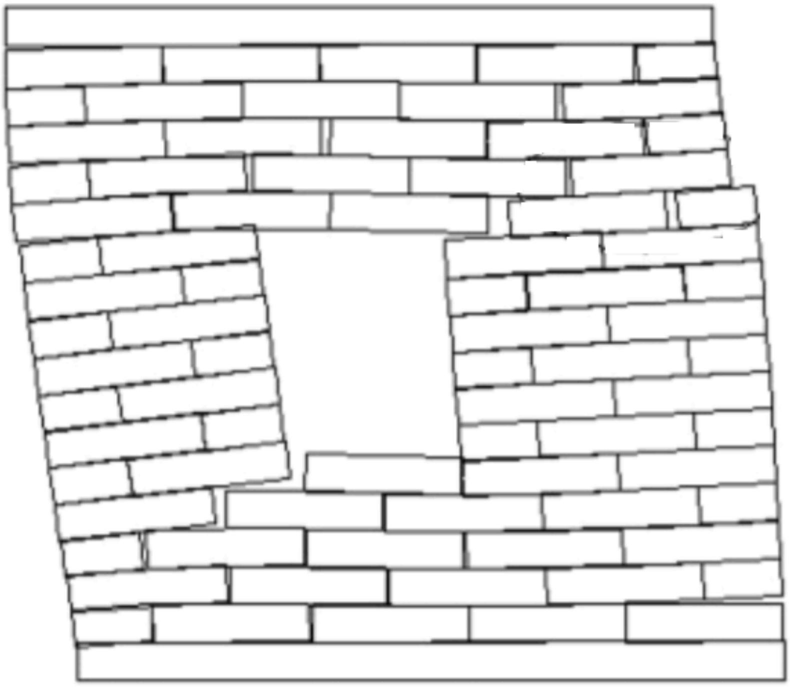


Figure 2.31: Masonry Cracking under Discrete Modelling approach [90]

2.9 Conclusions and Summary

Unreinforced masonry caused hundreds and thousands of casualties during past earthquakes due to its weak shear and tensile strength as compared to RCC or steel structures. However, the ease of construction, material availability, economy, aesthetics, sound and fire insulation makes it an attractive construction material in many rural and urban localities. To reduce casualties during earthquake, masonry structures should be appropriately strengthened to resist seismic loads. For this purpose, an understanding of masonry behaviour and its failure modes to identify the potential weaknesses in the structure is necessary to effectively provide retrofit. For this purpose, bench-scale experiments of shear walls are carried out and their findings are presented in Chapter-3.

Kashmir has been selected for case study under the funds provided for this research to help development in the region. It is located in the mountain ranges of Himalaya and has poor accessibility thus, setting it back in terms of economy and technology. The region has a history of frequent seismic activity and high number of casualties during past earthquakes. The region has abundance of non-engineered masonry structures constructed using locally available material. These structures do not follow the code specifications and require urgent retrofit to minimize the risks in case of any future earthquakes. To better understand the construction practices and problems faced by the construction industry in Kashmir, site visit of Muzaffarabad city and its surrounding villages was carried out. The details and findings of the site study are documented in Chapter-4.

After a detailed investigation in to the state of the art seismic retrofitting for masonry structures in terms of seismic efficiency, material availability, ease of application and economy, PP-band retrofit has been selected as the most viable retrofit solution for Kashmir region. The foremost reason for selecting PP-band retrofit is the low cost of PP-band and its availability in the region. The application process itself is also economical, as it does not require heavy equipment or skilled labour. The retrofit method is also sustainable if strips from packaging are reused. The only compromise made with PP-band retrofit is the aesthetics because the bands need to be covered with plaster to protect from UV radiations or any other physical damage.

Final recommendation on PP-band retrofit, as the viable solution in the region, can only be made after a thorough investigation of its seismic performance and application details is carried out with the help of experiments and numerical analysis. Therefore, a series of experiments are presented in Chpater-5 to obtain a better understanding of the properties of the material used in the region and provide data for numerical modelling. PP-band retrofit efficiency and its application on structures are studied through shake table tests conducted on wallette specimens and room structure in Chapter-6 and 7, respectively. Finally, an attempt is made at numerical modelling of masonry to predict cracks. Two different types of modelling strategies discussed earlier in Section 2.8 are used for carrying out the numerical analysis of masonry and the details of these modelling techniques and their results are given in Chapter-8.

Chapter 3 : Reconnaissance Study in Kashmir

3.1 Introduction

This chapter deals with the understanding of construction practices and problems faced by the local construction industry of Kashmir. Information provided in this chapter and the conclusions drawn are solely based on the field visit of Muzaffarabad and its suburbs. During this field trip, interviews were conducted with people involved in the local construction industry such as contractors, architects, masons and building officials (transcribed interviews given in Appendix-C). Based on the information provided by the interviewees and self-inspection of the region the inferences drawn are presented in this chapter.

3.2 Methodology

The city of Muzaffarabad, which suffered greatly during the 2005 Kashmir earthquake, was chosen as the site for research study. For better understanding the local construction practices and their problems; and for assessing the practicality of the selected PP-band retrofitting it was imperative to personally visit the region to acquire first hand knowledge of the local construction industry. Field study was conducted between15/12/2013 to 25/12/2013 during which people working at different posts in the construction industry such as, building officials, contractors and masons/labours were approached with questionnaires. In practice, the questionnaires were not handed to the people instead verbal question answer session was conducted and the answers were voice-recorded electronically. The names and identities of the people involved in the survey are kept confidential.

Following the visit to Muzaffarabad and its suburbs, another city called Mirpur was also visited to understand the construction practices of a region that was not badly devastated during 2005 earthquake but had its seismic zone revised to a higher level. This city lies in Kashmir region and is located approximately 200 km south of Muzaffarabad. The major construction material in the region is unreinforced brick masonry; therefore, it was decided that the city of Mirpur should be visited to inspect the level of preparation for future earthquakes.

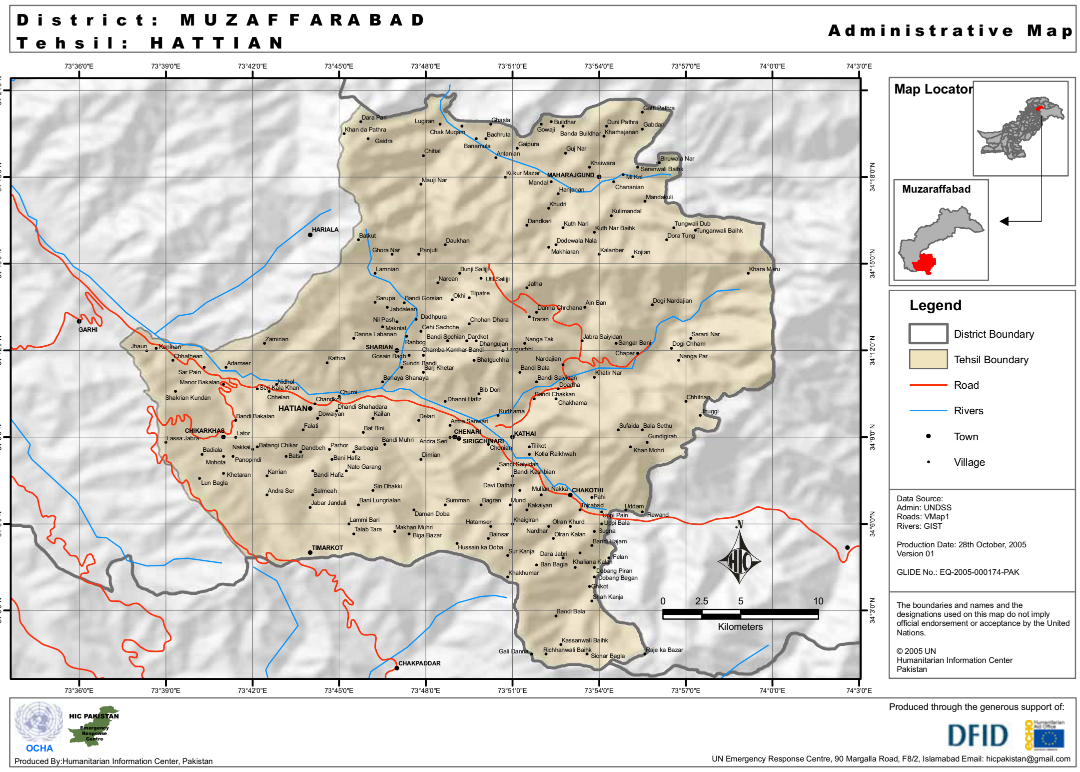


Figure 3.1: Map of Muzaffarabad showing its location [96]

3.3 Muzaffarabad

3.3.1 Pre 2005 Earthquake

a) Materials:

The local construction industry in the region was dominated by concrete frame structures and load bearing masonry wall systems. Concrete frame structures with block masonry infill, which served as partitioning walls, were mostly used for multi storey or commercial structures. Local authorities also prescribe concrete frame system for multi storey structures (i.e. 3 or more storeys) and commercial buildings. As red clay bricks are not locally made in the region therefore they had to be imported from other cities, which resulted in higher cost of bricks. However, brick masonry load bearing wall system was still the most preferred construction system for 1 or 2 storey single-family dwellings. Most government buildings were also load bearing brick masonry constructions. Floors and roof slab were cast in-situ reinforced concrete resting on masonry walls without the use of dowels for slab-wall connection.

Stone masonry was another common practice of the region that dominated the rural and sub-urban constructions. In rural and hilly areas stones are more easily available as compared to bricks, cement or steel, and are economically more feasible considering the region's economy. Occasionally, masonry would be combined with the locally available timber beams and columns, or a series of timber panels filled with stones called *Dhajji* construction (Figure 3.7). This construction style is later on discussed in detail in this chapter. Roof usually consisted of wooden panels covered with mud and earth or timber truss with corrugated iron sheeting.

b) Construction Practices:

As the local authorities were not expecting an earthquake in the region therefore there were no seismic provisions available in the local construction codes for the region. Structures were designed and detailed to sustain gravity loads. Pre 2005 earthquake, most of the constructions did not meet the specified provisions of the building codes e.g. material quality, mix ratios, brick-laying procedure, etc.

The contractual system was one of the main factors for poor construction in the region along with poor regulation and supervision of the authorities. Field study revealed that contractor's selection of material quality and quantity is based on the finances available from owner. Due to the poor supervision by the local authorities details shown in the drawings, presented to local authorities for approval, often differed from what was actually constructed on site. Privately hiring an engineer or architect for site inspection or supervision was unaffordable, or in other cases believed to be unimportant.

Single family dwelling of one or two storeys had no involvement of engineer in design and detailing of the structure and an architect's design were deemed acceptable in such cases. However, hiring an architect or engineer was too costly for most people to afford and they would rather have their structure designed and detailed by either a draftsman or a contractor who would follow a set pattern for layout and detailing. For public buildings or structures having 3 or more storeys, local authorities made it mandatory to use concrete frame system and get the structure designed and detailed from an accredited engineer. However, due to the lack of funds and technical staff, local authorities failed to implement these regulations.

Site selection was poor; as flat land is scarce and expensive therefore most people would build wherever they could acquire land. Hill slopes were cut to a straighter angle and construction was carried adjacent to the cut face without providing any gap between the structure and the hill slope. Furthermore, the size and location of openings had no limitation and were provided at will, thus having irregular plan and complex architectural features (Figure 3.2). Storeys above ground level would have outward projections and in many cases discontinuous columns which resulted in complex load path and structural unevenness. Incremental construction was another common practice in the region, which led to the addition of floors, to meet the growing requirement of the family, without any consideration for soil's or foundation's load bearing capacity.

Figure 3.2: Structure with large openings in one direction and discontinuous RCC horizontal member

In case of brick masonry constructions major cause of weakness was insufficient brick curing before laying e.g. instead of immersing the bricks in a water tank for sometime, the workers would simply spray water with a hose on brick stack. After the laying of bricks the required amount of curing was again inadequate which subsequently resulted in bricks absorbing the water from mortar and not allowing the masonry to develop complete strength. Even with all these drawbacks in the construction, brick masonry performed better in comparison to stone masonry and with careful construction and reinforcement masonry structures even survived the earthquake.

Due to economical constraints of the owners concrete ratio used for construction was normally 1:3:6 (cement: sand :aggregate), which is less than the minimum 1:2:4 ratio (21 MPa compressive strength) specified for structural members resisting earthquake induced forces according to 'Building Code of Pakistan - Seismic Provisions 2007' (Clause 7.3.4) [38]. Steel reinforcement for main bars and stirrups would be provided in lesser quantity than design requirement to cut cost. Insufficient lap lengths and improper beam joints were a common practice. Stirrups spacing were normally kept at 375-450 mm (1.25-1.5 feet) and lap lengths were around 300 mm (12 inches) or even less. Columns used to have four reinforcement bars in case of typical houses. Footing depths varied between 450-900 mm (1.5-3 feet) with no reinforcement provided in the footing pad. Column reinforcements would rest vertically on the footing pad without providing the necessary lap bend to spread the load from the columns on to the footing. Lintel beams were provided only above the openings and there was little evidence of continuous horizontal RCC band at lintel or any other intermediate level i.e. sill level, plinth level.

Rural constructions used locally available undressed stones with dry masonry rubble infill, instead of a proper grout material between the two wythes of the wall, along with mud/lime mortar for joints. In some cases the stone masonry was used with timber beams and columns or, a series of timber panels with stone infill called *Dhajji*. Due to the dry fine rubble infill between two wythes the wall behaved as two separate leaves and thus collapsed instantaneously during earthquake. Stone masonry construction lacked the provision of bigger stone pieces in the wall, which serves as a connection between the two wythes of the wall. Wall corners had no consideration for interconnecting stones to provide adequate bond between the two walls. Stone masonry structures that had proper grouting, opening lintels and jams did, in most cases, survived the earthquake with minor damage. The region thus lacked in skilled stonemasons with proper knowledge of stone masonry construction standards.

3.3.2 Post 2005 Earthquake

a) Materials:

Most of the concrete frame structures survived the collapse with partial or little damage and those that collapsed allowed sufficient time for evacuation. According to the local building authority official interview for this study, 80% of the new construction happening in Muzaffarabad is using concrete frame system with block masonry infill. The seismic performance is not the only reason for the growing popularity of concrete frame system; but also the fact that it uses cement blocks, which are bigger in comparison to fired bricks thus making wall construction cheaper. These block units can be made in the locality on a flat surface using cement, locally available sand aggregate and simple moulds without the need for special furnaces and clay.

Brick masonry structures on the other hand suffered quite a lot during 2005 earthquakes especially the unreinforced masonry structures, but still performed a lot better than stone masonry due to the regular size of the bricks. The high cost of brick import and its unsatisfactory performance makes it no longer a preferred construction material. However, government buildings are still using brick masonry along with RCC confinements. This encourages the masses to adopt load bearing masonry construction technique for their structures but with the provision of RCC confinements or embedded steel bars for seismic performance.

Stone masonry was the most problematic material as all unreinforced stone masonry structures collapsed during the earthquake without allowing any time to its habitants for evacuation. After 2005, stone masonry is no longer used in the city or its suburbs because of its poor performance but is still prevalent in rural areas and villages located high on mountains.

b) Construction Practices:

After 2005 earthquake lateral loads were recognised and with the new seismic zoning of the country Muzaffarabad moved from seismic zone 2 to zone 4 (that is, moved to a more earthquake prone zone). Current seismic zoning for the region is shown below in Figure 3.3.

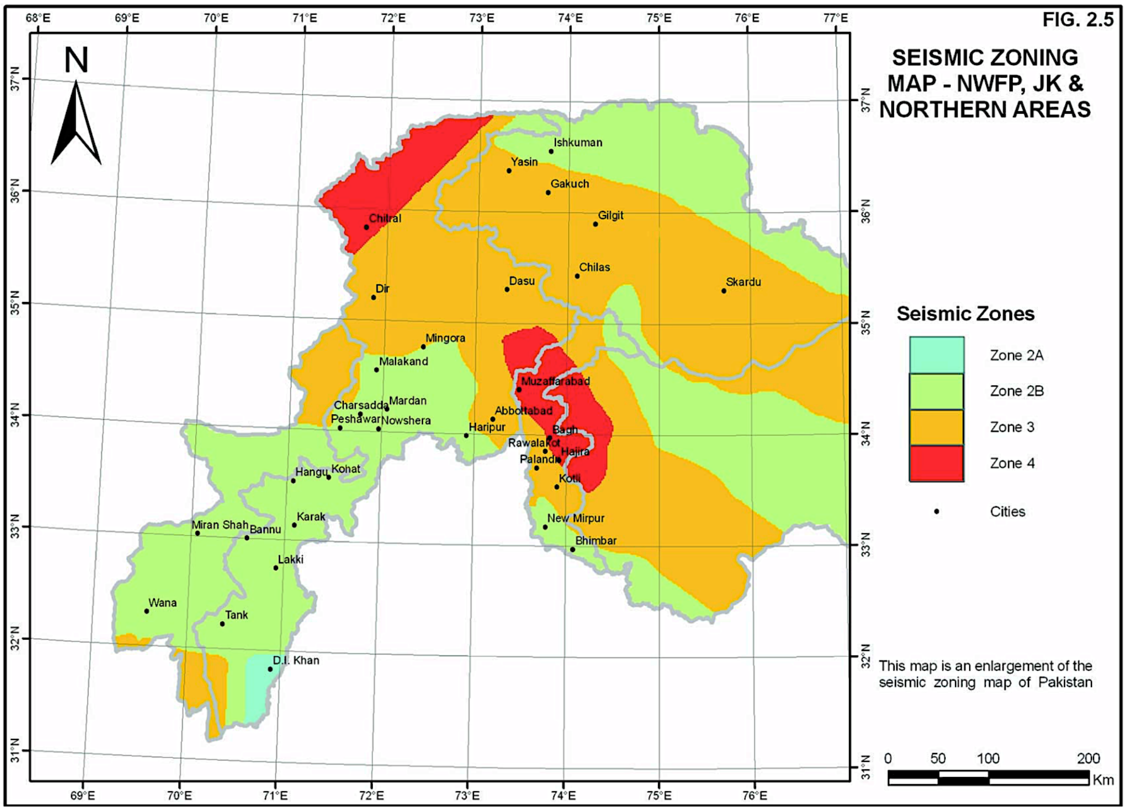


Figure 3.3: Seismic zoning of NWFP, Jammu & Kashmir and Northern Areas of Pakistan [38]

Table 3.1: Seismic Zones as per Building code of Pakistan, 2007 [38]

Seismic Zone	Peak Horizontal Ground Acceleration
1	0.05 to 0.08g
2A	0.08 to 0.16g
2B	0.16 to 0.24g
3	0.24 to 0.32g
4	> 0.32g

Where “g” is the acceleration due to gravity.

Figure 3.4 shows seismic hazard map for peak ground accelerations with a return period of 500 years. Muzaffarabad city, located in Pakistan occupied Kashmir, and the region around the Main Thrust Boundary, as shown in Figure 3.5, has a high expected PGA of around 0.43 g or 4.31 m/s^2 with 500 year return period.

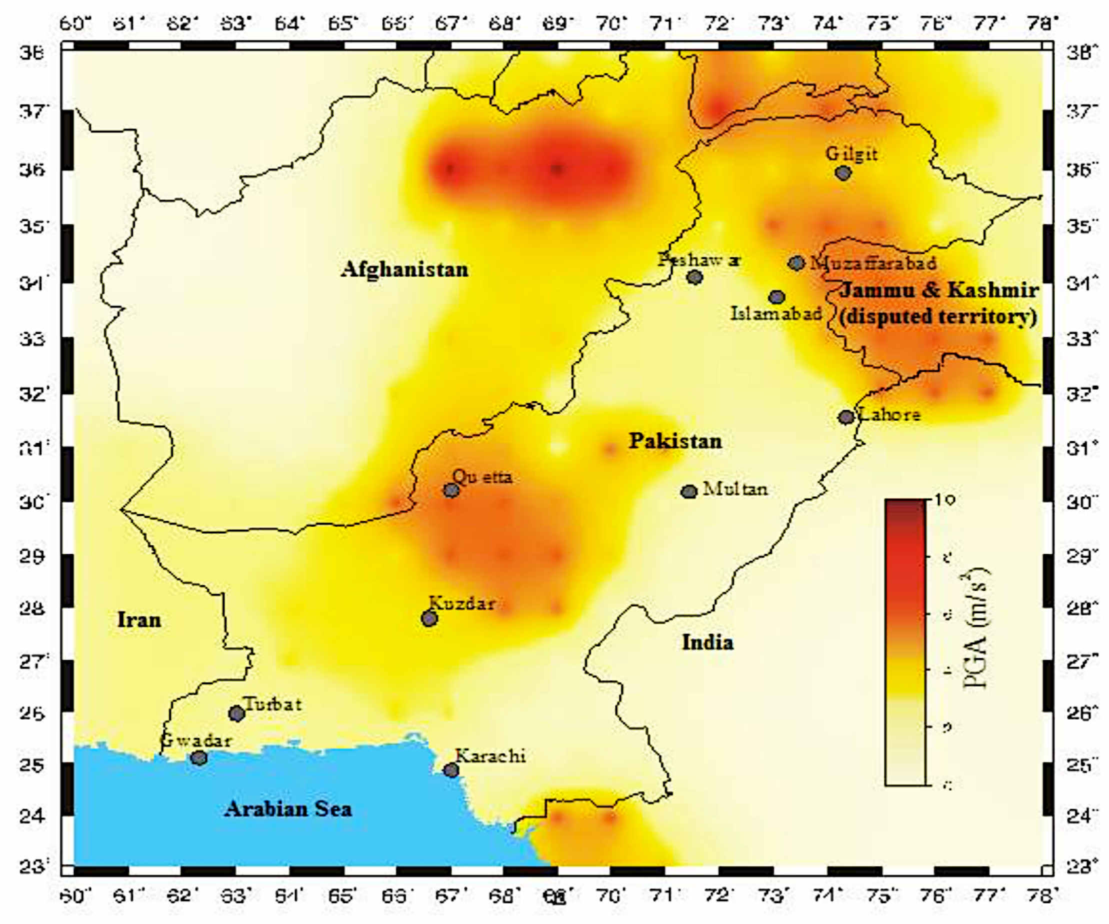


Figure 3.4: Seismic hazard map of Pakistan prepared for PGA for 500 years return period [97]

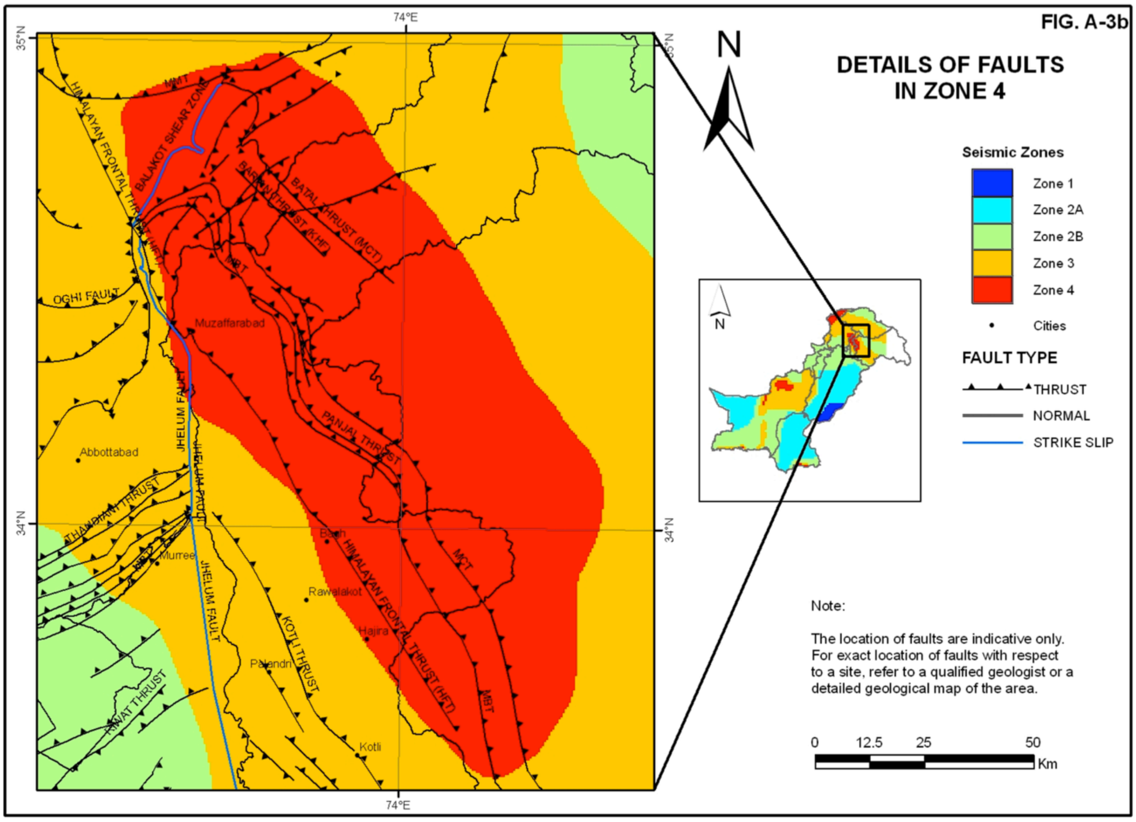


Figure 3.5: Details of faults in Zone 4 of Kashmir region [38]

Followng 2005 earthquake seismic provisions were introduced in the Building Code of Pakistan in 2007 and compliance to these provisions was made mandatory for all works conducted in seismic zones 3 and 4. But due to the lack of efficient system and work force in the local authority, these provisions could not be fully implemented. Public works did adhere to the new standards of construction but in private sector, due to financial constraints, people are not in a position to fully comply with the specifications of the code. Hiring an engineer or architect still remains costly for the masses that are struggling to overcome the losses suffered during the earthquake. These structures add to the existing stock of non-engineered structures in the region.

Figure 3.6: Government school building with proper confinements and lightweight steel roof

After 2005 local authorities emphasized masonry construction for single or double storey structures with smaller room sizes. For multi storey structures or buildings with larger spans RCC frame construction is made obligatory. For bigger room and opening sizes in housing works, RCC confinement should be provided for masonry load bearing construction. Due to the poor performance of masonry in 2005 earthquake in comparison to concrete frames along with the added cost and complicacy of RCC confinements in masonry load bearing structures, RCC frame construction is becoming more dominant in private works. This type of construction uses concrete frame for load bearing and block masonry infill as partitioning walls. The use of redbrick masonry as load bearing construction is getting limited to public works (Figure 3.6).

Following the 2005 earthquake, NGOs such as, EERI, KIMSE YOK MU, Sarhad Rural Support Programme, Earthquake Relief and Rehabilitation Project and other international and national government organisations such as, State Earthquake Reconstruction & Rehabilitation Agency, Earthquake Reconstruction & Rehabilitation Authority, Japan International Cooperation Agency, and several others carried out workshops to educate locals about efficient construction techniques for earthquake resistance. Emphasis was made on following construction standards and cash incentives were announced to those who implement seismic provisions in their construction. These provisions included the use of reinforced masonry technique in which maximum room size was restricted to 4.5 m (15 feet). Every corner should have 16 mm (5/8 inch) vertical bar starting from the foundation along with stitches made of steel or concrete in L and T shape spaced at 450 mm (1.5 feet) along the height of the wall. Reinforced concrete band of 100 mm (4 inches) with two steel bars of 12.5 mm (½ inch) diameter were suggested at plinth level. All edges of window and opening should have a vertical bar from plinth level to roof level. Soon after these vertical bars were suggested at every 1.2 m (4 feet) distance along the span of the wall. Sill, lintel and roof levels of the wall were to be reinforced with a continuous RCC band of 75 mm (3 inches) [98]. These provisions were similar for block, brick and stone masonry construction. Authorities claim this technique to be implemented in rural areas but not in the city because of the shift to concrete frame structures.

Figure 3.7: Dhajji Wall (Top), Patchwork quilt, 'Dhajji' in Persian [99]

Local architecture had more usage of wood prior to 2005 earthquake especially in urban areas where more and more people are moving to concrete frame system. Local timber frame structures known as *Dhajji* served as a breakthrough for earthquake resistant construction. *Dhajji* type construction consists of small panels of braced timber frames filled up with stone or brick masonry with mud mortar between them. This construction technique takes its name from the Persian word 'Dhajji' which means patchwork quilts in the ancient language of carpet weavers [99]. According to the local government official, there were 2,500 dhajji structures before 2005 earthquake and displayed satisfactory performance during the earthquake. Now the number has increased to 125,000 dhajji structures in the rural regions located at higher altitudes.

Column stirrups in concrete frame constructions now have 100-150 mm (4-6 inches) of spacing with six main bars instead of four, which used to be the earlier practice. Concrete ratio changed from 1:3:6 to 1:2:4 for private works and 1:1.5:3 for government works. Main reinforcement bars for structural members prescribed are #4 to #6 bars depending on the type of structure. Significant change was made to the shape of stirrups by providing a bend of 25-38 mm (1-1.5 inches) at the ends of the bar to enhance the seismic resistance of stirrups. Lap lengths were revised to 450-600 mm (1.5-2 feet) and stirrups on lap should have a spacing of 100 mm (4 inches). Half of the city population is now providing plinth beams in their construction. Footing pads now have steel reinforcement mesh with proper lap from column reinforcement to distribute the load from the column.

Generally, people are now more aware of the earthquake damage and are making effort to provide some form of confinement in their structures. Those who cannot afford complete RCC confinement try to provide partial confinement. According to local contractors, house owners now consider building a smaller house in case of financial constraints rather than compromising on the material quality or construction standards. All this awareness was created by the collaboration of research societies and NGOs with the local authority. Everyone now seem aware of the danger of earthquake but due to the financial constraints some people are still taking risks with their constructions.

Masons have realised the need for proper curing of bricks and leave them in water tanks for sufficient time to allow the bricks to absorb adequate moisture before laying. In case of stone masonry, masons have been educated to provide *connecting* stones between the two wythes and corners of the wall to achieve better structural integrity. They have been advised to stabilize the cut hill slope with reinforced concrete and provide a gap of 600-900 mm (2-3 feet) from the structure. The wall marked ‘A’ in Figure 3.8 is the back wall of the house against the hill slope and constructed of reinforced concrete. Zone ‘B’ shows the discontinuity between the column of the two storeys and the gap between the beam and the rear column. These types of negligence are still prevalent in the local construction industry due to the lack of engineers and negligence from local authorities in monitoring and regulating construction standards.

Figure 3.8: Back wall against hill slope constructed of RCC

Similar to pre 2005 earthquake, there is no concept of material testing and quality assurance. Those who can afford to use better quality material in their structures while the rest use sub standard and cheap materials for their construction. Many people due to their economic condition use debris of the previously collapsed structures as construction material. The only material-testing lab in the region available with the local authority is in poor condition due to inactivity and negligence. Site investigation or soil testing is carried out for government works only. The depth and size of footing is usually based on the contractors' judgment of the soil condition. Due to scarcity of flat land and high land prices in flat terraces people build wherever they can acquire land without any consideration for site hazards, as shown in Figure 3.9.

Figure 3.9: Congested terrace construction without proper planning

Local authority has restricted construction on the fault line that passes through the city called ‘Muzaffarabad Thrust Fault’ (Figure 3.5). The region along the fault line is marked as red zone where construction is prohibited. But due to the financial constraints people with their lands in the red zone are still constructing in and around the red zone [100]. Local authorities do not having the resources to relocate these people, advise them to at least limit their constructions to temporary structures using steel or other lightweight material. Figure 3.10 shows an example of poor site selection common in the region. The structure on top and bottom of the hill face imminent danger of landslide as the region of Muzaffarabad has medium height hills of clay and rubble. This kind of loose and soft material is not only prone to landslides during earthquakes but also to rainfalls, which seem to be abundant in the region. The annual rate of rainfall in Pakistan controlled Kashmir region amounts to 418.7 mm, which is lot higher than the country’s average of 297.6 mm for data recorded from 1961-2010 [101].

Figure 3.10: Poor site selection

The PP-band mesh technique for masonry construction introduced by University of Tokyo has not been much used in the region. No new structure had been constructed using the technique. The only structure with PP-band reinforcement in the region is the one constructed by the Tokyo research team themselves during their visit following 2005 earthquake for training local masons to employ PP-band mesh technique to their structures. That structure has been covered with plaster and now is being used by a local bank as shown in Figure 3.11. The main reason for the PP-band mesh retrofitting technique not being implemented by the locals in their constructions is due to the lack of awareness amongst the masses regarding its reliability and due to the lack of fixing details. In addition, the model structure constructed in Muzaffarabad uses lightweight roofing as opposed to the hard concrete floors and roof system which people are more commonly use for their housing structures. After the initial training program by the Tokyo team no further workshop was carried out for this technique and thus only a handful of people are aware of it. New construction in urban areas is using concrete frame system with block masonry infill, which makes the use of PP-band retrofitting unnecessary.

Figure 3.11: (Left) Structure in-use by local bank. (Right) PP-band mesh structure [73].

3.4 Mirpur

Mirpur district is located in Pakistan controlled Kashmir region with its capital city called Mirpur City. It is situated approximately 200 kms south of Muzaffarabad and is mostly plain area as opposed to the mountainous terrain of Muzaffarabad. Major construction material in the region is unreinforced red brick masonry especially for single-family dwellings. As the region was quite far from the epicentre of 2005 earthquake therefore the intensity of ground vibrations in 2005 and its subsequent damage was minor as compared to Muzaffarabad region. However with the new seismic zoning of the country the region now has a higher potential for seismic activity and therefore the region was studied for assessing the local construction practices to avoid any future damages similar to the one suffered in Muzaffarabad during 2005 earthquake. The district of Mirpur now lies between zone 2B (0.16-0.24g (g = 9.81 m/s^2) and zone 3 (0.24-0.32g) [38].

Figure 3.12: Absence of RCC lintels over openings

Construction in Mirpur has no seismic provision and follows the exact same practice as seen in Muzaffarabad. With unreinforced masonry dominating the city construction has the same weaknesses as mentioned in the Pre 2005 construction of Muzaffarabad. For instance, discontinuous lintels or no lintels are provided instead of complete horizontal band running through all walls (Figure 3.12).

Figure 3.13: No consideration for opening size and location

Guidelines provided for opening sizes and locations are often neglected as shown in Figure 3.13. Another noticeable feature in the figure is the projection in front of the room for providing a shaded walkway in the courtyard. This projection rests on slender columns with unreinforced shallow footings, and no plinth beam is provided as opposed to the standard for house construction.

Figure 3.14: Projected overhead storage room

Architectural recommendations for construction in seismic zones are not followed in the region. People construct fancy architectural features with irregular geometry, plan and shear member distribution that results in disparity between centre of mass and centre of rigidity. Such complexities are always a big nuisance during earthquakes especially in masonry construction. In Figure 3.14 an overhead storage room is shown which is projected on one side and the slab supporting the overhead structure has no beam or column to support its cantilever action. The slab is supported on the bottom walls without any reinforced connection to prevent it from toppling during earthquake.

Another significant feature for seismic resistant masonry is the joints between two walls such as the corners of structure. Mirpur construction is negligent of the implications of proper wall joints during an earthquake (Figure 3.15).

Figure 3.15: Poor Corner joints

Figure 3.16: Stairs resting on the wall of structure

Earlier in Chapter-2 it was noted that staircases or where the height of the structure differs should be separated from the main structure and should not support or be connected to one other. In Figure 3.16, this aspect is clearly neglected and the wall of the adjacent structure supports the staircase. This type of negligence can result in complex stress due to the projection of staircase and cause damage to the main structural members.

3.5 Concluding Remarks

Construction in Muzaffarabad still lacks the standards of seismic zone construction. Major factor contributing towards the substandard construction is the poor economic conditions of the region and the negligence of local authorities. Those who can afford have safer and durable structures while, the rest have no other option than to take chances with their construction. Current field study dealt with the construction practices and problems of Muzaffarabad city and its suburbs. Factors affecting rural constructions on higher altitudes cannot be analysed with certainty but only with the information gathered from the questionnaires. Economic conditions in these regions are much worse with limited access to modern technology and materials. The best start towards safer society with respect to earthquakes would be to effectively monitor and regulate the construction in the region. Local authorities should make sure that building codes are being followed and should facilitate people in doing so by providing necessary guidance and incentives.

Local disaster management authority, which is supposed to provide shelter and rescue during natural calamities such as earthquakes, flood and landslides, is apparently non-existent in the region. Interviews with the locals suggest their inactivity just after a few years following the 2005 earthquake. No awareness exists amongst the masses regarding the presence and functionality of such organization. People are not made aware of any disaster management plan to cope up with future earthquakes.

Most of the people interviewed for this study suggested 70% less damage in case of a similar earthquake as of 2005. Few mentioned the devastation to go up to the same levels as that of 2005. The main reason for lesser damage prediction in comparison to that of 2005 is due to the shift to concrete frame construction system. Whereas, the proper understanding of the reinforcement detailing system required for seismic resistant construction is still inadequate. The location and orientation of columns is decided to match architectural purposes rather than to achieve same rigidity in both orthogonal directions. The concept of coinciding centre of mass and rigidity is non-existent because most housing constructions are non-engineered and designed and detailed by contractor.

Construction practice in Mirpur is negligent of the possibility of future earthquake and its effect on structures. This region should be given attention to strengthen its constructions before an earthquake of greater magnitude strikes. The reason for mass damage in Muzaffarabad was due to similar flaws in construction, and the negligence of the earthquake prediction. With seismic zones been revised for Pakistan, Mirpur now lies in Zone-3. It is high time to take lesson from Muzaffarabad and take appropriate measures to minimize the damage in future earthquakes. The best start would be to train the local masons about the key features of masonry construction discussed in section 2.4. They should be made aware of its implications and effects on structure's performance during earthquake rather than trying out expensive strengthening methods or materials.

Chapter 4 : Preliminary Shear Tests

4.1 Introduction

For the purpose of this research it is essential to understand the behaviour of masonry shear walls under different configurations. Seismic loads are dynamically induced cyclic loads that act, both, vertically and horizontally on a structure. As the conventional design takes into account the vertical gravity loads therefore, seismic design is chiefly concerned with the effects of horizontal or lateral loads. For the sake of simplicity and better understanding of masonry failure patterns, the experiments were conducted under static one-directional lateral loads. The nonlinear static response of the shear walls offers close representation of failure patterns observed under dynamically induced one-directional loading [102, 103]. Small-scale walls made out of *Medium Density Fibreboard* (MDF) blocks were used to study the in-plane behaviour of shear walls (Figure 4.1). This type of setup saves time and cost for construction, allows easy handling of the specimen and the bricks can be reused for subsequent trials.

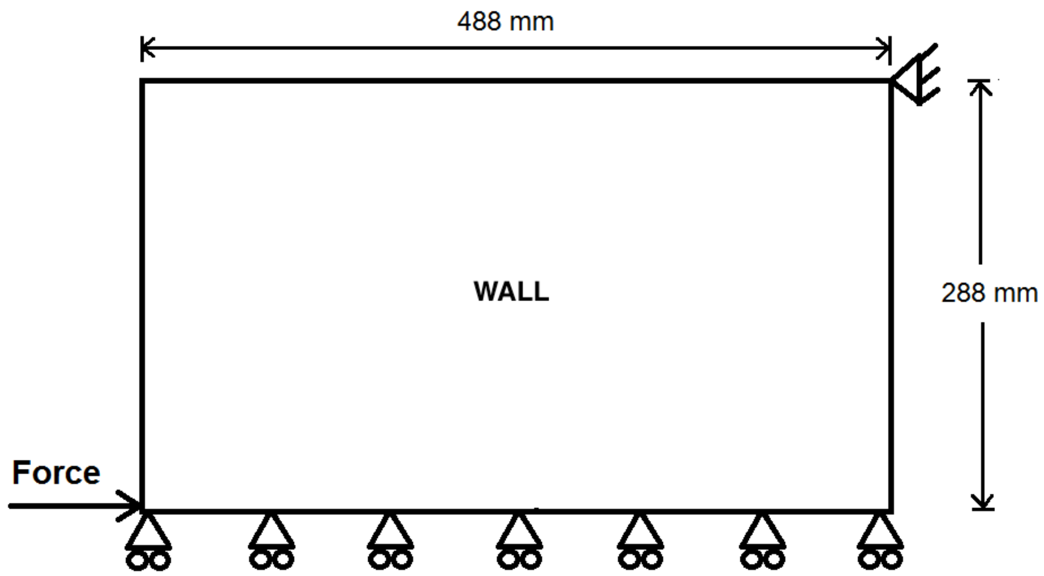


Figure 4.1: Schematic for shear test of masonry walls

4.2 Objectives

Objective of the experimental work was to study the failure pattern of small-scale wall under lateral base displacement and associate it with the behaviour of actual masonry during seismic loading. In this chapter an attempt is made to identify the cracking pattern for different types of masonry walls i.e. with and without openings. Identifying the locations of potential weaknesses in masonry would aid in achieving a better earthquake resistant design for masonry. Furthermore, the specimen would be strengthened using PP-band retrofitting method and the tests would be repeated to study the effects of retrofitting on crack formation.

Finally the experimental setup will support verification of FE simulations of lab experiments (Chapter-8).

4.3 Experiment Methodology

Analysis was carried out on freestanding small-scale masonry walls (480mm x 288mm) made from *Medium Density Fibreboard* (MDF) blocks of size (12mm x 12mm x 24mm). Following wall types were studied to observe the crack patterns generated by lateral base displacements:

a) Solid Wall
   i. Glued
   ii. Non-Glued
   iii. Non-Glued Retrofitted
b) Wall-with-Opening
   i. Glued
   ii. Non-Glued
   iii. Glued Retrofitted

The walls were supported on horizontal base of MDF strip hardly bonded to a Perspex sheet, which served as a background and as a support for wall in its out-of-plane direction. Vertical and horizontal restraints were applied to the top-right corner of the wall and horizontal displacement was applied at the bottom-left corner with the help a loading screw to induce shear failure in the wall (Figure 4.1 and Figure

4.2). To record the appearance of cracks and to study their patterns, pictures were taken at an interval of every 1 mm push, which corresponded to the thread pitch on loading screw. Experimental setup and the associated dimensions are shown in Figure 4.2.

In case of glued wall specimens to replicate the effect of mortar cohesion paper-glue diluted with water, with glue-to-water ratio of (1:5), was used for sticking the blocks together. The dilution allowed for the bonds to break easily without damaging the MDF bricks. For retrofitting, strips of sellotape in place of PP-bands were used. The width of sellotape strips was 25 mm and they were tied around the wall in two orthogonal directions. The bands were tied around the wall to maintain the integrity of wall after crack initiation and replicate the effect of PP-band retrofit in real masonry i.e. around the edges of wall and openings. Retrofitting was applied at locations that were deemed to be the point of potential crack opening. For solid wall five sellotape bands were evenly distributed in both directions as shown in Figure 4.3. In case of wall-with-opening retrofitting bands were provided at wall edges and at the edges of opening as shown in Figure 4.4. For walls with opening, a window was provided in the wall and a horizontal lintel of MDF was provided over the opening.

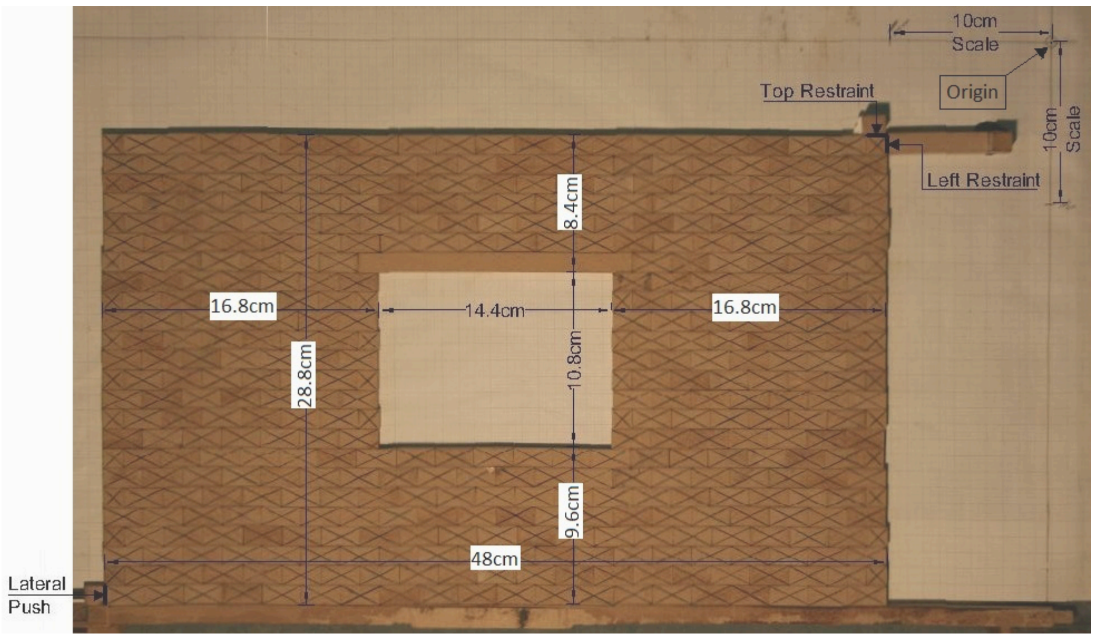


Figure 4.2: Lab Model of Small Scale Solid Wall

Figure 4.3: Retrofitted solid wall

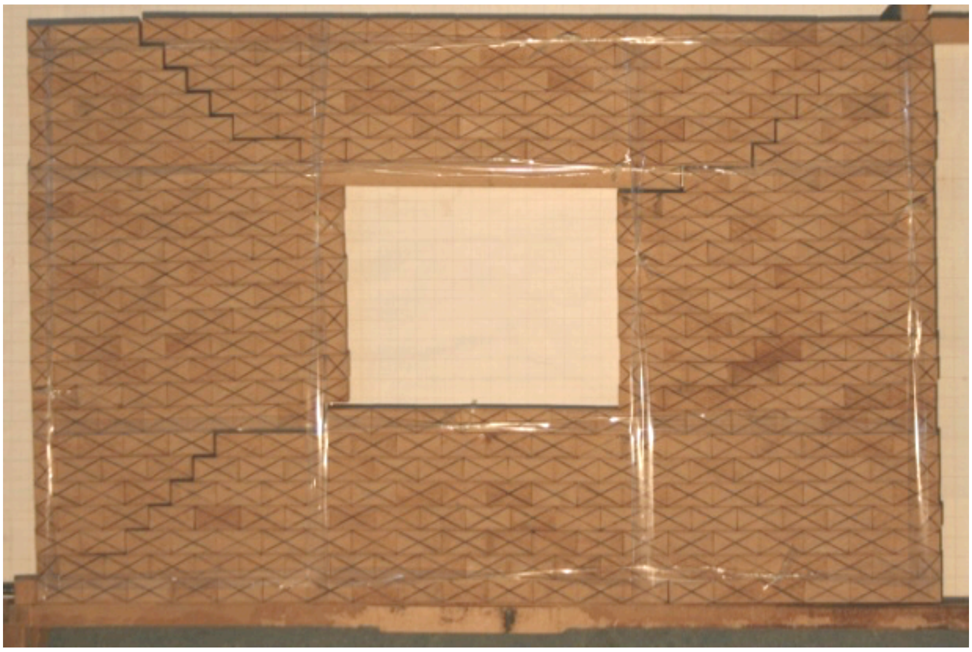


Figure 4.4: Retrofitted Wall-with-opening

4.4 Experimental Data Analysis

To measure the displacement individual bricks were marked with two diagonal lines connecting the two opposite corners of the masonry unit. The point of intersection for the lines was considered the centre point of the brick and was used to mark the coordinates of each masonry unit through image processing software as shown in Figure 4.5.

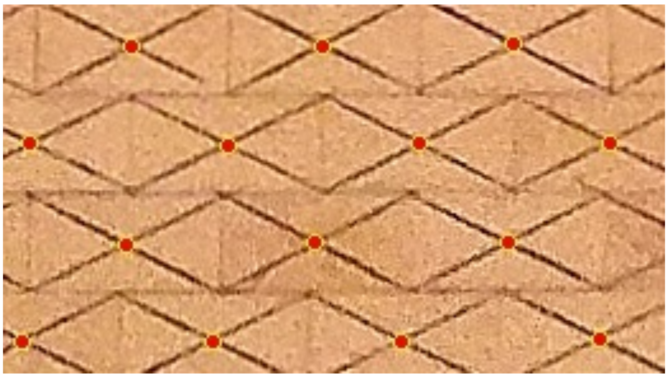


Figure 4.5: Brick with their centres marked

Image processing software called '*Digimizer'* [14] was used for finding the coordinates of the brick with respect to the origin and in accordance with the scale shown in Figure 4.2. All images taken at start and at every 1 mm of base displacement were processed to find the relative displacement of each brick from its initial position at the start of the test.

After calculating the relative displacement of each brick from its initial position at the beginning of test, '*Displacement vectors'* were plotted over the images using '*Matlab*'[104].Displacement vectors plotted with the help of Matlab enabled to visualize the direction and magnitude of motion of each masonry unit with respect to their initial positions, an example is shown in Figure 4.6. Matlab Code used for plotting displacement vectors is given in 'Appendix-B'.

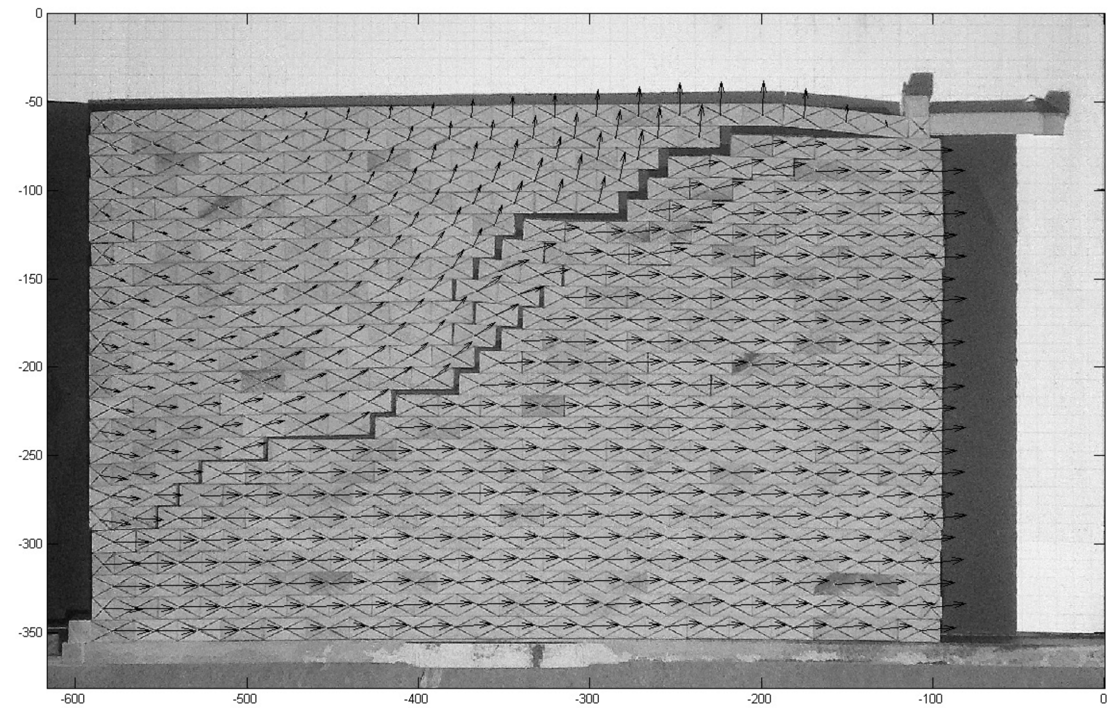


Figure 4.6: Displacement vector plot on an image

4.5 Results and Discussion

Table 4.1 gives the values of base displacement applied to the specimen walls and recorded at the time of initiation of crack and the instance of wall failure characterized by the loss of load carrying capacity of the wall. Unlike the glued specimens where the initiation of crack and wall failure are two distinct occurrences due to the presence of cohesion between masonry units, the results of non-glued walls are defined in terms of single phenomenon which is the time of crack appearance. This instance of crack appearance is due to the overcoming of initial frictional resistance between the masonry units and any further push at the base simply widens the these cracks.

Table 4.1: Results of lab experiment

Wall type	**Base displacement at crack initiation, U^o(mm)**	**Base displacement at wall failure, U^f (mm)**
Glued solid wall	4.2	5.3
Non-glued solid wall	2.1	2.1
Retrofit non-glued solid wall	4.5	4.5
Glued wall with opening	3.2	5.2
Non-glued wall with opening	2.4	2.4
Retrofit glued wall with opening	4.0	8.4

a) *Non-Glued Solid Wall*

Total of 8-specimens were tested for non-glued solid wall without retrofitting as shown in Figure A.1. For non-retrofitted and non-glued walls the cracks were mostly slip failure and the major resistance offered to the applied displacement was due to the friction between masonry layers. In some cases the crack pattern also showed diagonal shear cracks as can be seen in specimen '3' and '6'. The occurrence of crack was sudden and happened at approximately 2.1 mm of base displacement, averaged for all specimens. The pictures taken for the test show that all cracks originate from one edge of the wall and travel towards the opposite.

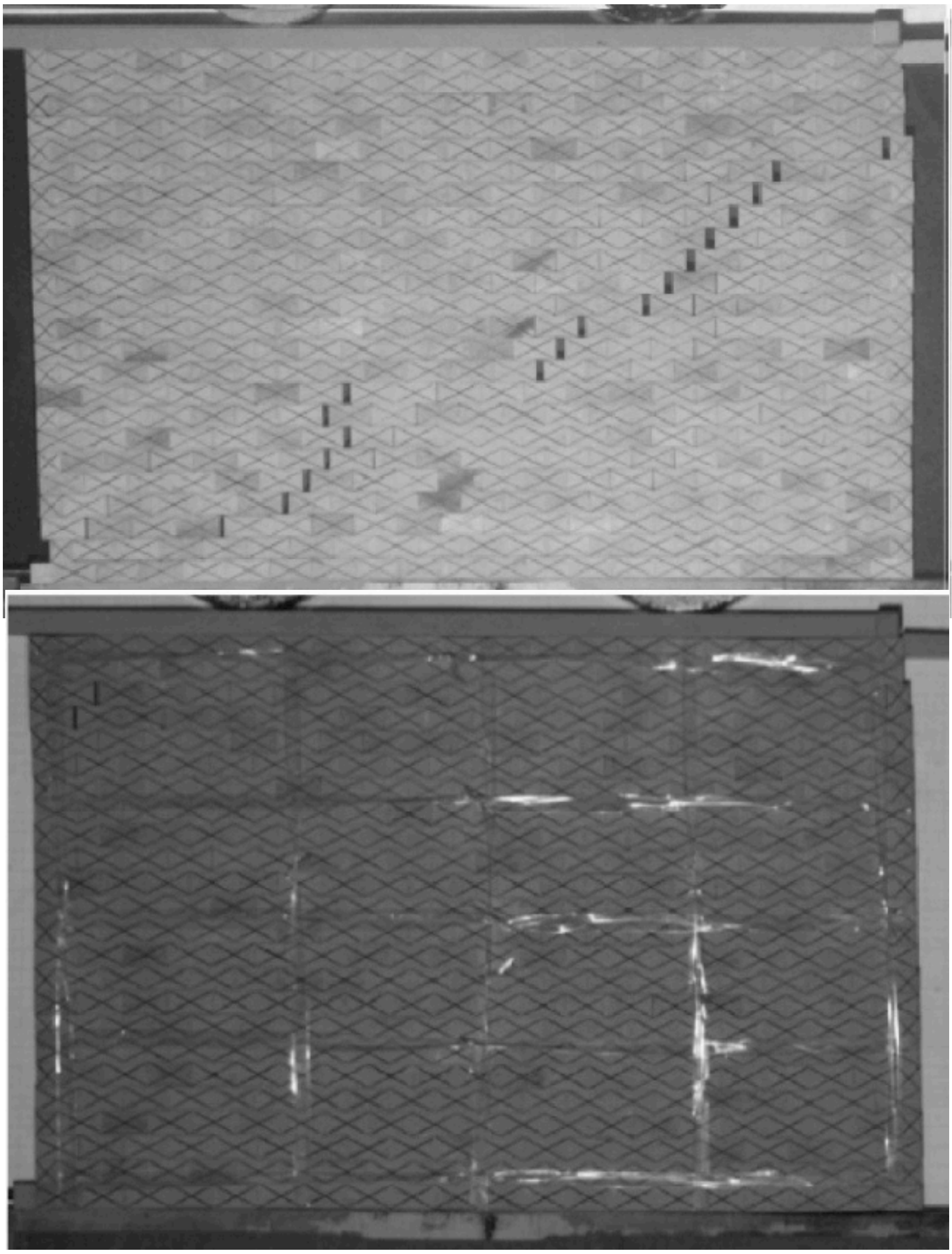


Figure 4.7: Solid wall at 7mm base displacement; non-retrofit (top), retrofit (bottom)

Four specimens were tested with retrofitting and they revealed higher integrity between brick units as compared to non-retrofitted specimens for the same values of base displacement as shown in Figure A.2. Retrofitted walls sustained load to a greater base displacement of approximately 4.5 mm, solely due to the tensile resistance offered by the retrofitting bands, and hence kept the wall intact. Figure 4.7 shows the difference in cracking observed for non-glued solid wall with and without retrofitting. The wall with retrofit showed 114% increase in the load carrying capacity of the wall.

b) *Glued Solid Wall*

For solid wall with glued interface, six specimens were tested as shown in Figure A.3. Displacement vectors were plotted only for four samples at every millimetre of base displacement to aid in studying the wall movement before and after the appearance of crack as shown from Figure A.4 to Figure A.7. The size and direction of vectors represents the magnitude and direction of the movement of the brick unit in comparison to the initial condition, respectively. However, the size of arrows shown here has been scaled up to allow visibility, especially at small displacements. The length of these vectors shows the comparative movement between individual masonry units and gives a description of the wall behaviour as a whole.

Figure 4.8 to Figure 4.11 show that under the lateral loads wall tend to rotate in counter clockwise direction before the crack actually appears but this rotation is very small. The direction of rotation is due to the displacement applied at the bottom left and restraints provided at top-right. Arrows for the bricks on left end of the wall show downward displacement and as we go towards right these displacement vectors gradually change direction towards right. If the top restraint was removed then arrows on the right would have an upward direction and the wall would be free to rotate.

The study of displacement vector (from Figure 4.8 to Figure 4.11) after crack appearance suggests that part of the wall located on the bottom right of the crack continued to move in the direction of push. Whereas, part of the wall located on left side of the crack rotates in counter-clockwise direction for specimens 2, 3 & 4, and clockwise for specimen 1. For specimen-1 the main crack is purely diagonal shear but another crack at the left edge of the wall is an in-plane flexural crack.

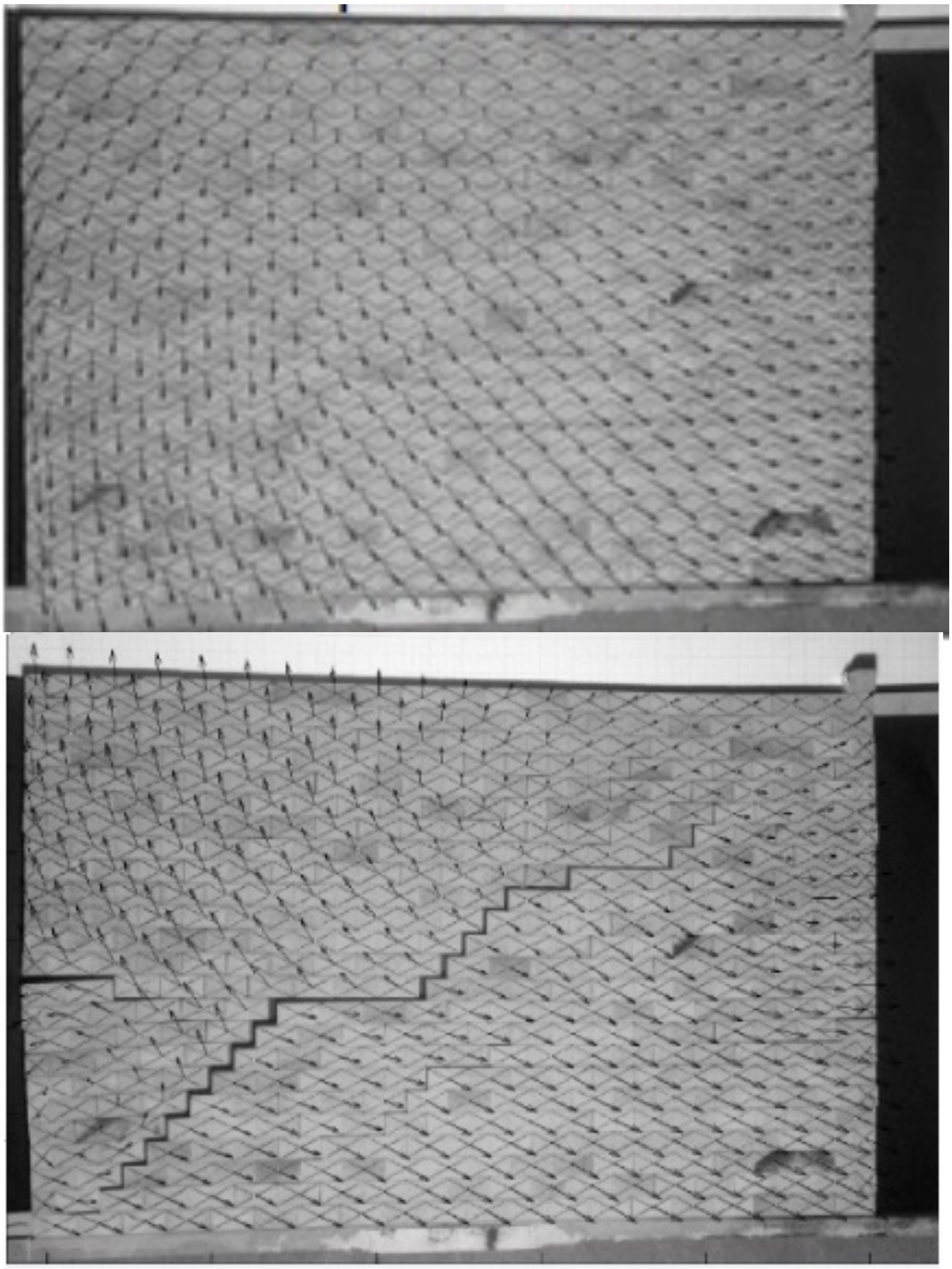


Figure 4.8: Displacement vectors for glued solid wall specimen-1: Before (top), after (bot.) cracking

Cracking in glued wall was quite brittle and categorised by two separate instances; crack initiation and loss of load carrying capacity. Cracks appeared at approximately 4.2 mm of base displacement, and the load carrying capacity of wall due to cohesion between masonry units was completely lost at 5.3 mm base displacement. Cracks were mostly shear cracks and diagonal in most cases running from top-right support to bottom-left where the displacement is applied, hence following the load transfer path direction as explained earlier in Section 2.5.

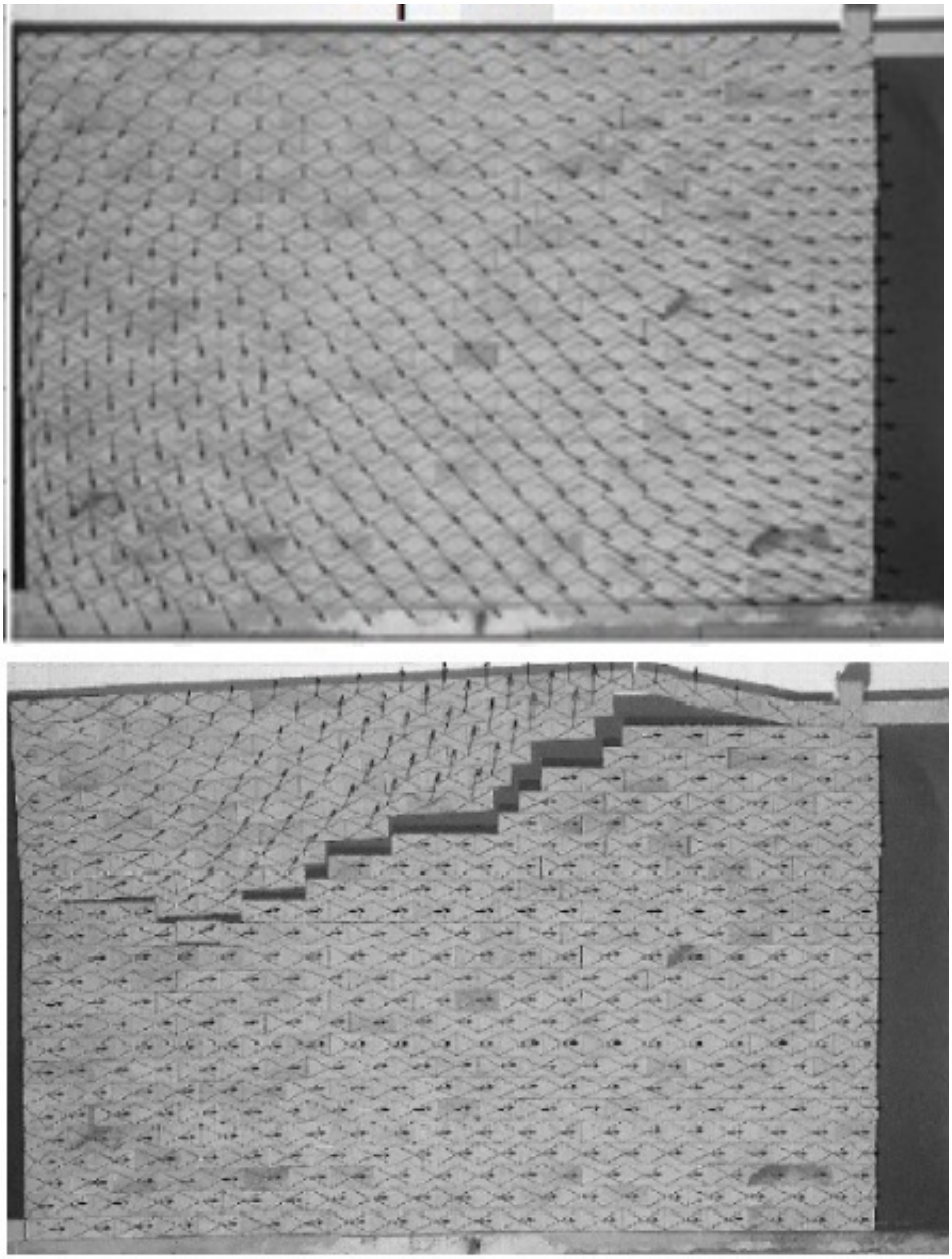


Figure 4.9: Displacement vectors for glued solid wall specimen-2: Before (top), after (bot.) cracking

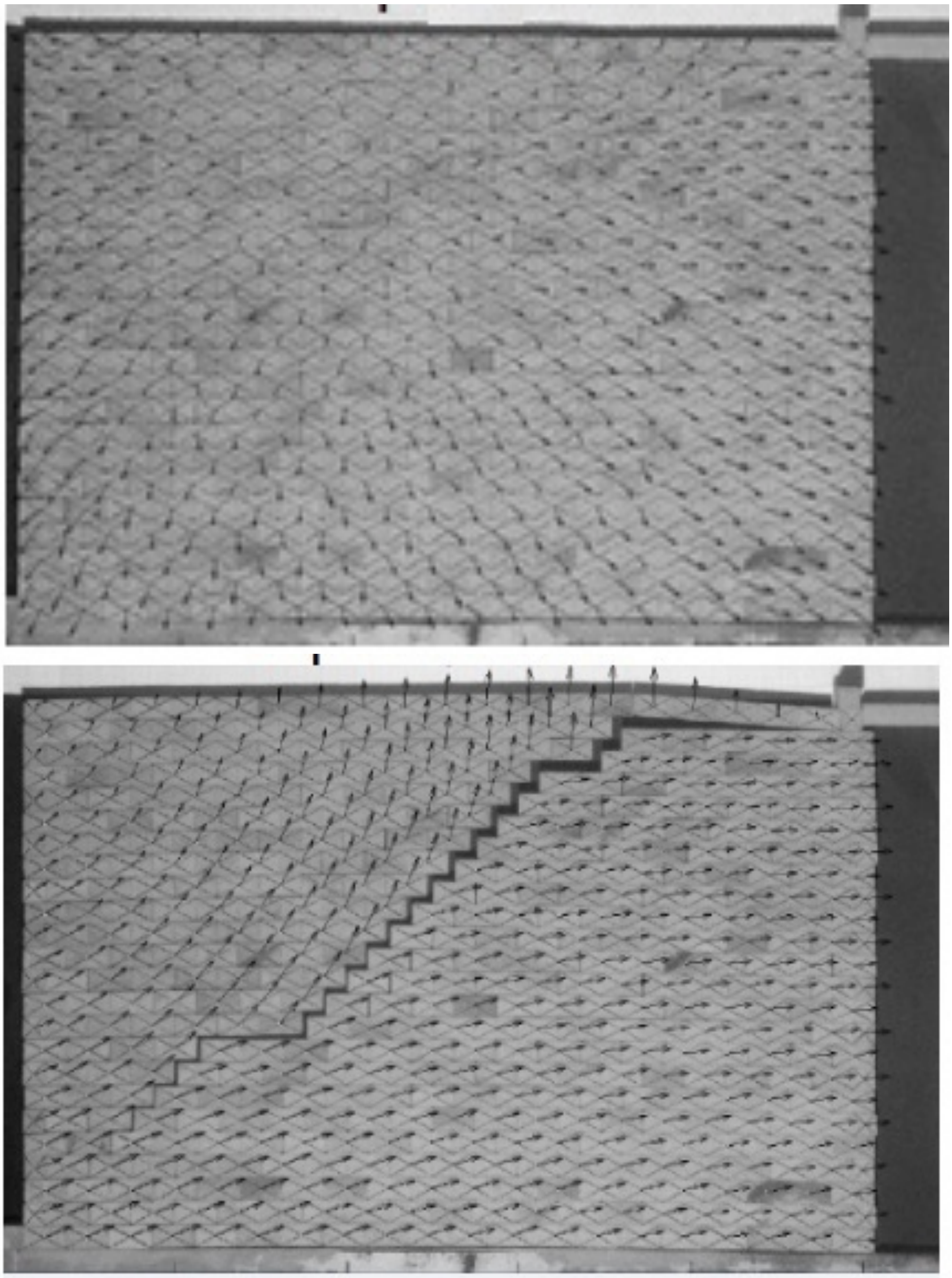


Figure 4.10: Displacement vectors for glued solid wall specimen-3: Before (top), after (bot.) cracking

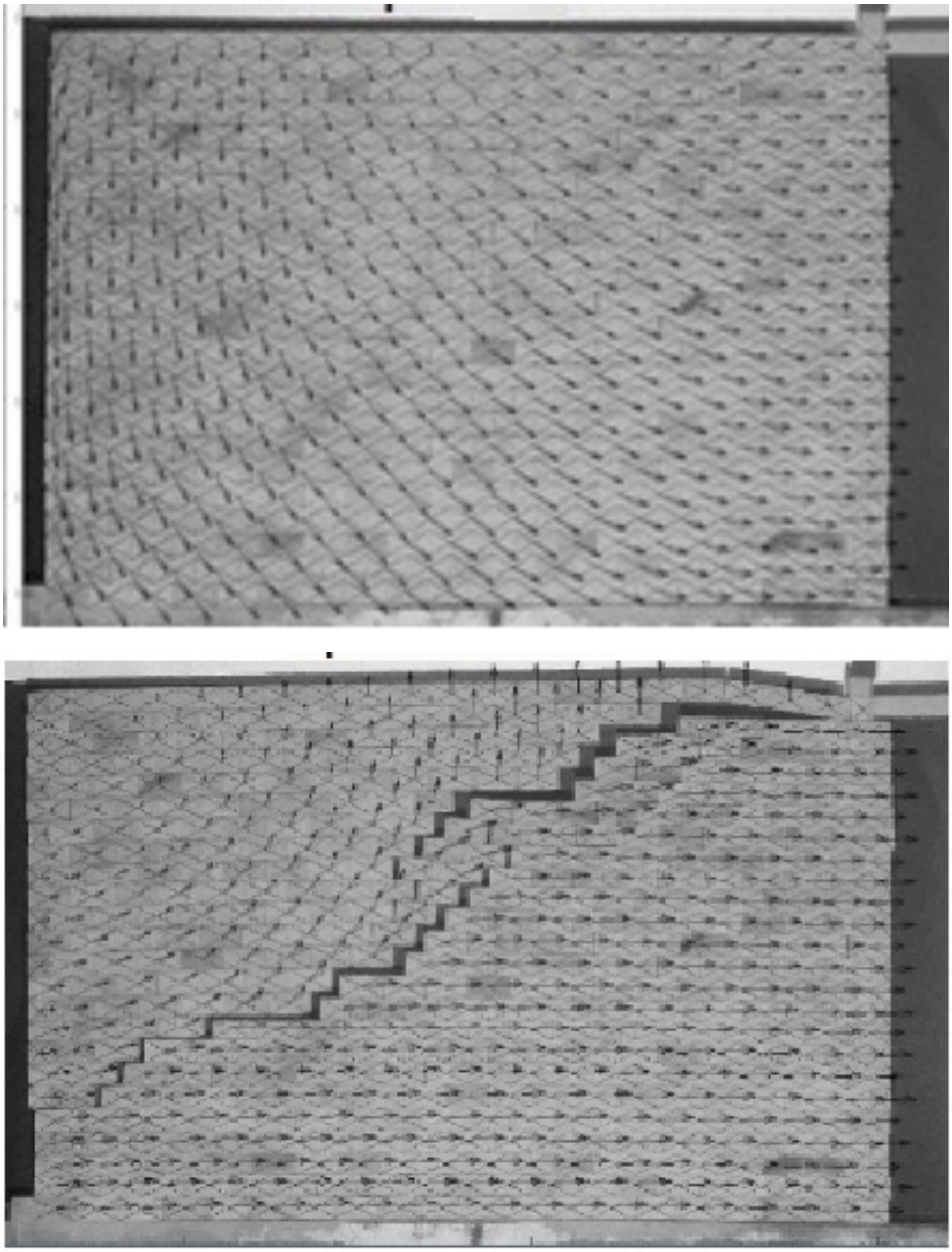


Figure 4.11: Displacement vectors for glued solid wall specimen-4: Before (top), after (bot.) cracking

c) *Non-Glued Wall-with-Opening*

Eight specimens were tested for Non-glued walls with an opening in the centre. Once again similar to the case of non-glued solid wall the resistance offered by the wall is solely frictional and categorized as a single instance of crack appearance, which in case of wall with opening was measured to be 2.4 mm. The results show diagonal cracks on both side of the opening as shown in Figure A.8. Location of these cracks is persistent throughout all specimens and they are predominantly shear cracks with similar direction as in case of solid wall i.e. from top-right to bottom-left. The crack travels from one edge of the wall to the opposite edge and due to the presence of opening in the centre the crack propagation is facilitated by the opening, providing an easy path for the crack to travel. Providing an opening size of 11.25% of the wall area increases the crack initiation (U^0) displacement value by 14%, thus proving that opening reduces wall stiffness and consequently increases its ductility. Similarly an increased value for displacement at crack initiation (U^0) and wall failure (U^f) (both instances being the same in case of non-glued specimens) is observed for wall with opening.

d) *Glued Wall-with-Opening*

Total of 6 specimens were tested for glued wall with opening and displacement vector for first four specimens are plotted. Matlab was used to plot displacement vectors of the brick for instance before and after cracking as shown from Figure 4.12 to Figure 4.15.

Under the action of initial displacement, before crack initiation wall tends to rotate before the crack actually appears and this rotation is very small. The rotation of the wall is counter clockwise as the displacement is applied from the bottom left and restricted at top right. Arrows for the bricks on left end of the wall show downward displacement and as u go towards right these displacement vectors gradually change direction towards right.

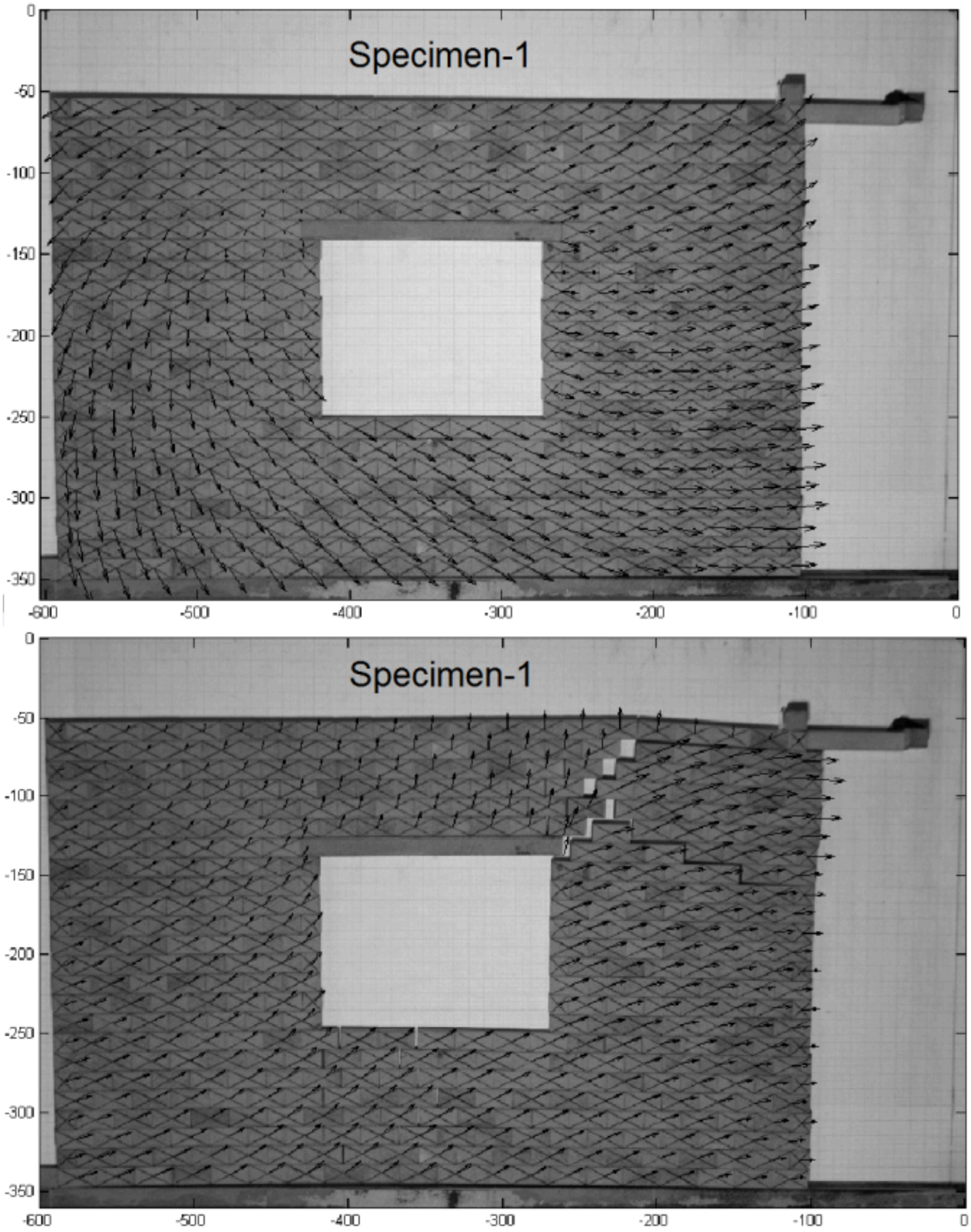


Figure 4.12: Displacement vectors for wall with opening specimen-1: Before (top), after (bot.) cracking

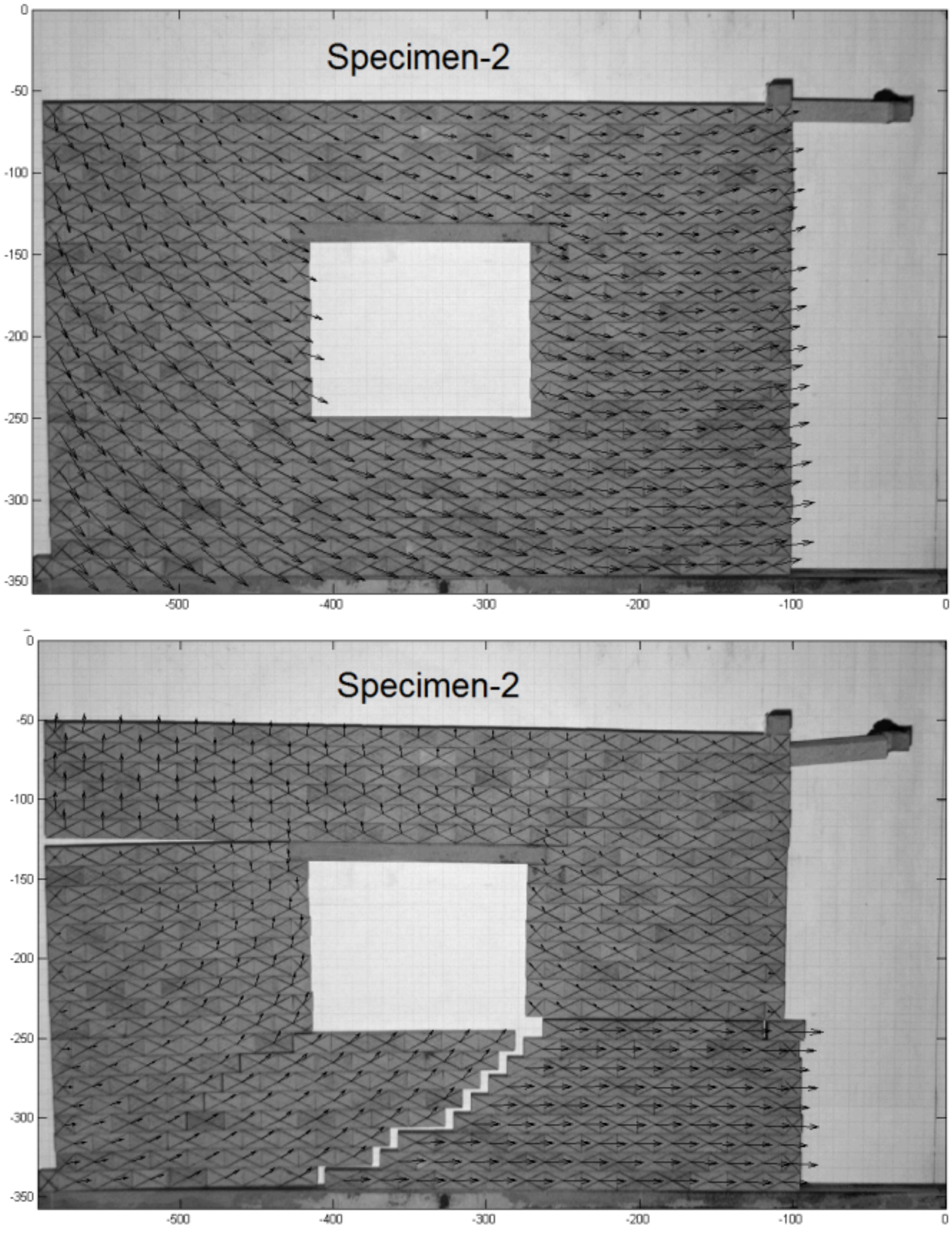


Figure 4.13: Displacement vectors for wall with opening specimen-2: Before (top), after (bot.) cracking

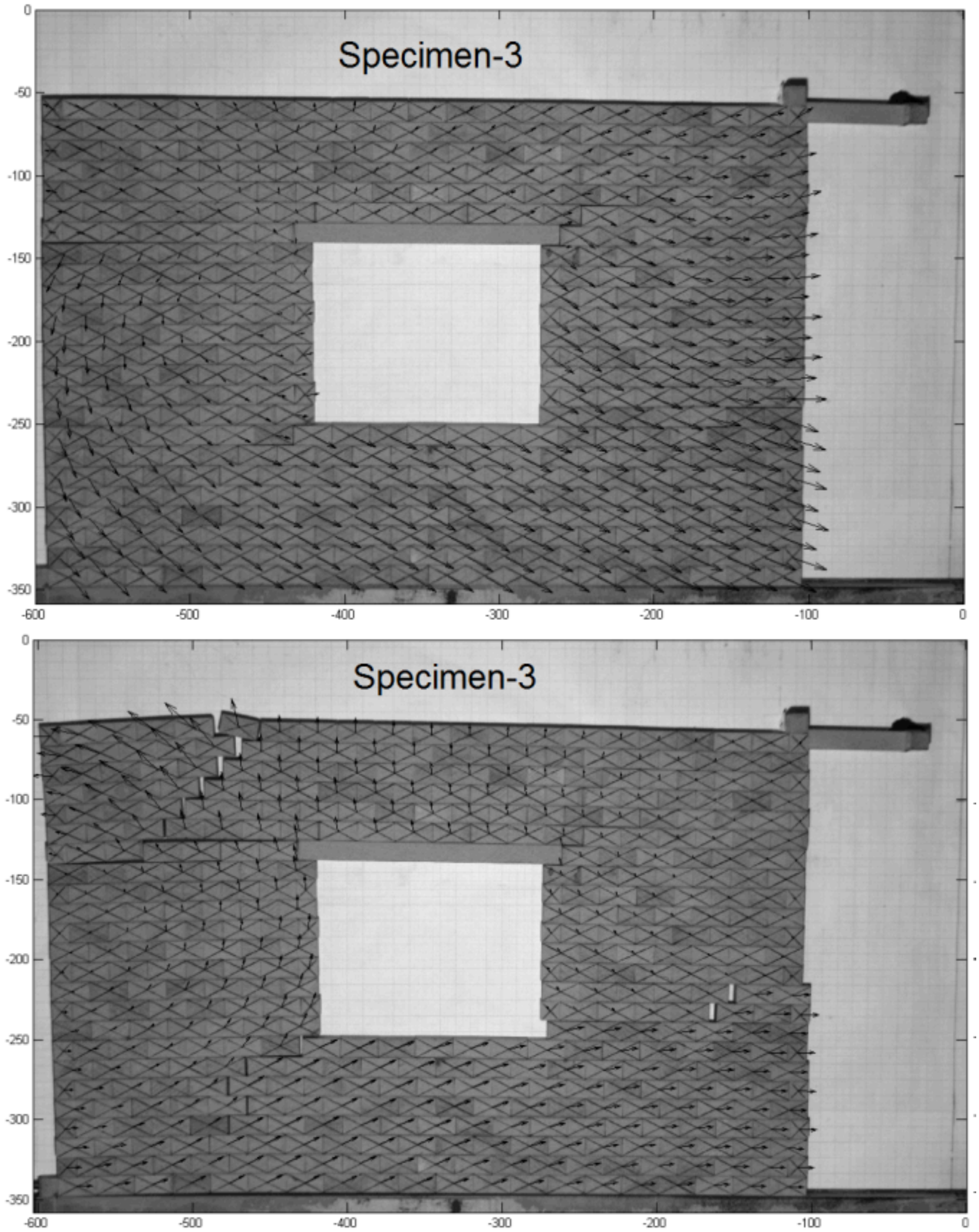


Figure 4.14: Displacement vectors for wall with opening specimen-3: Before (top), after (bot.) cracking

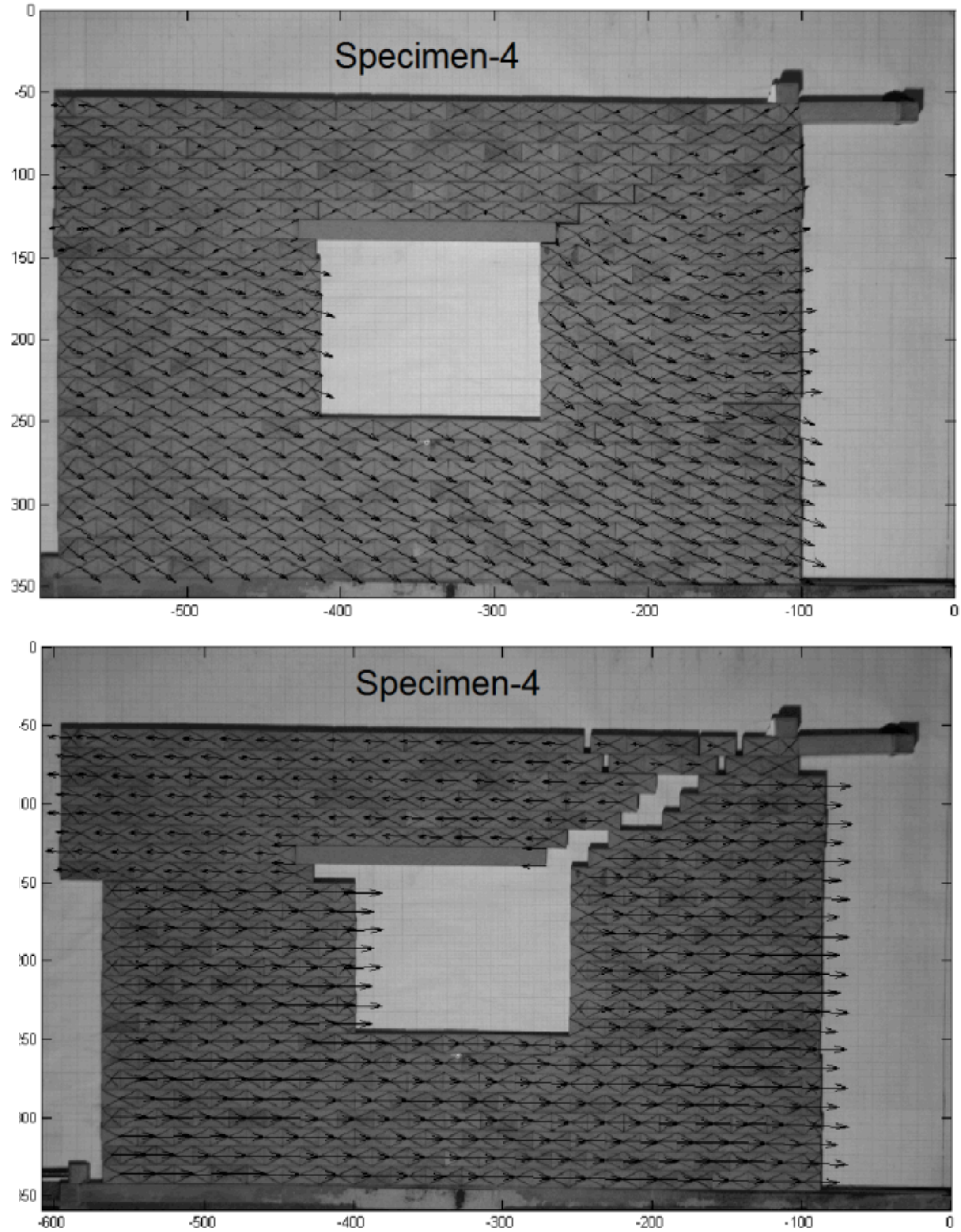


Figure 4.15: Displacement vectors for wall with opening specimen-4: Before (top), after (bot.) cracking

In case of glued wall with opening the crack initiation occurred at 3.2 mm and total loss of shear strength occurred at 5.2 mm. Providing an opening size of 11.25% of the wall area reduces the displacement at crack initiation (U^0) value by 24%. Failure pattern observed in case of glued wall with opening is a mix of diagonal shear cracks, slip failure and flexural cracks. In specimen-1 a flexural crack can be seen in the middle of the bottom edge of the window, this crack although not visible in Figure 4.12 but can easily be seen in Figure A.9 due to the background light. At the top right corner of the window another flexural crack is present along with a shear crack progressing towards right edge of the wall. Similarly, specimen-2 has a flexural crack at the top-left corner of the opening and a shear slip at the bottom-right corner of the opening along with a shear crack progressing downwards. Specimen-3 has a flexural crack at the top-left corner of the wall and a shear slip on the right of the opening. Also a flexural crack opening is observed at the bottom-left corner of the window. Specimen-4 has a shear crack in the top-right corner and a shear slip at the left side of opening. Specimen-5 & 6 are very similar in terms of their crack appearance. There are two shear cracks at the top-right and bottom-left of the wall along with a possible flexural crack opening on the right face.

All these specimens suggest that discontinuities in a wall, such as openings, are potential weaknesses in the wall. Every crack either generated from or progressed towards the opening. In other words, crack travels through an opening to reach to the other side of the wall, thus, experiencing lesser resistance in its propagation.

Five specimens were tested for retrofitted glued wall with opening (Figure A.10) and for the same amount of displacement as for non-retrofitted walls they showed lesser crack widths and higher shear strength. The cracks appeared at 4 mm of base displacement and the wall failure, categorized by development of full-length cracks, occurred at 8.4 mm. The crack patterns showed a mix of diagonal, slip and flexural cracks. It was observed that the retrofitting does not affect the crack appearance or its location. Only after the cracks have appeared the retrofitting bands come into play by prohibiting cracks from further widening and preventing complete loss of wall’s shear strength. Figure 4.16 shows the difference in cracking for single opening retrofitted and non-retrofitted wall specimen.

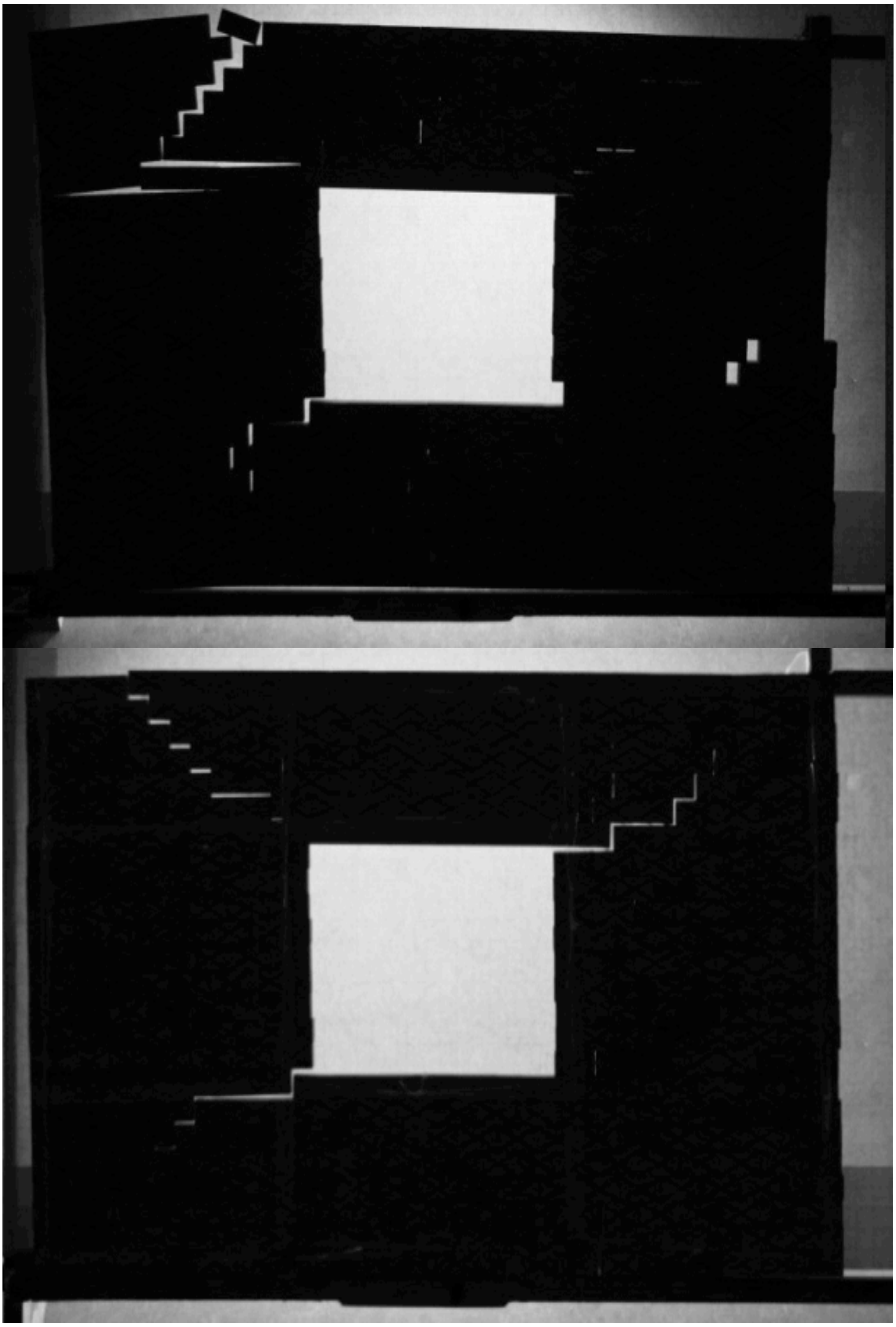


Figure 4.16: Non-retrofitted (top) and retrofitted (bot.) wall-with-opening at 6mm base displacement ***(pictures taken in dark room against a lit background for easy viewing of cracks)***

To prohibit crack widening, the retrofitting bands were only applied at locations deemed to be weak zones such as around the window edges and another set provided close to the exterior edges of the wall Figure 4.4. This was done keeping in mind that all cracks progress from window edges to the exterior edge of the wall as shown by the tests on non-retrofit walls. And with this approach it is clearly visible that cracks opening at the corners of the window are restricted by the tension in the bands and the thickness of cracks is less as compared to non-retrofitted specimens. Providing retrofit increase the U^0 by 25% and U^f by 62%, thus suggesting a higher influence of PP-bands in the post peak behaviour of masonry.

4.6 Concluding Remarks

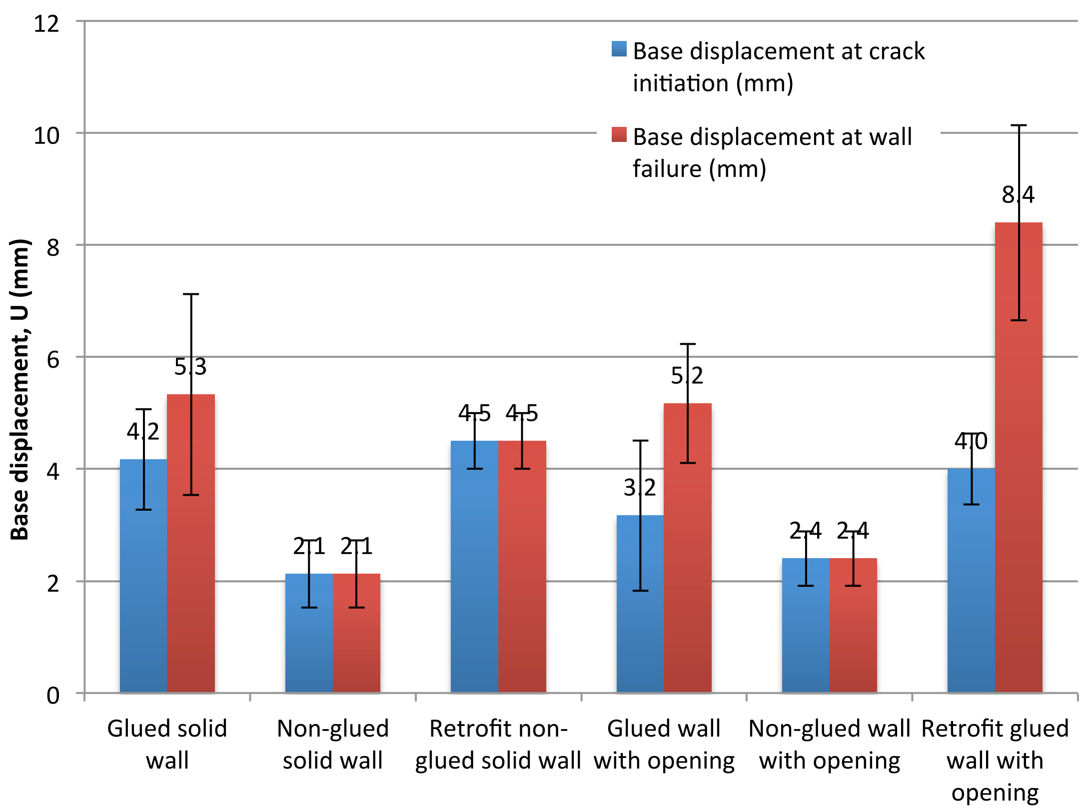


Figure 4.17: Comparison of various wall strengths in terms of base displacement

For the case of solid non-glued walls the initial strength of the retrofit wall was almost twice that of non-retrofit counterpart because of the pre-tension in the bands applied during application procedure. As the main resistance to external forces comes from friction between masonry units, which are a very weak force as in case of MDF, therefore retrofitting significantly affected the peak strength of the wall. Also, the post peak behaviour of the wall is affected by the presence of retrofit as shown in Figure 4.7 where the cracks in retrofitted wall have lesser width as compared to non-retrofitted specimen. The applied retrofitting therefore enhances the overall integrity of the wall with better peak and post peak strengths.

While in case of glued specimens with opening the initial strength of retrofit wall is increased by 25% because unlike non-glued specimen the peak strength is majorly governed by the cohesion between masonry units, which is a significant force. The post peak strength of the wall for retrofitted wall with opening had an increase of 62% as compared to its non-retrofit counterpart. Also, for the retrofit glued wall with opening the difference in the base displacement at the time of crack initiation (U^0) and failure (U^f) is higher (120%) as compared to the non-retrofit glued wall with opening, suggesting a better post peak strength behaviour and wall integrity in case of retrofit specimen.

From the bar chart shown in Figure 4.17 it can be seen that the difference in the base displacement at crack initiation and wall failure is greater for glued wall with opening as compared to glued solid wall. This suggests that due to the presence of opening the stiffness for wall with opening is lesser as compared to glued solid wall and can deform up to higher values of base displacement. However the appearance of crack in glued wall with opening occurs much earlier as compared to glued solid wall, suggesting that the glued solid wall has higher strength as compared to the one with opening, thus, reinforcing the idea that openings are source of weakness in the wall.

Thus it can be concluded with confidence that retrofit enhances the post peak-strength behaviour of masonry under continued shear loading. Even after the initiation of cracks the bands help to prevent the further widening of cracks and eventual disintegration of masonry.

Chapter 5 : Material Testing of Masonry

5.1 Introduction

Masonry as discussed earlier is a widely used historic construction material. Though the material characteristics and the manufacturing process have changed over the course of time the laying of masonry and its load transfer mechanism is essentially the same. Similarly, masonry units produced in different regions have different mechanical properties due to the locally available soil and the manufacturing practice in that region. Therefore, it was deemed necessary to test the material and workmanship used in Pakistan construction.

This chapter details the test procedure followed for material testing of masonry in Pakistan region and also lists the results obtained subsequently. The results obtained would then be used to develop FE model for masonry (Chapter-8).

5.2 Objective

The objectives for this chapter is to determine the mechanical properties of the materials used in Kashmir region to provide data for micro modelling of masonry in numerical analysis.

5.3 Methodology

For the scope of this research, material characteristics for masonry used in Kashmir region was tested in Pakistan at NED (Nadirshaw Edulji Dinshaw) University, Karachi using samples prepared by local workmen. Different types of masonry units and mortar mixes, commonly used in the local construction industry, were selected for the purpose of this study to identify their mechanical properties. The materials used are listed below:

1. Concrete blocks – cement:aggregate = 1:6 (L1)
2. Concrete blocks – cement:aggregate = 1:8 (L2)
3. Red clay brick – machine pressed (R1)

4. Red clay brick – hand pressed (R2)
5. Mortar – cement:sand = 1:4 (M1)
6. Mortar – cement:sand = 1:8 (M2)

Bricks denoted by 'R1' and 'R2' in this study are mentioned as Wire cut bricks (WCB) and Table moulded bricks (TMB) or Country burnt bricks (CBB), respectively, corresponding to the terminologies used in the literature used to compare with the results of this study. The dimensions and density of the masonry units used in this study are given in the table below. These values have been averaged over 10 samples for every masonry unit type. As the masonry units have no holes therefore they fall in the "Group-1" category of BS-EN 1996-1-1:2005 [105].

Table 5.1: Table for masonry unit dimensions and density

Masonry unit type	Length, l (mm)	Width, b (mm)	Height, h (mm)	Density, ρ kg/m^3
L1	300	100	200	2086
L2	300	100	200	2034
R1	227	108	70	1645
R2	225	110	70	1411

A series of tests were carried out to obtain the properties of masonry units and mortar, along with their subsequent effect on bonding properties. For this purpose two different mortar mixes were used in combination with the different masonry unit types to evaluate bonding properties based on the type of masonry unit and mortar combination. To obtain the material properties of masonry for FE modelling following tests were carried out:

1. Compression test for masonry unit and mortar
2. Tensile splitting test for masonry unit
3. 3-point bending test for mortar
4. Initial shear strength test for masonry joints
5. Bond wrench test for masonry joints
6. Axial tensile test of PP bands

Figure 5.1: Machine pressed brick - R1 (top); Hand pressed brick - R2 (bottom)

5.3.1 Compression test for masonry unit and mortar

a) Sample Preparation

Compression tests for masonry units were carried out according to BS EN 772-1:2011 [15] for masonry units and BS EN 1015-11:1999 [20] for mortar specimens, which requires minimum 6 specimens to be tested for each masonry unit and mortar type. The brick units were soaked in water tank overnight and then left to dry at room temperature before using them in the test. The brick units were capped and the frogs in the brick specimens were filled with cement-sand grout of ratio 1:2 to achieve sufficiently higher stiffness and strength compared to the brick unit.

Mortar used for this study was made from the Ordinary Portland Cement available in market and using the sand available in the locality. Sand used for the mortar was sieved to conform to the definition of fine aggregate of BS EN 13139:2013 [106]. Water content for the mortars was decided by the mason to replicate the actual on-ground practice where unregulated amount water is used by the mason to achieve a workable mix. This gave exact replication of the mortar actually used by masons on ground and not what is prepared in the laboratory under strict guidelines. The volume of water to cement ratio used by mason was observed to be around 1. The mortar specimens were cast in size of 160mm × 40mm × 40mm for the purpose of 3-point bending test and the two halves obtained from that test were used for compression testing. Mortar specimens were cured for 2 days in the mould and 5 days outside the moulds. After that the specimens were left to dry at room temperature to allow 28-day strengthening period before conducting the test.

b) Test Setup

Masonry units were placed in the testing machine in similar orientation, as they would be laid in actual construction and pressure was applied on the top and bottom surface as shown in Figure 5.2. Tests were carried out in strain controlled Universal Testing Machine with capacity of up to 500 kN, equipped to measure vertical load and displacement through the attached computer-based data acquisition system with measurement resolution of 0.002 mm and of 5 mN, respectively. Loading rate was regulated in terms of displacement and was maintained at 2 mm/min till the point of crushing.

In addition two samples for each masonry unit type were tested in the 1800 kN stress controlled Universal Testing Machine which gave the peak compressive strength through the dial meter attached to the machine. The brick type R1 showed strengths higher than 500 kN therefore they had to be tested in a higher load capacity machine.

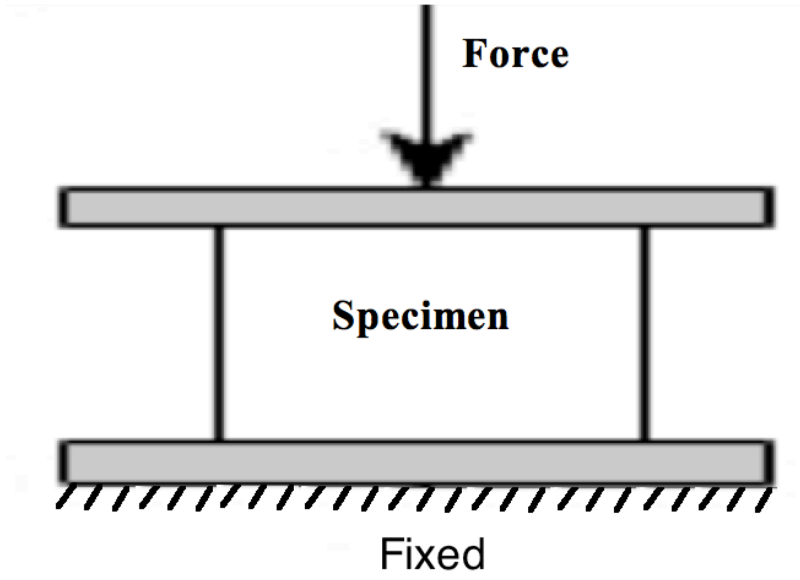


Figure 5.2: Test setup for compression testing

5.3.2 <u>Tensile splitting test for masonry unit</u>

a) Sample Preparation

Tensile splitting test for masonry units were carried out in accordance with BS EN 12390-6-2009 [16]. A total of 5 specimens were prepared for each masonry unit. The curing of bricks for tensile splitting test was carried out as described in Section 5.3.1 *Sample Preparation*.

b) Test Setup

Testing was performed in the strain controlled Universal Testing Machine specified earlier in the *Test Setup* of Section 5.3.1. The sample was laid in the same orientation as it would be laid in actual construction and a line load was applied from the top and bottom as shown in Figure 5.3. The specimen was manually held in place to prevent rotation until the loading plate came on contact with the specimen and locked it in place. The loading rate was maintained at 2 mm/min for concrete block units and 5 mm/min for red bricks. Load was applied until cracking was observed and the load-deformation curve started to fall.

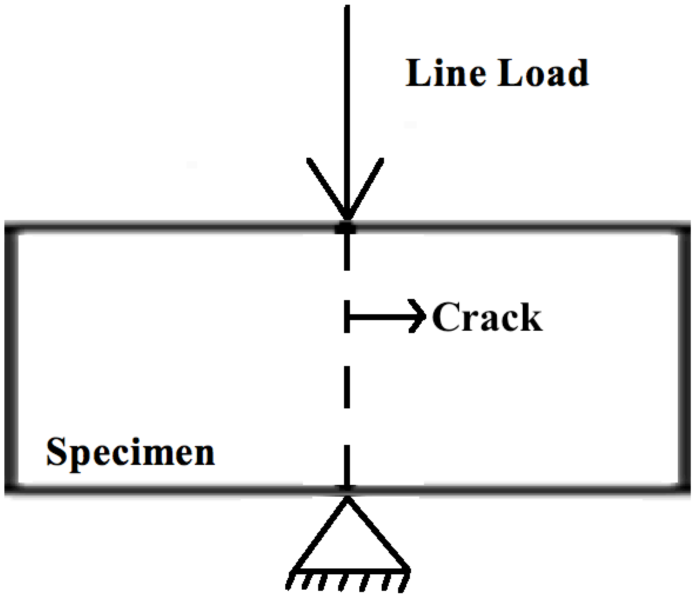


Figure 5.3: Tensile splitting test setup

5.3.3 3-point bending test for mortar

a) Sample Preparation

Test was carried out in accordance with BS EN 1015-11:1999 [20] which requires minimum of 3 samples to be tested for each mortar type. Samples were created in dimension of 160mm × 40mm × 40mm. Curing procedure for the mortar prisms is mentioned in the *Sample Preparation* of Section 5.3.1.

b) Test Setup

The mortar prism was supported on its longer dimension on either ends with two steel bars of diameter 10 mm and at a distance of 50 mm from the centre, which allowed 30 mm overhang on both sides. The load was applied at the centre of the sample with a similar round bar as used for supports (Figure 5.4). The test was performed in strain controlled Universal Testing Machine specified earlier in the *Test Setup* of Section 5.3.1. Loading rate was maintained at 1 mm/min and was applied till failure.

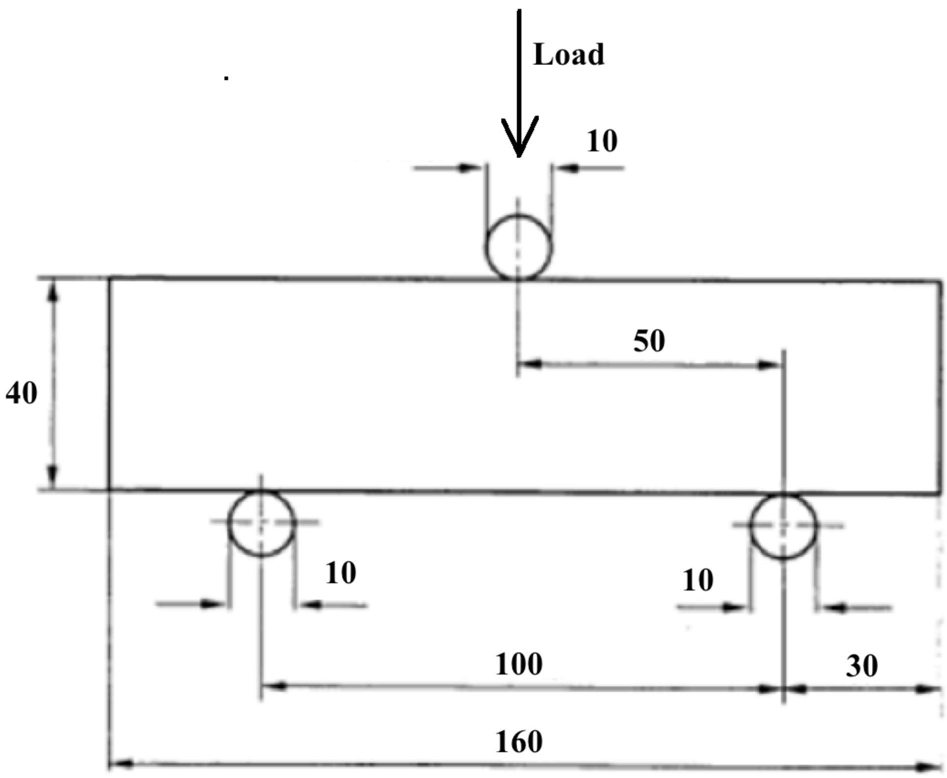


Figure 5.4: 3-point bending test setup (*dimensions in mm.*) [20]

5.3.4 Initial shear strength test for masonry joints

a) Sample Preparation

Initial shear strength test of masonry joints was carried out in accordance with BS EN 1052-3:2002 [18]. Total of 9 samples were prepared for each masonry unit in combination with each mortar type e.g. for machine pressed brick (R1), 9 samples were prepared with 1:4 mortar (M1) and 9 samples with 1:8 mortar (M2). Similar samples were prepared with other masonry unit types as well. Samples were prepared by joining three masonry units with one another on their bed surface with mortar. Approximately 10 mm thick mortar joints were used.

Immediately after preparing the specimens, pre-compression load was applied on each specimen to give a uniformly distributed vertical stress between 2 kN/mm^2 and 5 kN/mm^2. The curing for the brick units carried out prior to the preparation of test samples is similar to that mentioned earlier in the *Sample Preparation* of Section 5.3.1. To prevent the test specimens from drying out during the curing period the samples were closely covered with polyethylene sheet, and were maintained undisturbed until testing. Curing of samples was carried out for 5 days after which they were left at room conditions to dry and to achieve the 28-day period of strengthening before the test.

Capping was done for the surfaces to be in contact with the loading plates to provide flat uniform contact. Capping material used was the same as mentioned earlier in the *Sample Preparation* of Section 5.3.1.

b) Test Setup

The test was performed using strain controlled Universal Testing Machine specified earlier in the *Test Setup* of Section 5.3.1. Loading rate was maintained at 1 mm/min and the specimens were loaded until the load-deformation curve started to fall. The test was closely monitored to record the instance of first crack appearance in the joints.

The samples were placed on their header surface with the outer units supported at the bottom and load applied from top on the centre unit as shown in Figure 5.5. Three pre-compression loads were applied from the side of the specimen and three samples were tested for each pre-compression load. In case of brick specimens where the compressive strength of the brick units was 10 N/mm^2 or higher, BS-EN standards suggests pre-compression load of 0.2 N/mm^2, 0.6 N/mm^2 and 1.0 N/mm^2. In case of concrete block specimens where the compressive strength of block units was less than 10 N/mm^2, pre-compression loads of 0.1 N/mm^2, 0.3 N/mm^2 and 0.5 N/mm^2 were applied.

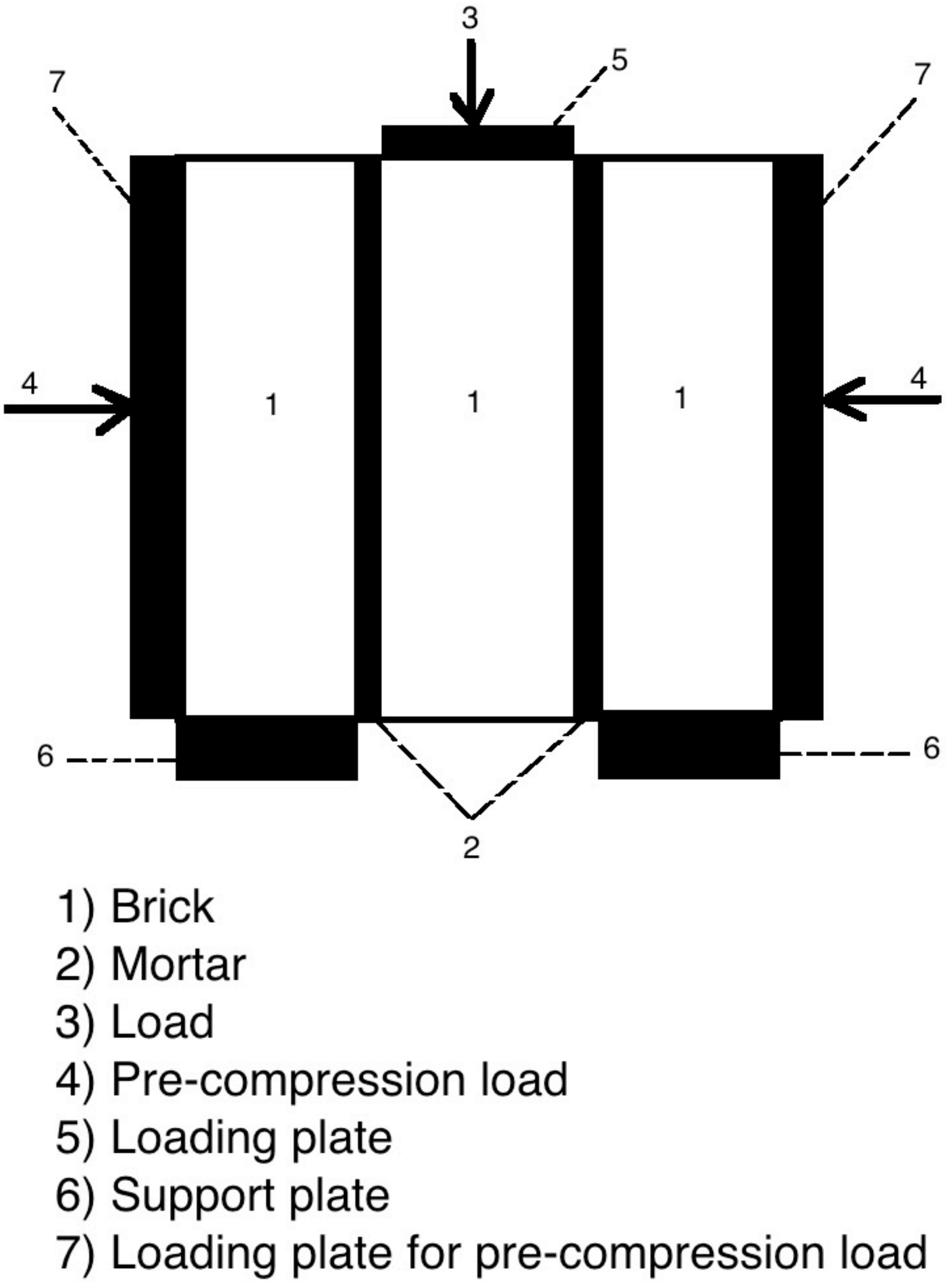


Figure 5.5: Schematic for Initial shear strength setup

5.3.5 Bond wrench test for masonry joints

a) Sample Preparation

Bond Wrench Test to determine the bond strength of masonry joints was performed in accordance with BS EN 1052-5:2005 [17]. The sample were prepared using two masonry units placed on top of one another in the orientation that corresponds to the actual laying of the brick during construction. For every masonry unit type 10 samples were prepared for 1:4 mortar (M1), and 10 samples with 1:8 mortar (M2).

Immediately after preparing the specimens, pre-compression load was applied on each specimen to give a uniformly distributed vertical stress between 2 kN/mm^2 and 5 kN/mm^2. The curing for the brick units carried out prior to the preparation of test samples is similar to that mentioned earlier in the *Sample Preparation* of Section 5.3.1. To prevent the test specimens from drying out during the curing period the samples were close covered with polyethylene sheet, and were maintained undisturbed until testing. Curing of samples was carried out for 5 days after which they were left at room conditions to dry and to achieve the 28-day strength before the test.

b) Test Setup

Bond Wrench Test is not common in Pakistan and for that reason no standard setup or equipment exist. Test setup was devised in the lab using the available resources. The specimen was rigidly held in place by clamping the bottom unit between two vertical steel plates. The clear distance from the edge of the steel plates to the joint was no less than 10 to 15 mm specified by the BS-EN standards. The top masonry unit was also clamped between two vertical steel plates with a minimum 10 to 15 mm clear distance from the joint to be tested. A lever arm was attached to one of the plates protruding out at right angles to the face of the masonry unit. The actual arrangement of the test setup is shown in Figure 5.16.

Load applied at the free end of the lever arm to induce bending moment in the joint. The application of load was done manually and the point at which the joints failed was recorded as the strength for the joint. The weight of the top unit and any adhering mortar was measured and used in calculating the bond strength as prescribed by BS-EN standard.

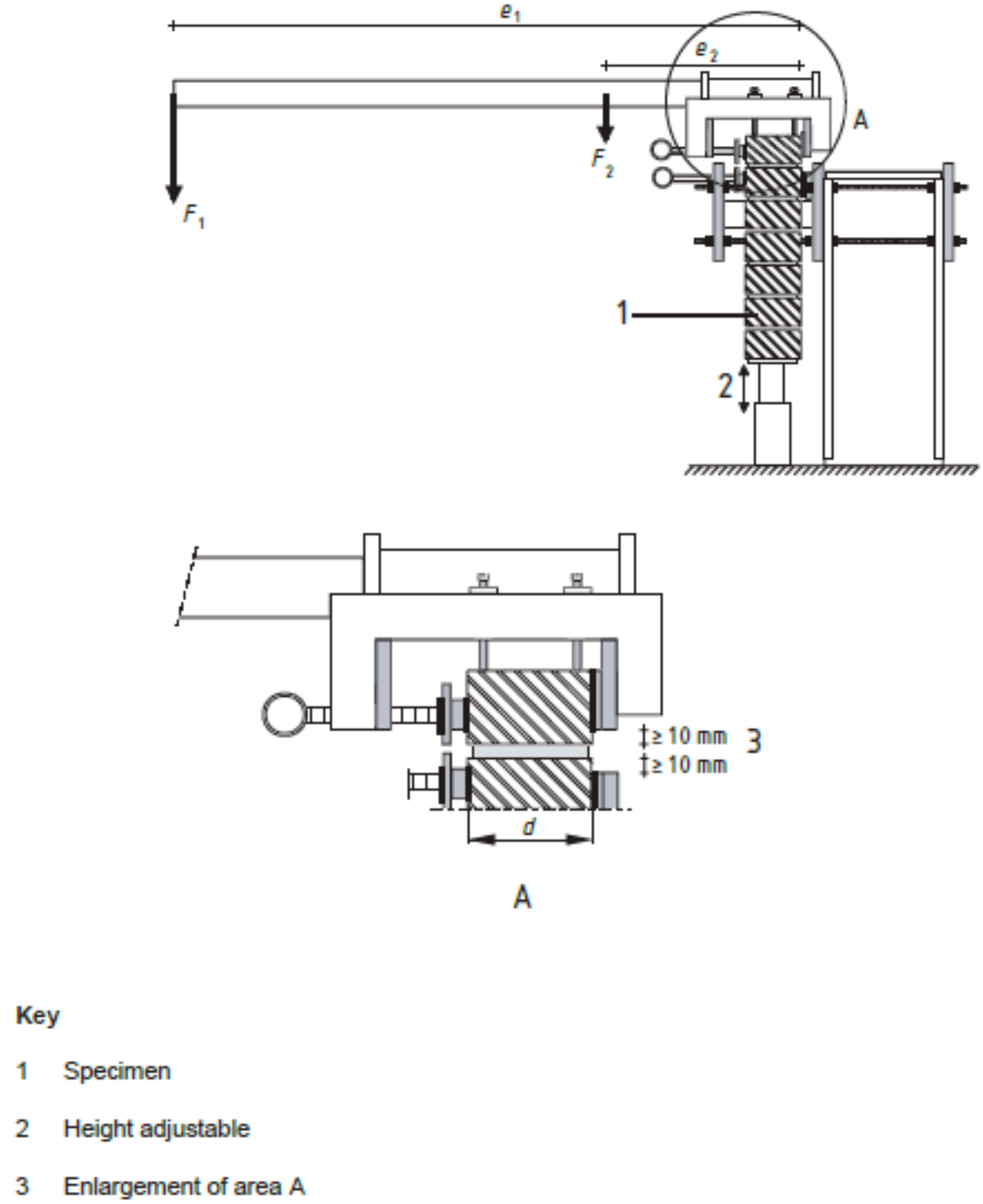


Figure 5.6: Bond wrench test setup according to BS EN 1052-5:2005 [17]

5.3.6 Axial Tensile Test for PP-bands

a) Sample Preparation

To test the tensile strength of PP-bands used in the subsequent shake table tests of retrofitted masonry BS-EN 2747:1998 [107] was consulted. The standard specifies specimen length of 150 mm, however due to irregularities in the geometry of the bands, three different band lengths were tested i.e. 150 mm, 300 mm and

450 mm. The aim was to identify the effect of specimen length on result outcome for strength and elasticity. For each length type three specimens were tested in tension.

Having established the influence of specimen length on the results, a second set of tests on similar specimens were conducted but this time markers were placed at regular intervals on the band (as shown in Figure 5.7) to monitor the local elongation in each segment using images taken at every 5 mm of displacement of loading clamps.

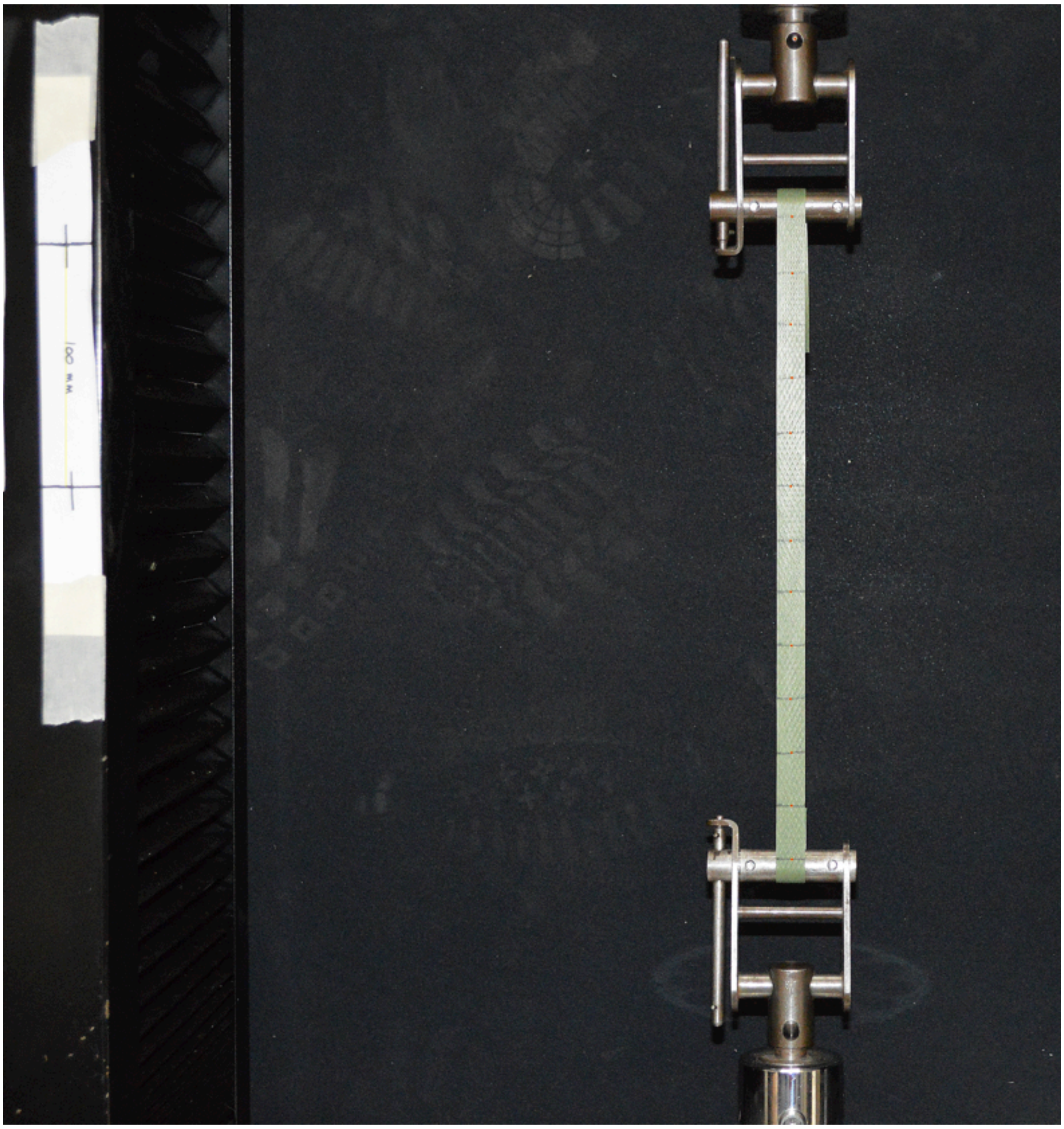


Figure 5.7: Test setup for axial tensile of PP-bands

b) Test Setup

The test was performed using strain controlled Universal Testing Machine with capacity of up to 500 kN which recorded vertical load and displacement through the attached computer-based data acquisition system. Loading rate specified in BS-EN 2747:1998 is 2±0.2 mm/min [107], which took a long time till failure therefore test speed of 10 mm/min was used and the specimens were loaded until failure. The clamps were specially designed to hold the bands without any slipping and without damaging them (Figure 5.7).

5.4 Results

5.4.1 Compression Test for Masonry unit and Mortar

List of Compressive strength tests performed are as follows:

- 6 x Concrete Aggregate Blocks – 1:6 (L1)
- 6 x Concrete Aggregate Blocks – 1:8 (L2)
- 6 x Red Bricks – machine pressed (R1)
- 6 x Red Bricks – hand pressed (R2)
- 6 x Mortar – 1:4 (M1)
- 6 x Mortar – 1:8 (M2)

To obtain the compressive strength of the specimen the maximum load carried by the specimen was divided by the area of the specimen perpendicular to the loading direction. Compressive strength of all the samples tested for every specimen type was averaged and given in the Table 5.2.

Table 5.2 show red bricks to have higher strengths as compared to concrete blocks. R1 has 3.5 times higher strength than L1, and R2 is approximately 3 times higher. The strength value of L2 corresponds with M2 due to the same approximate ratio of cement to sand/aggregate. M1 has higher strength compared to L1 due to higher cement ratio.

The load-deformation graphs of the tests were used to plot stress-strain curves for the purpose of finding out the modulus of elasticity. The initial slope of the stress-strain curve up to 25-30 % of the ultimate strength was used to determine the modulus of elasticity. Table 5.2 below shows the stress strain curves for 1:8 mortar (M2) samples used to estimate the modulus of elasticity.

Table 5.2: Table of Compressive strength of masonry units and mortar with COV

Specimen Type	Compressive Strength, f_c (MPa)	Coeff. Of Variation, COV (%)
L1	5.00	12
L2	2.79	14
R1	17.7	16
R2	9.42	21
M1	9.74	8
M2	2.77	7

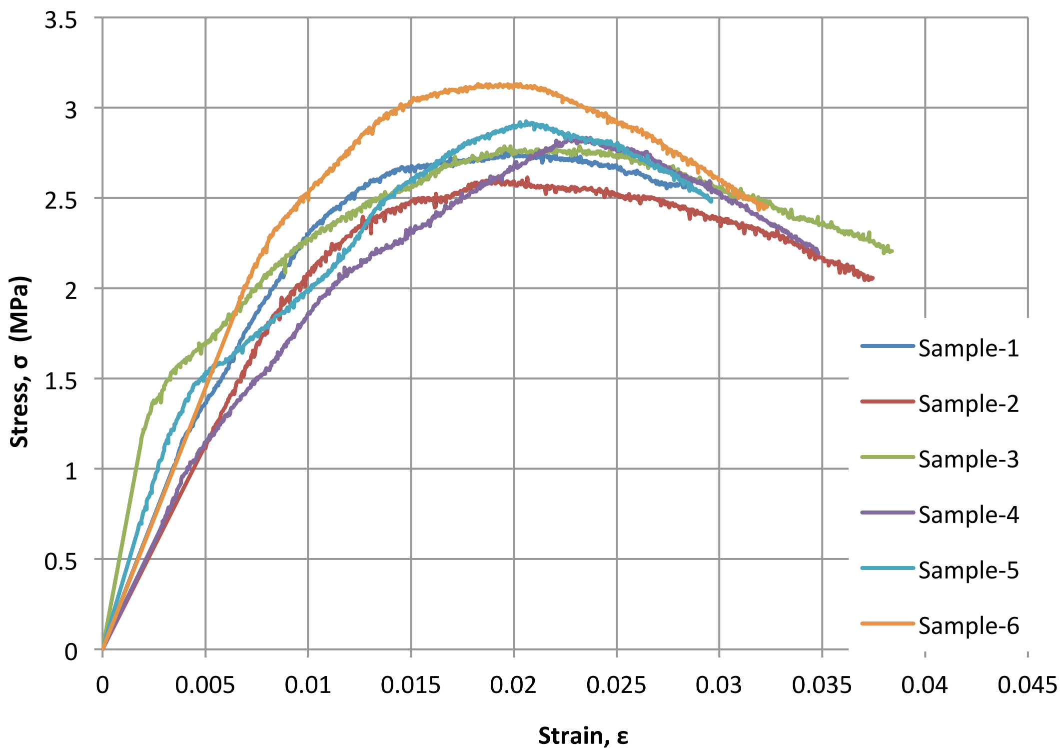


Figure 5.8: Stress-strain graphs for 6 samples of M2 mortar tested in compression

Figure 5.9: Compression Test for Concrete Block (*top*); Red Brick (*centre*); mortar (*bottom*)

The modulus of Elasticity obtained from compressive test on different types of Masonry units and mortar is given in the Table 5.3. M1 mortar has the highest modulus of elasticity in comparison to the concrete blocks due to higher cement content. The modulus of elasticity of clay bricks suggests softer material compared to mortar even with higher compressive strengths.

Table 5.3: Table for Modulus of elasticity of masonry units and mortar with COV

Specimen Type	Modulus of Elastictity, E (MPa)	Coeff. Of Variation, COV (%)
L1	533	21
L2	229	20
R1	429	19
R2	239	9
M1	707	47
M2	340	39

Table 5.4 gives a review of compressive strength and modulus of elasticity for masonry units and mortar reported in various literatures. The mechanical properties of masonry differ geographically, bricks from one manufacturer differ from the other based on raw material and process. Therefore the comparison above mentions the countries where the bricks were made and tested. According to Narayanan et al. [108], bricks in Kollam (India) have very low compressive strength and modulus of elasticity. The mortar mixes tested by Narayanan et al. [108] are close to the strengths observed in the current study but with very high modulus of elasticity. Whereas, those manufactured and used in developed countries have much higher strength and elasticity [108]. Wire cut bricks tested by Ali et al. [109] gives close strength as compared to R1 bricks of this study. However the mortar strengths for 1:4 and 1:8 are approximately 40-50% greater than the findings of this study.

Table 5.4: Compressive strength and elasticity from tests and literatures

Author and Country	Sample Description	Compressive Strength, f_c (MPa)	Modulus of Elasticity, E (MPa)
Narayanan and Sirajuddin [108] (Kollam)	WCB-1	4.64	166.7
	WCB -2	6.18	133
	CBB	2.19	66.7
	1:4	10.1	2650
	1:6	6.7	2000
	1:8	2.35	1167
	Western Bricks	15-150	3500-34000
	Australian Bricks	n/a	7000-12000
	Pressed red clay bricks	n/a	14000
	Concrete blocks	n/a	14000
Ali et al. [109] (Pakistan)	WCB	16.91	n/a
	1:4	17	n/a
	1:8	6	n/a
Kaushik et al. [110] (India)	WCB	20.8	6095
	1:3	20.6	3750
	1:6	3.1	545
Alecci et al. [111] (Italy)	WCB	17	n/a
	1:4	8.33	n/a
Gumaste et al. [112] (India)	WCB	23	3372
	TMB	5.7	976
	1:6	6.6	8568
Khoso et al. [113] (Pakistan)	Larkana Bricks	11.49	20230
Radovanovic et al. [114] (Montenegro)	Concrete block	3.26	n/a
Current study (Pakistan)	L1	5.00	533
	L2	2.79	229
	R1	17.7	429
	R2	9.42	239
	M1	9.74	707
	M2	2.77	340

Italian bricks, suggested by Alecci et al. [111], have similar strength as R1 of this study and the 1:4 mortar strengths in both the studies are in close proximity. Study carried out by Khoso et al. [113] to determine the properties of bricks found in Larkana region of Pakistan, gives an average strength of wire cut brick and table moulded brick as 11.49 MPa which is in between the strengths of R1 and R2 found in the curernt study, but with high elastic modulus. Concrete masonry units tested by Radovanovic et al. [114] also confirms with the concrete block strength from experiments. It should be noted that M1 and M2 of this study corresponds to the M7.5 and M2.5 mortar strength class of BS-EN 1052-1:1999, respectively [19].

5.4.2 Tensile Splitting Test for Masonry Units

List of Tensile splitting tests performed are listed below:

- 5 x Concrete Aggregate Blocks – 1:6 (L1)
- 5 x Concrete Aggregate Blocks – 1:8 (L2)
- 5 x Red Bricks – machine pressed (R1)
- 5 x Red Bricks – hand pressed (R2)

To determine the tensile strength of masonry units, tensile splitting test was carried out to determine the ultimate load at failure. The computerized data acquisition system attached to the machine generated load-deformation curves for the test. Equation used to calculate the splitting tensile strength of masonry units is given by BS EN 12390-6-2009 [16] for hardened concrete specimen:

$$\boldsymbol{f_t = \frac{2F}{\pi h b}} \qquad \textit{Eq. 5.1}$$

where, f_t, splitting tensile strength

F, force applied to the specimen

h, height of specimen

b, width of the specimen

Figure 5.10: Tensile splitting test for concrete block (*top*); red brick (*bottom*)

The tensile strengths of different masonry units obtained after using the equation 5.1 and averaging over the number of samples tested for each masonry type are given in Table 5.5. Tensile strength of red brick samples is higher than concrete masonry units. R1 tensile strength is twice more than that of L1and similar relation is observed in case of R2 and L2. This shows that red bricks available locally have higher strengths as compared to concrete blocks but with lesser modulus of elasticity suggesting that bricks are softer compared to concrete blocks.

Table 5.5: Table of Tensile strength of Masonry units with COV.

Type	Tensile Strength, f_t (MPa)	Coeff. Of Variation, COV (%)
L1	0.69	4
L2	0.48	15
R1	1.62	32
R2	1.06	33

5.4.3 3-Point Bending Test of Mortar for Flexural Strength

List of 3-point bending test performed on mortar are listed below:

- 3 x Mortar – 1:4 (M1)
- 3 x Mortar – 1:8 (M2)

To calculate the flexure strength of mortar prism from the load values obtained through tests BS EN 1015-11-1999 [20] suggests the use of following equation:

$$f_f = 1.5\frac{Fl}{bh^2} \qquad Eq.\ 5.2$$

where f_f, flexural strength

F, applied load on the specimen

l, length of the specimen

b, width of the specimen

h, height of the specimen

The averaged flexure strength of specimens tested for M1 and M2 are shown in Table 5.6.

Figure 5.11: 3-Point bending test for mortar

Flexure strength of both mortar mixes determined through tests are greater the tensile strength of clay bricks and the concrete blocks of this study. Flexural strength normally coincides with the tensile strength for homogeneous material with no defects on outer fibres. For non-homogenous materials, flexural strengths are usually higher than the tensile strength. This is due to the defects present within the material, which under pure tension might yield lower tensile strength. Hence, the flexure strength of mortar specimen are higher than the concrete blocks for the same cement content such as M2 and L2 where the cement to sand/aggregate ratio is 1:8.

Coefficient of variation for flexural strength of M1 is 11.8% and that of M2 is 11.2%. Values for flexural strength obtained in this study are presented in the Table 5.6 along with literature findings.

Flexural strength of M1 used in this study has slightly higher compressive and flexural strength as compared to the findings of Alecci et al. [111]. Percentage increase in compressive and flexural strength is 16.9% and 30%, respectively. While the strength of M2 in comparison to the Zimmerman et al. [115] results show 22.6% lower compressive strength and 86% higher flexural strength.

Table 5.6: Flexural strength of mortar obtained from tests and literature

Author and Country	Mortar Description	Compressive Strength, f_c MPa	Flexural Strength, f_f MPa
Zimmerman et al. (Vienna) [115]	Mix 2	3.58	1.02
Alecci et al. (Italy) [111]	1:4	8.33	2.63
This Study (Pakistan)	M1	9.74	3.42
	M2	2.77	1.90

5.4.4 Initial Shear Strength Test for Masonry Joints

List of initial shear strength test performed for various combinations of masonry unit and mortar are as follows:

- 9 x L1M1 – (Block type L1 used with mortar type M1)
- 9 x L1M2 – (Block type L1 used with mortar type M2)
- 9 x L2M1 – (Block type L2 used with mortar type M1)
- 9 x L2M2 – (Block type L2 used with mortar type M2)
- 9 x R1M1 – (Brick type R1 used with mortar type M1)
- 9 x R1M2 – (Brick type R1 used with mortar type M2)
- 9 x R2M1 – (Brick type R2 used with mortar type M1)
- 9 x R2M2 – (Brick type R2 used with mortar type M2)

Figure 5.12: Initial shear strength test for concrete blocks

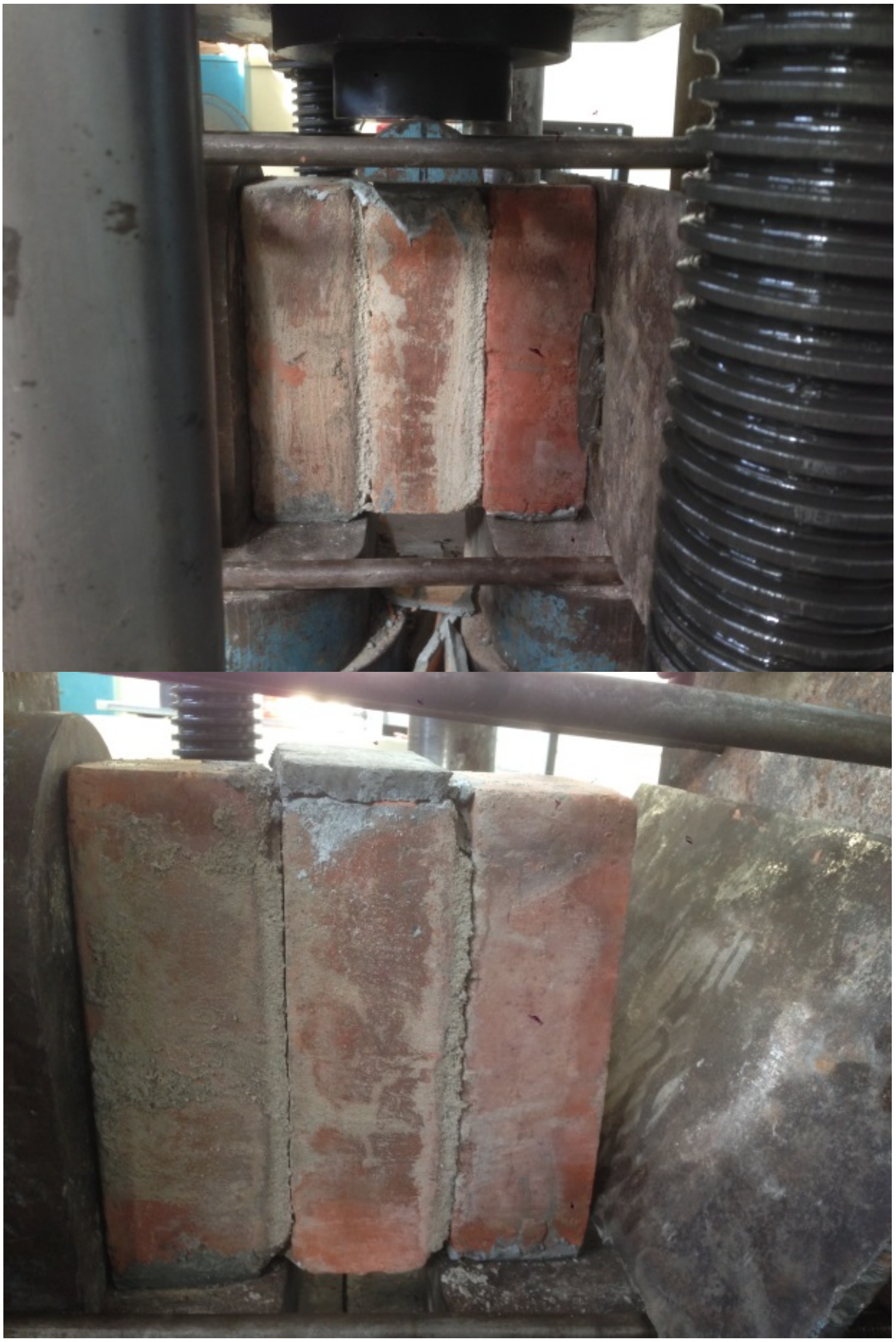


Figure 5.13: Initial shear strength test for red bricks

For determining the initial shear strength of masonry, the ultimate load is divided by the area of the middle brick's surface in contact with the adjacent one. As there are two contact surfaces of the middle brick, one on either side, therefore the surface area taken should be a sum of both sides. Once the individual shear strengths have been calculated for all three pre-compression loads, a graph is plotted as the one shown in Figure 5.14 for L1M1.

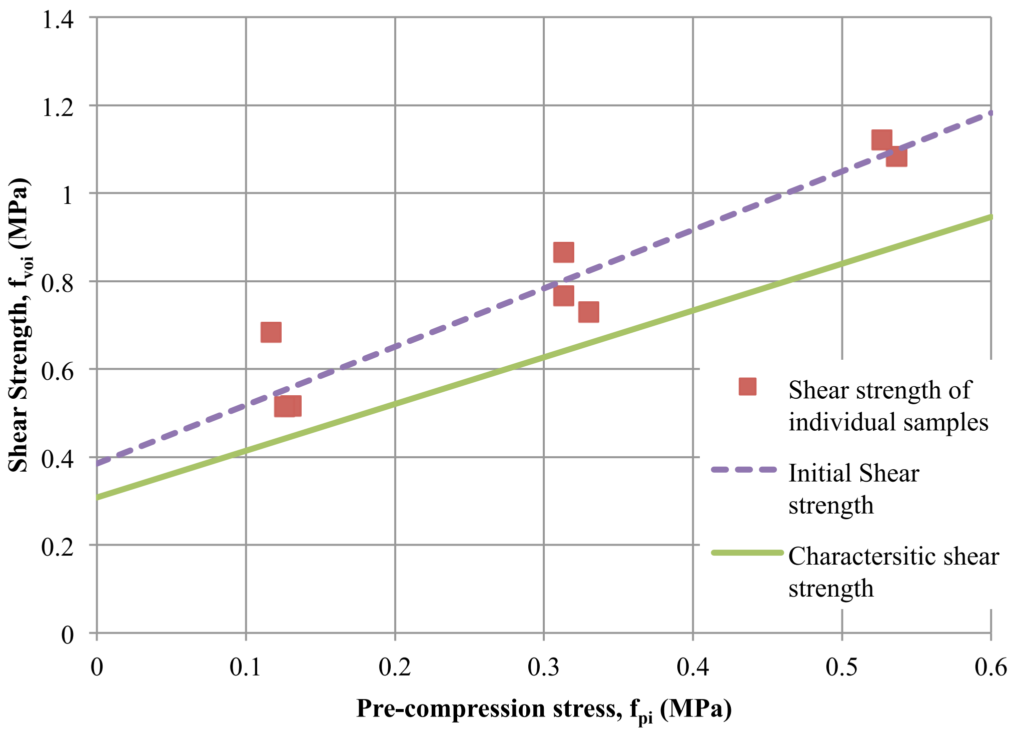


Figure 5.14: Shear strength vs. Pre-compression load of L1M1 samples

Three samples were tested for L1M1 for each pre-compression load of approximately 0.1, 0.3 and 0.5 MPa (Figure 5.14). A best-fit straight line is plotted through these points to intersect the y-axis, shown as dotted line in Figure 5.14. This y-intercept gives the value of shear strength at zero pre-compression loads. The slope of the dotted line gives the internal friction coefficient whose tangent inverse gives the internal angle of friction. A factor of 0.8 as prescribed by BS EN 1052-3:2002 [18] is multiplied by shear strength to obtained characteristic shear strength, shown as solid line in Figure 5.14, and subsequently the characteristic friction coefficient and angle is worked out from the slope of solid line.

$$f_{vok} = 0.8 \times f_{vo} \qquad \textit{Eq. 5.3}$$

where, f_{vok} is characteristic shear strength

f_{vo} is initial shear strength

In total 72 specimens (9 for each masonry type) were tested for initial shear strength and the findings of these tests are given in

Table 5.7.

Table 5.7: Initial shear strength and friction coefficient from Initial shear strength test results

Type	Initial Shear Strength, f_{vo} (MPa)	Characteri-stic Strength, $f_{vo\,k}$ (MPa)	Angle of internal friction, α (deg.)	Characteri-stic angle of internal friction, α_k (deg.)	Coeff. of Friction μ	Charact-eristic Friction coeff. μ_k
L1M1	0.38	0.31	53.1	46.7	1.33	1.06
L1M2	0.11	0.09	46.0	39.6	1.03	0.83
L2M1	0.37	0.3	56.6	50.5	1.52	1.21
L2M2	0.11	0.09	48.9	42.6	1.15	0.92
R1M1	0.6	0.36	17.9	14.5	0.32	0.26
R1M2	0.16	0.12	27.4	22.5	0.52	0.41
R2M1	0.16	0.13	41.2	35.0	0.87	0.7
R2M2	0.10	0.08	40.6	34.5	0.86	0.69

It should be noted that R2 brick specimens created with M1 mortar have 200-250% higher shear strengths as compared to their M2 counterparts. However, in case of R2 bricks the increase in strength of M1 sample is only 65% of M2. R2 bricks have rough surface and are not well compacted as compared to R1 bricks and the failure of specimen is governed by frictional resistance rather than the shear strength of interface. Having rough surface allows the brick to grip mortar well, but due to the poor compactness of the R2 brick, failure observed was not entirely interface based. The high bond strength of M1 mortar scraped off some material from the low strength and weakly compacted R2 brick surface as shown in Figure 5.15, where thin red film of clay is visible on the mortar layer after the failure of joints during shear strength tests of R2M1 samples. This loosening of the brick material caused the samples of R2M1 have lesser shear strength as compared to other specimens of M1 mortar.

L1 and L2 samples yield close values of shear strength for both mortar types. This is because the shear strength values are more governed by the friction between masonry units and mortar rather than the individual strengths of the material, and as L1 and L2 masonry specimens have similar surface finish, which is evident in the close values of friction coefficients (Table 5.7) therefore, they yield close values for shear strengths.

Table 5.8 provides a comparison of angle of internal friction for different masonry types. Here however, it can be observed that R2 brick samples have significantly higher friction as compared to R1 samples. The reason is the rough surface of R2 bricks that gives higher frictional resistance to sliding in comparison to the smooth surface of R1 bricks. For concrete block specimens the friction angles of M1 samples are approximately 15% higher in comparison to M2 samples. This is due to the higher bonding strength of M1 as compared to M2. L2 samples yield slightly higher friction angles (approximately 6%) than the L1 samples because of the lower modulus of elasticity of L2 masonry units that makes them softer and able to deform and stay intact for higher deformations in comparison to L1. Contrary to concrete block samples, R1 samples yield higher friction angle with M2 rather than M1. This is because the surface of R1 bricks is smooth and plain which makes it difficult to produce friction in case of M1. However, in case of M2 due to the higher content of sand granules present, the friction angle is increased by 52%.

Figure 5.15: R2M1 samples after Initial shear strength test

A comparison of R1 bricks samples with similar brick types studied by other researchers (Table 5.8) show that higher cement ratio in the mortar yields higher shear strength, but not necessarily higher friction coefficient. Sand granules present in the mortar are a major contributing factor towards frictional resistance of the specimen and thus higher sand content yields higher value for friction. Shear strength of 1:4 mortar samples from Ali et al. [109] gives 50% less strength as compared to the 1:4 samples of current study but are in good agreement with friction coefficient values. Alecci et al. [111] gives the strength for 1:4 mortar samples (0.531 MPa) which is only 19% higher than the findings of this study. For 1:8 mortar samples Ali et al. [109] obtained shear strength value (0.12 MPa) which is 22% lesser than the findings of this study, but his friction coefficient is higher by 41% and fairly close to the findings of Zimmerman et al. (0.709) [115]. However, the shear strength given by Zimmerman et al. [115] is 29% higher than the results of this study.

Table 5.8: Initial shear strength and friction coefficient obtained from tests and literature

Study and Country of brick	Sample	Compressive strength, f_c MPa	Mean Shear Strength, f_{vo} MPa	Characteristic Shear strength, f_{vok} MPa	Friction coeff. µ
Ali et al. (Pakistan) [109]	WCB	16.91	n/a	n/a	
	1:4	17	0.22	n/a	0.32
	1:8	6	0.12	n/a	0.73
Alecci et al. (Italy) [111]	WCB	17	n/a	n/a	n/a
	1:4	8.33	0.531	n/a	n/a
Zimmerman et al. (Austria) [115]	Old Solid Clay brick	19.28	n/a	n/a	n/a
	Mix-2 (1:8)	3.58	0.21	0.168	0.709
	Mix-4	0.22	0.027	0.01	0.643
Current Study (Pakistan)	R1	17.7			
	M1	9.74	0.44	0.36	0.32
	M2	2.77	0.16	0.12	0.52

Stiffness of the joints has been worked out using the slope of load-displacement graphs from the initial shear strength test results. The stiffness values were obtained to provide data for the cohesive joint modelling in finite element analysis. The slope of load-displacement graph was divided by the area of the joint and by the number of joints in each specimen i.e. 2. This was done to obtain the value of joint stiffness in terms of $N/m/m^2$ as required in Abaqus. Table 5.9 lists the values for joint stiffness for various masonry types tested for current study.

Table 5.9: Joint Stiffness for Finite Element modelling

Type	Stiffness, K_{ss} (N/m)	Stiffness, K_{ss} ($N/m/m^2$)
L1M1	3.81E+07	6.35E+08
L1M2	2.17E+07	3.61E+08
L2M1	2.48E+07	4.14E+08
L2M2	1.89E+07	3.14E+08
R1M1	4.25E+07	8.67E+08
R1M2	2.50E+07	5.10E+08
R2M1	3.38E+07	6.82E+08
R2M2	2.42E+07	4.90E+08

Comparison of joint stiffness reveals that samples prepared using M1 (1:4) mortar have higher stiffness as compared to the M2 (1:8) samples. Stiffness of joints does not solely depend on the mortar properties but also on masonry unit properties. Comparison between the R1 and R2 samples show that samples using stronger and stiffer bricks have higher joint stiffness and it holds true when comparing L1 samples to L2 samples. This means that joint stiffness is a combined effect of the mortar and masonry unit properties. The difference in joint stiffness between samples using M2 mortar is lesser as compared to samples using M1 mortar. For instance, the percentage difference in joint stiffness from R1M2 to R2M2 is only 4% whereas; difference from R1M1 to R2M1 is 21%. The difference in modulus of elasticity of masonry unit and mortar for R1M2 and R2M2 samples is 88 MPa and 100 MPa, respectively. While for R1M1 and R2M1 samples it is 278 and 468, respectively. Hence the R1M2 and R2M2 samples have lesser difference in the joint stiffness as compare to those using M2 mortar. Same phenomena holds true in case of concrete block masonry samples.

5.4.5 Bond Wrench Test for Masonry Joints

List of Bond Wrench tests performed to assess the bond strength of various masonry types is given below:

- 5 x L1M1 – (Block type L1 used with mortar type M1)
- 5 x L1M2 – (Block type L1 used with mortar type M2)
- 5 x L2M1 – (Block type L2 used with mortar type M1)
- 5 x L2M2 – (Block type L2 used with mortar type M2)
- 5 x R1M1 – (Brick type R1 used with mortar type M1)
- 5 x R1M2 – (Brick type R1 used with mortar type M2)
- 5 x R2M1 – (Brick type R2 used with mortar type M1)
- 5 x R2M2 – (Brick type R2 used with mortar type M2)

BS EN 1052-5:2005 [17] requires a total of 10 specimens to be tested for each masonry unit and mortar type, but due to the wastage of samples during handling and testing, only 5-specimens could be tested for each type of masonry-mortar combination. Accordingly, the factor 'k' which code specifies for calculating the characteristic bond strength from 10 samples was modified due to the lesser number of samples tested for each combination. To calculate the bond strength by bond wrench method BS EN 1052-5:2005 [17] specifies the following equation:

$$f_{wi} = \frac{F_1 e_1 + F_2 e_2 - \frac{2}{3} d \left(F_1 + F_2 + \frac{W}{4}\right)}{Z} \qquad \textbf{\textit{Eq. 5.4}}$$

$$\textbf{\textit{Where, }} Z = \frac{bd^2}{6}$$

where , f_{wi} is the bond strength of *i*th sample in MPa

F_1, the applied load in N

F_2, weight of bond wrench in N

W, weight of the masonry unit pulled off the specimen and any adherent mortar in N

e_1, distance from the applied load to the tension face in mm

e_2, distance from the centre of gravity to the lower and upper clamp from the tension face of the specimen in mm

b, mean width of the bed joint in mm

d, mean depth of the specimen in mm

Table 5.10: Bond Strength from Bond Wrench Test results

Type	Bond Strength, f_w (MPa)	Characteristic Strength, f_{wk} (MPa)	Coeff. Of Variation, COV (%)
L1M1	0.61	0.26	29
L1M2	0.10	0.06	101
L2M1	0.38	0.10	69
L2M2	0.18	0.08	85
R1M1	0.17	0.06	107
R1M2	0.036	0.02	148
R2M1	0.10	0.03	181
R2M2	0.067	0.04	133

Figure 5.16: Bond Wrench test setup

Test results Table 5.10 shows that bond strength increases with higher cement content in the mortar, therefore, specimens with M1 mortar have higher bond strength as compared to M2 mortar specimens. The bond strength of L1M2 is lesser compared to L2M2 because of the greater difference in modulus of elasticity in L1M2 samples which produces weaker bond as compared to L2M2. Similar phenomenon is observed in R1M2 and R2M2 samples. In case of M1 samples the bond strength of R1 and L1 are higher than R2 and L2 samples, respectively. Here again the reason is the difference in the elastic modulus of masonry unit and mortar, which is lesser in case of R1M1 and L1M1 as compared to R2M1 and L2M1. This shows that higher the difference in the elastic modulus of masonry unit and mortar weaker would be the bond. Furthermore, the bond strength of concrete block samples is higher than those of red brick samples due to the fact that concrete blocks have rougher texture that grips mortar more strongly as compared to bricks.

BS-EN 1996-1-1:2005 [105] prescribes a minimum value for characteristic flexural strength of concrete block masonry along the bed joint using M2.5 and M9 mortar as 0.05 MPa and 0.1 MPa, respectively. These values seem to be satisfied only by the concrete block samples using M1 (or M7.5 as per Eurocode classification) and M2 (or M2.5 as per Eurocode classification) mortar. For clay brick masonry specimens code prescribes a minimum value for characteristic flexural strength along the bed joint using M2.5 and M9 mortar as 0.1 MPa, which is higher than the findings of this study, which shows the inadequacy of bonding properties of brick units manufactures locally.

The bond strength found in the literature is higher as compared to the tests carried out for current study (Table 5.11). This is because the bricks and mortar used in western countries have higher strengths as compared to those used in developing countries where no set regulation and specification for brick manufacturing process exist. The higher strength of literature values is due to the difference in test methods used for assessing masonry tensile strength. Mortar used in current study had unmonitored water content to replicate the common practice of site and not subject it to strict guidelines of laboratory preparation, which does not portray an accurate picture of actual site condition for non-engineered masonry houses.

Table 5.11: Bond Strength obtained from tests and literature

Author		Sample description	Test type	Bond strength, f_w (MPa)	COV. (%)
Fouad M. Khalaf (2005) [1]	Present Investigation	Clay solid wire cut facing with 1:1:6 mortar	Z-shaped	0.35	21
	fkx, BS 5628 (British 1992)	Clay with 1:1:6 mortar (absorption<6%)	Wallette failure parallel to bed joints	0.5	n/a
		Clay with 1:1:6 mortar (6%<absorption<12%)	Wallette failure parallel to bed joints	0.4	n/a
		Clay with 1:1:6 mortar (absorption>12%)	Wallette failure parallel to bed joints	0.3	n/a
	De Vekey et al. (1990)	Clay with 1:1:6 mortar	crossed couplet	0.28	35
		Clay with 1:1:6 mortar	wallette	0.33	48
	Riddington and Jukes (1994)	Clay solid wire cut facing with 1:1:6 mortar	Direct tension	1.04	16
		Clay solid wire cut facing with 1:1:6 mortar	Brick stack parallel	0.77	18
Ahad Ullah et al. 2013 (Bangladesh) [116]		WCB with 1:4 mortar	n/a	0.0328	n/a
Current Study (Pakistan)		R1M1	Bond Wrench	0.17	107
		R1M2	Bond Wrench	0.04	148
		R2M1	Bond Wrench	0.10	181
		R2M2	Bond Wrench	0.07	133

5.4.6 Axial Tensile Test for PP-bands

Tensile strength was calculated from the load-displacement graph generated by the tensile testing machine. Ultimate load values were divided by the cross sectional area of PP-band, which was measured to be 14mm x 0.6mm, and averaged over the number of specimen tested. Secant modulus of elasticity for the polypropylene bands were calculated using the initial stress strain curve up to 25% of ultimate strength (Figure 5.18), employing the same technique used for the calculation of modulus of elasticity of masonry units and mortar specimens.

Table 5.12 gives the result for tensile test of PP-bands. The mean tensile strength for all the tested specimens is 111 MPa with a coefficient of variation around 15%. The mean modulus of elasticity for the bands is 1.75 GPa with 18% coefficient of variation. The length of specimen affected the tensile strength and modulus of elasticity values for PP-bands. Increasing the specimen length gave lower strength and higher elasticity.

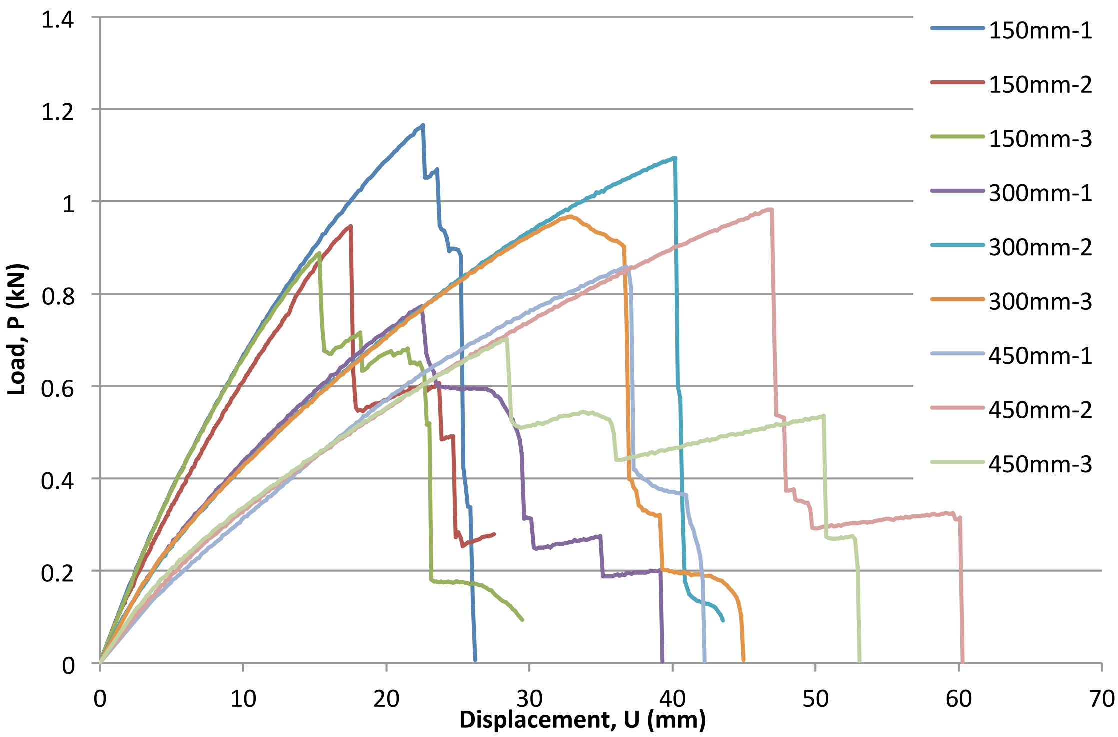


Figure 5.17: Load-displacement graph for tensile test of PP-bands

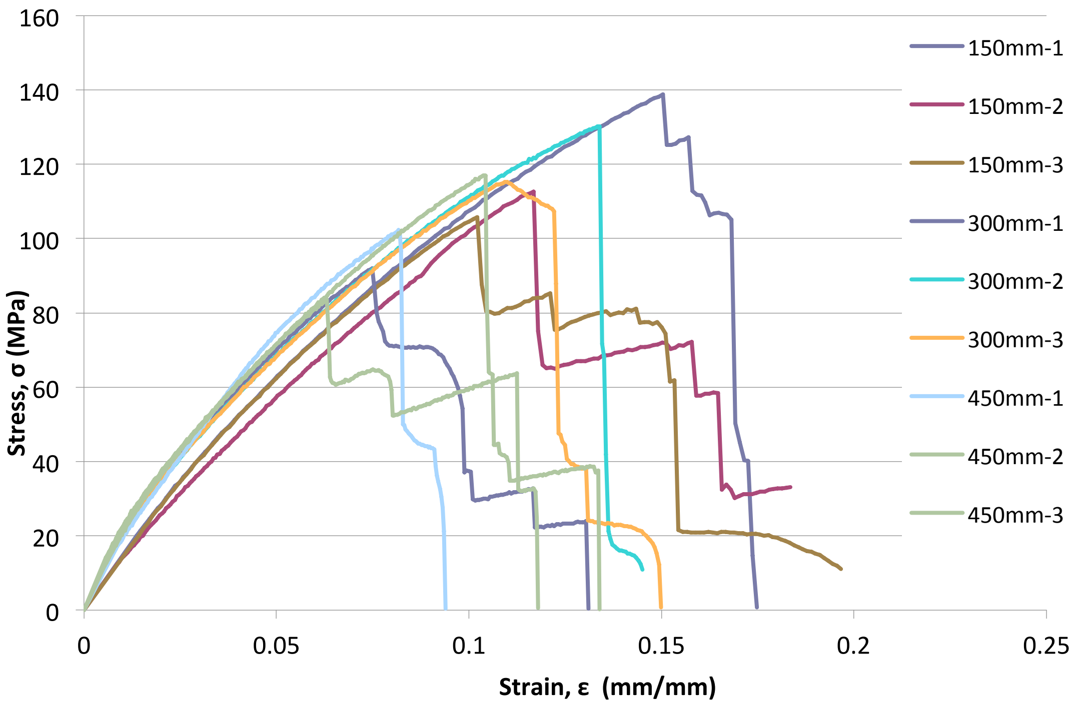


Figure 5.18: Stress-strain graph for tensile test of PP-bands

Similar tests carried by Sathiparan et al. [64] records a bilinear elastic behaviour for PP-bands with 3.2 GPa and 1 GPa as initial and residual modulus of elasticity, respectively. The cross section for the bands used by Sathiparan et al. is 15.5mm x 0.6mm [64].

Table 5.12: Tensile strength and Modulus of Elasticity for PP-bands

Strip Length	Tensile Strength, f_{ti} (MPa)	Mean Tensile Strength, f_t (MPa)	Modulus of Elasticity, E_i (MPa)	Mean Modulus of Elasticity, E (MPa)
150 mm	139	119	1386	1358
	113		1268	
	106		1422	
300 mm	92	112	1978	1883
	130		1774	
	115		1896	
450 mm	102	101	1819	2019
	117		1936	
	84		2302	
		111		**1753**

As PP-bands are composite material unlike steel specimen therefore the elongation during test might occur in a small segment of the band, and using the entire specimen length for calculating strains may be the reason behind higher modulus of elasticity at greater lengths. Increasing specimen length induces greater non-uniformity/defects, thus resulting in lesser strength values. To check this phenomenon, the tests were repeated with markers placed on the bands (Figure 5.7) at approximate intervals of 22 mm and pictures taken at every 5 mm displacement of the loading clamps. The aim of these markers was to study band elongation during the test to ascertain whether the elongation in the bands occur locally in a small region or globally through out the specimen length.

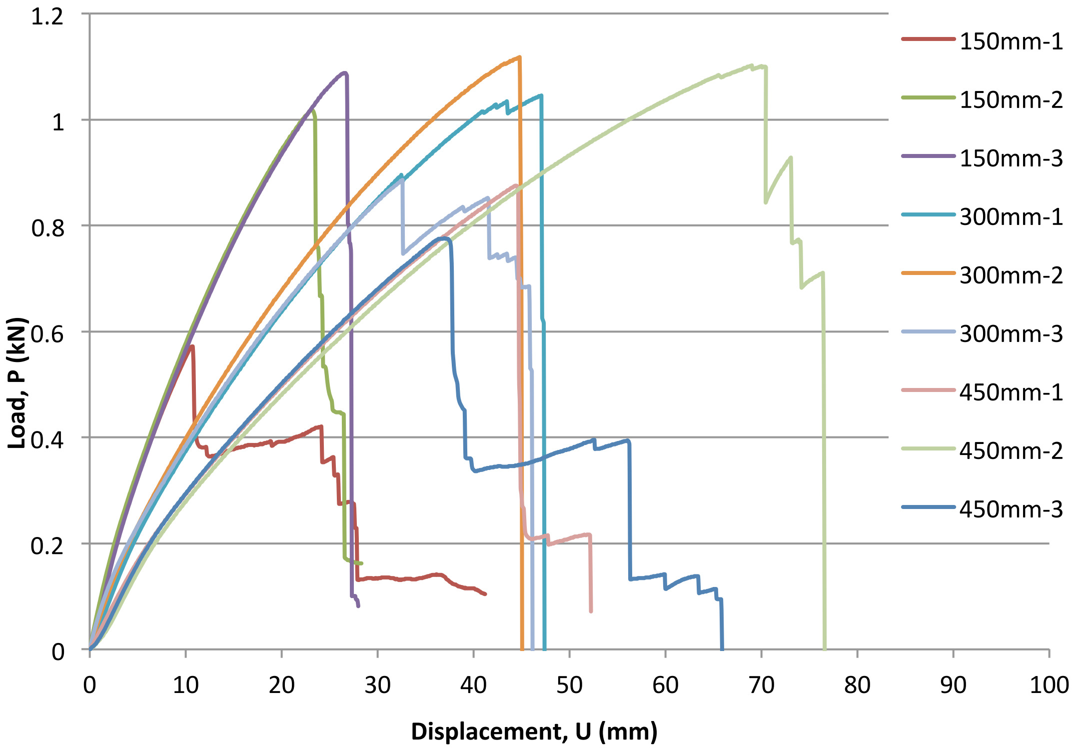


Figure 5.19: Load-displacement graph for tensile test of PP-bands with markers

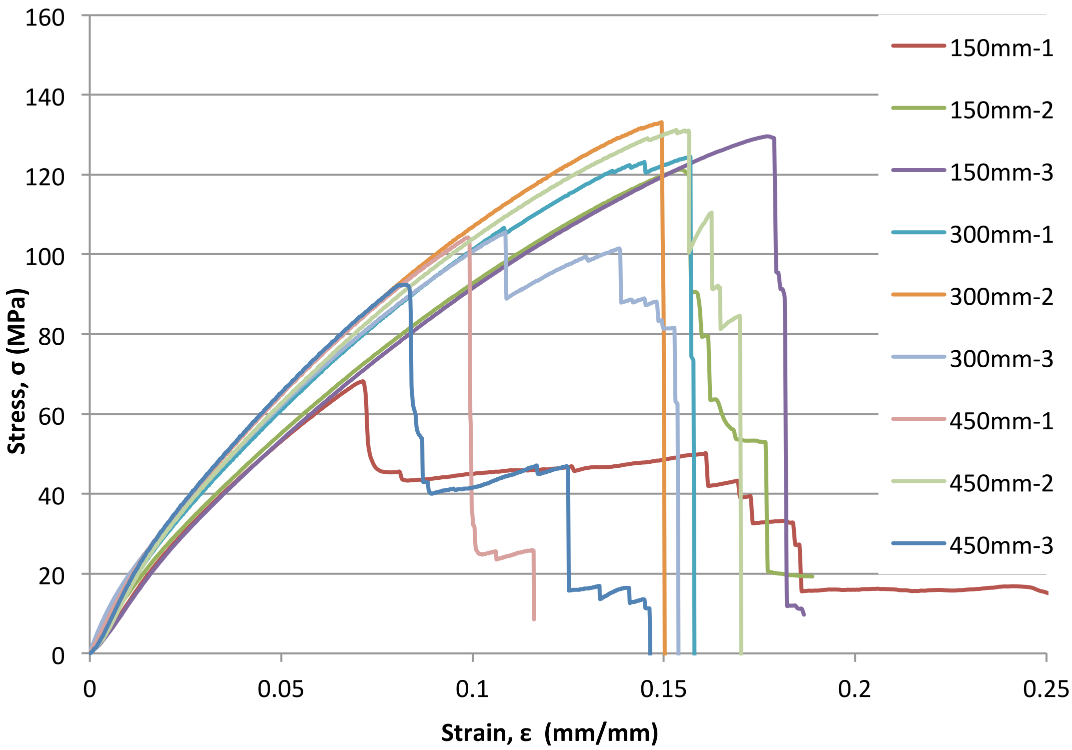


Figure 5.20: Stress-strain graph for tensile test of PP-bands with markers

Table 5.13: Tensile strength and Modulus of Elasticity for PP-bands with markers

Strip Length	**Tensile Strength, f_{ti} (MPa)**	**Mean Tensile Strength, f_t (MPa)**	**Modulus of Elasticity, E_i (MPa)**	**Mean Modulus of Elasticity, E (MPa)**
150 mm	68	125.	1288	1262
	121		1304	
	130		1194	
300 mm	124	121	1477	1585
	133		1580	
	105		1699	
450 mm	104	109	1700	1638
	131		1503	
	92		1711	
		119		**1495**

Results of the second set of tests performed shows similar trend of increasing modulus of elasticity and decreasing tensile strength with an increase in specimen length (Table 5.13). The study of individual segments marked on the specimen reveals material inconsistency in the bands. Each segment shows roughly the same overall elongation at the instance before failure, but for the first 20 mm displacement of the loading clamp, the images taken at every 5 mm displacement of loading clamp show slight variation in elongation of each segment (Appendix-D).

For two specimens (150mm-1 and 450mm-3) the elongation for some segments initially show negative values before extension. Both of these specimens had failure along the length rather than across. This type of failure is due to the skew in the bands making one edge slightly shorter than the other. This type of skew occurs when PP-bands are rolled into bundles and kept that way for long durations. The image processing study of the PP-bands reveals no conclusive values for its strength and elasticity. The final values for strength and modulus of elasticity of PP-band based on the average of the two trials are 114 MPa and 1624 MPa, respectively.

5.5 Concluding Remarks

Material tests were conducted to provide data for numerical modelling and to obtain a better understanding of the mechanical properties of the masonry used in Kashmir region. Two different types of masonry units were tested; red bricks and concrete blocks. Different classes of masonry units and mortar mix proportions were tested to draw comparison of their mechanical properties and the subsequent effect on joint strength and stiffness under varying combinations of mortar and masonry unit.

Initially compressive strength tests were carried out on masonry units and mortar cubes. Resulting average compressive strength for the specimens tested in each class are documented along with their Modulus of Elasticity. The modulus of elasticity for the masonry units found through tests is quite low when compared to most of the finding of the literature. This low elastic modulus is a common feature found in the masonry units produced in Pakistan (and used in Kashmir region) because of the absence of any set regulations and standards from the government.

Modulus of elasticity for mortar increased with higher cement content. Low elastic modulus of mortar specimens in comparison to literature findings is due to high water content.

Research conducted by Jikai Zhou et al. shows that increase in the water content of mortar decreases its compressive strength and modulus of elasticity and subsequently the ultimate failure strains are increased [117]. Use of locally available coarse sand, which is the usual practice on site, can be attributed to the low elastic modulus of mortar as suggested by the study of V.G. Haach et al. [118]. Figure 5.21 shows how the water-cement ratio and the aggregate size affect the modulus of elasticity of mortar. It should be noted that the result of Figure 5.21 are not compared to the result of this study but are only presented to demonstrate the effect of water to cement ratio on modulus of elasticity of mortar.

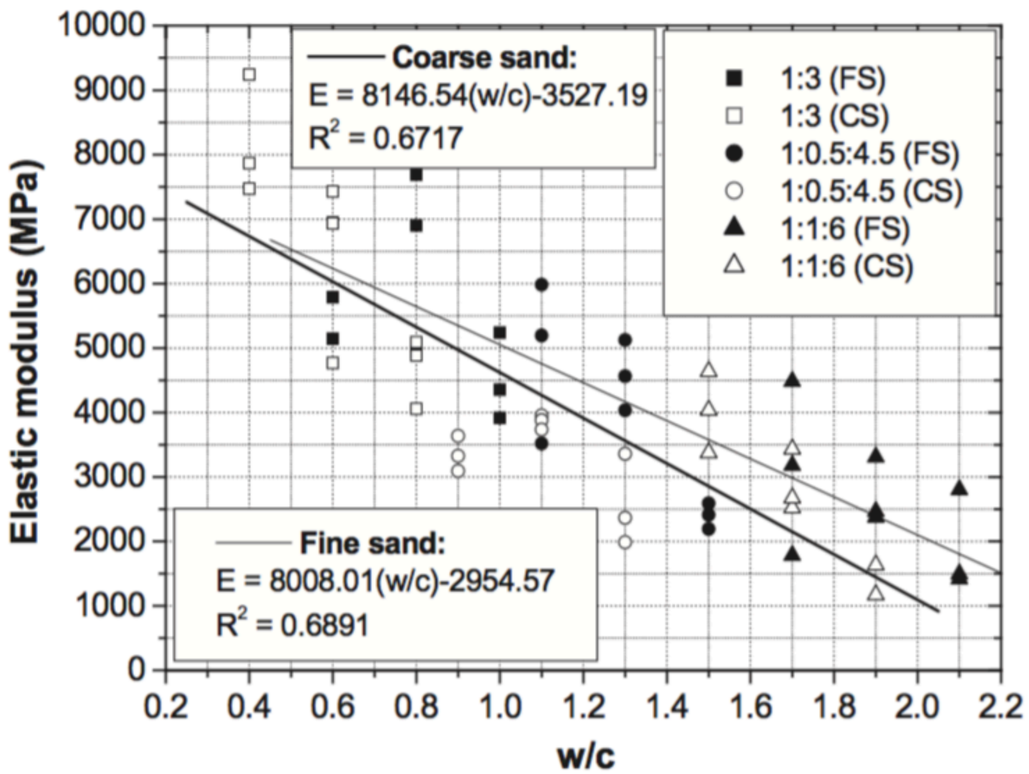


Figure 5.21: Relation between secant elastic modulus and water/cement (w/c) ratio. [118]

Tensile splitting tests were carried out for various masonry units and mortar types. The results showed higher tensile strengths for bricks units in comparison to concrete blocks. For mortar 3-point bending tests were performed to assess the flexural strength, which was twice greater for M1 mortar as compared to M2 mortar due to twice the amount of cement.

For determining mortar interaction with masonry units initial shear strength and bond wrench tests were performed. Results show that the joint strength is combined action of mortar and masonry unit's strength and elasticity. Using a very high strength masonry unit with weak mortar or vice versa would not result in high bond strength. Furthermore the surface finish of the masonry units is another factor that alters the bond strength especially in case of shear strength, where masonry units with rough surface tends to have higher strength and friction coefficient as compared to smooth ones.

The axial tensile tests were carried out to determine the strength and modulus of elasticity of PP-bands used for retrofitting in subsequent chapters. The strength evaluated in this chapter is 114 MPa with 1.62 GPa mean secant modulus for the initial 25% of stress-strain graph. The bands are linearly elastic for the initial 20-25% of the stress-strain graph and after that their elasticity gradually decreases with increasing strain, thus making it a Non-linear Elastic material.

Material used in Kashmir region are tested to be of sub standard nature and with low modulus of elasticity in comparison to those used in developed countries. The amount of water used in mortar should be properly regulated to achieve the desired mechanical properties of masonry. Mortar is a major constituent that affects the joint interaction between the masonry units and subsequently the strength of masonry structure. The tests show that the properties of masonry does not conform to the standards of 'Pakistan Building Code – Seismic Provisions 2007' and should be tested to prove its applicability before using in seismic zones.

Chapter 6 : Shaking Table Test – Wallettes

6.1 Introduction

Shaking table tests are the most effective way to assess the seismic behaviour of structure since the 60's. Earlier, due to the limitation of actuator power, shake tables were used to assess the dynamic properties such as natural frequencies and mode shapes of the structure [116]. With the advancement in technology and with more powerful actuators made available, it is now possible to witness and record the collapse mechanism under severe ground vibrations.

Modern shake tables are capable of simulating various types of waves (sine, square, triangular, random and impulse waves) and any recorded earthquake time history that yields the most realistic picture of structure performance under actual earthquake time history. Most simplistic shake tables are single degree of freedom i.e. their motion is confined to one direction only, whereas, the most sophisticated ones can go up to as many as 6-degrees of freedom [116].

This chapter examines the concurrent dynamic behaviour of two masonry wallette specimens with and without PP-band retrofit. The shaking intensity of the table was increased gradually to determine the intensity of wave that initiates cracking in the wall. Further increase in the intensity of shaking provided useful information about the collapse mechanism especially in terms of PP-band Retrofit.

6.2 Objective

The objective of the chapter is to compare the earthquake resistance of PP-band retrofit masonry wallette with the non-retrofit masonry wallette. For this purpose both retrofit and non-retrofit masonry wallettes would be tested concurrently on the shaking table and quantitative comparison would be based on the amount of Arias Intensity endured by each specimen in grade-3 of EMS damage scale.

6.3 Methodology

Table Specifications:

For the purpose of studying the impact of PP-band reinforcement concurrent tests were performed on retrofit and non-retrofit masonry wallettes on a uni-directional shake table available with NED University, Karachi. The table uses a Hydraulic actuator that generates force of 500 kN. The size of the table is 3m x 3m and maximum movement or stroke capability of ±300 mm with a nominal peak velocity of 1 m/s and acceleration range of ±2g. Maximum payload capacity of the table is 20 metric tonnes and can operate up to frequencies of 50 Hz under maximum payload.

Specimen Description:

English bond masonry, which is the most common bond type for 230 mm thick load bearing walls [119], was used to prepare the wallette specimens of size 1200mm×1200mm, which is the standard wallette size used in diagonal shear test of masonry panels [21], having 230 mm thickness. The type of masonry used for the wallettes was R1M2 as described in the previous chapter i.e. machine pressed bricks with 1:8 mortar. The walls were erected in a steel channel base, which was bolted against the surface of shake table on its web. The clear inside dimension between the two flanges was 230 mm and perfectly accommodated the thickness of masonry wall, as shown in Figure 6.1.

Figure 6.1: Steel channel base used as rigid foundation for masonry wallettes

One of the walls was reinforced with PP-bands spaced at approximately 230 mm vertically and horizontally. The bands were wrapped around the specimen and the ends were fixed to the Wallette using circular steel washers and 25 mm steel screws. 5 mm diameter holes were drilled in the brick unit and PVC plugs were inserted to provide firm grip for the screws. The details of the PP-band connection are shown in Figure 6.2.

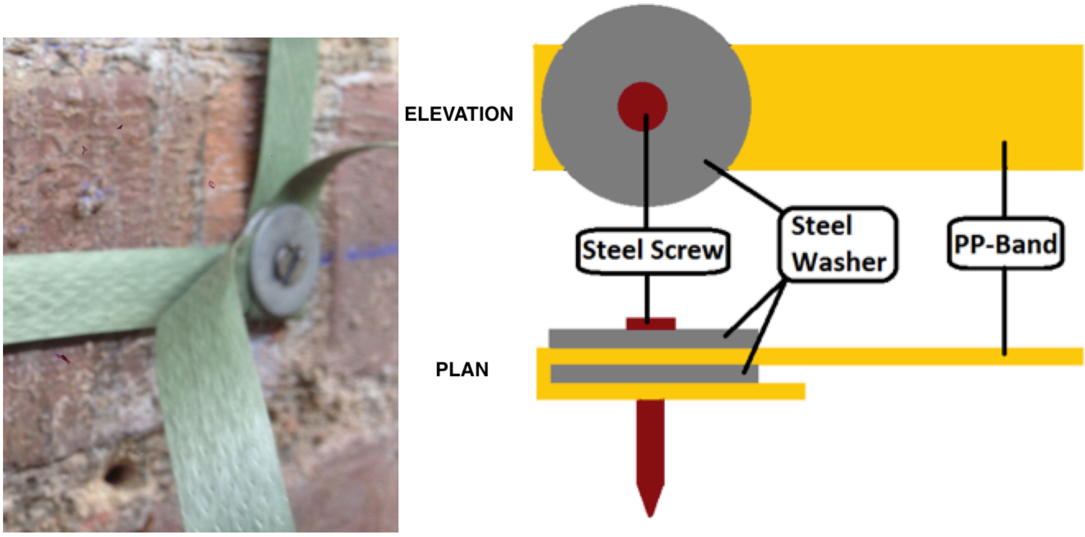


Figure 6.2: Connection detail used to fix the ends of PP-band on wallettes (*Right*), Schematic of connection (*Left*)

<u>Instrumentation:</u>

An accelerometer and a string pot were attached to the top of each specimen to measure response acceleration and displacement of the specimen, respectively. The wooden box attached on the top of each wallette was used to avoid damage to enclosed accelerometer in case of sudden collapse (Figure 6.3). Strings, from string-pots to measure displacement, were attached to the top right corner of each specimen just below the accelerometer. The sampling frequency of both string pots and accelerometer was 100 Hz.

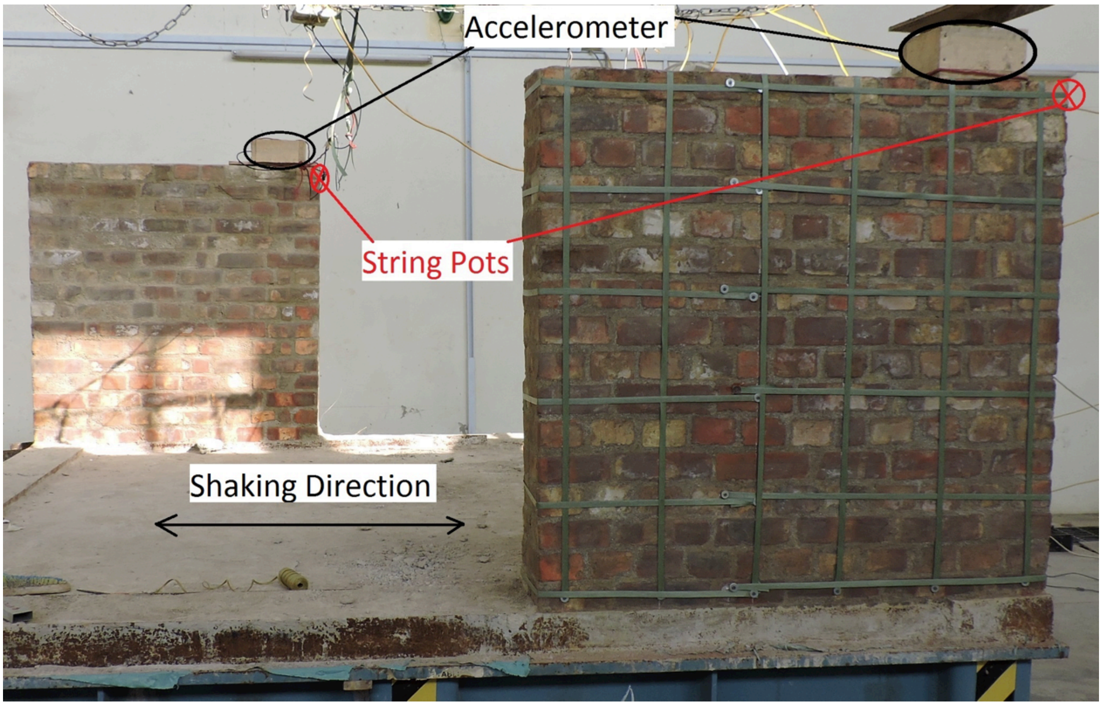


Figure 6.3: Instrumentation for wallette specimens

System Identification Tests:

Initially, *System Identification Tests* were carried out to determine the dynamic properties of the specimens before subjecting them to stronger shake table vibrations for evaluating their performance. System identification tests are non-destructive tests in which the amplitude of the induced vibrations was kept low to avoid any damage to the specimen. Tests performed to determine the dynamic properties of the specimen are explained below:

a) *Hammer test:*

To assess the frequency of wallettes, an impulse load was induced in the specimen by striking with a rubber hammer to avoid damage to the specimen. The frequency of the free vibration response of the specimen is essentially the natural frequency of the system. This frequency can be determined by plotting power spectral density plot against frequency for the response wave. The frequency corresponding to the graph’s peak contributes most power to the wave and in case of free vibration the frequency at peak is the natural frequency of the system. The logarithmic decrement method applied to the response curve determines the equivalent modal viscous damping ratio of the system [120]. Damping ratio is calculated using the approximate damping ratio equation and correction factor as prescribed by Clough and Penzien [121]:

$$\xi = \frac{u_n - u_{n+m}}{2m\pi u_{n+m}} \qquad \textbf{\textit{Eq. 6.1}}$$

where, ξ stands for damping ratio,

u_n stands for *nth* displacement amplitude,

u_{n+m} stands for *(n+m)th* displacement amplitude,

n and *m* being positive integers.

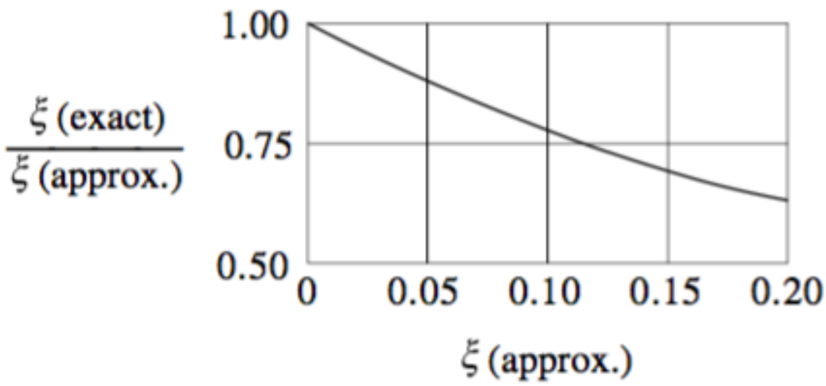


Figure 6.4: Damping ratio correction factor given by Clough and Penzien [121]

Each wallette was struck thrice with hammer and the final result is based on the average of the results for the three trials.

b) *Resonance test:*

Another system identification test performed to assess the natural frequency of the system includes running a series of sine waves at various frequencies. The method as prescribed by FEMA [120] suggests the acceleration amplitude of waves to be maintained at 0.1±0.05g. Each wave was run for the duration of 5 sec., which was sufficient to achieve steady state response in the wallettes. The frequency of sine waves were increased by 5Hz and ranged between 5-30 Hz. The frequency of input wave, which generates maximum response peaks in the system, determines the natural frequency of the specimen. The equivalent modal viscous damping ratio of the system can be determined by the logarithmic decrement method applied to the free vibration response decay curves [120].

Performance Evaluation and Failure Tests:

Once the system identification tests were successfully performed, the vibration intensity of the shake table was gradually increased till the point of crack initiation and the subsequent collapse of the specimen. For evaluating their performance, the wallettes were subjected to increasing amplitudes of sine waves at frequencies of 10 Hz and 5 Hz. The initiation of crack was recorded by sensor attached to a computerised “Data Acquisition System” and video cameras, while the progressive collapse of the wallettes was recorded only through video camera because by then the sensors were removed to avoid damage to them.

6.4 Results

System Identification:

a) *Hammer test:*

The power spectral density plots of the specimen response measured by the inline monitoring sensors reveal a frequency of around 25Hz for retrofitted wall whereas, for non-retrofit wall the frequency obtained is around 40Hz. This disparity may be due to the drilling carried out on retrofit wallette specimen to fix PP-bands. The accelerometer used for non-retrofit wallette had higher sensitivity and the one used for retrofit wallette had higher noise content in the recorded data, which may also be a possible reason for the discrepancy in natural frequency recorded for the two specimens. Figure 6.5 shows the screenshot of the software “SeismoSignal” used to filter and process the response signal of hammer test on non-retrofit wallette. The software is also capable of plotting the *Fast Fourier Transform (FFT)* and *Power Spectral Density (PSD)* plots. The response wave from the wallette is shown in the top plot whereas the FFT and PSD plots are shown bottom left and bottom right, respectively. The table on the left side of the FFT plot can be used to figure out accurately the frequency at which the graph peaks. The peaks after the 50Hz value can be ignored because the sampling frequency of the accelerometers cannot detect frequencies higher than 50Hz. And the graph beyond 50Hz is a replica of the pre 50Hz graph.

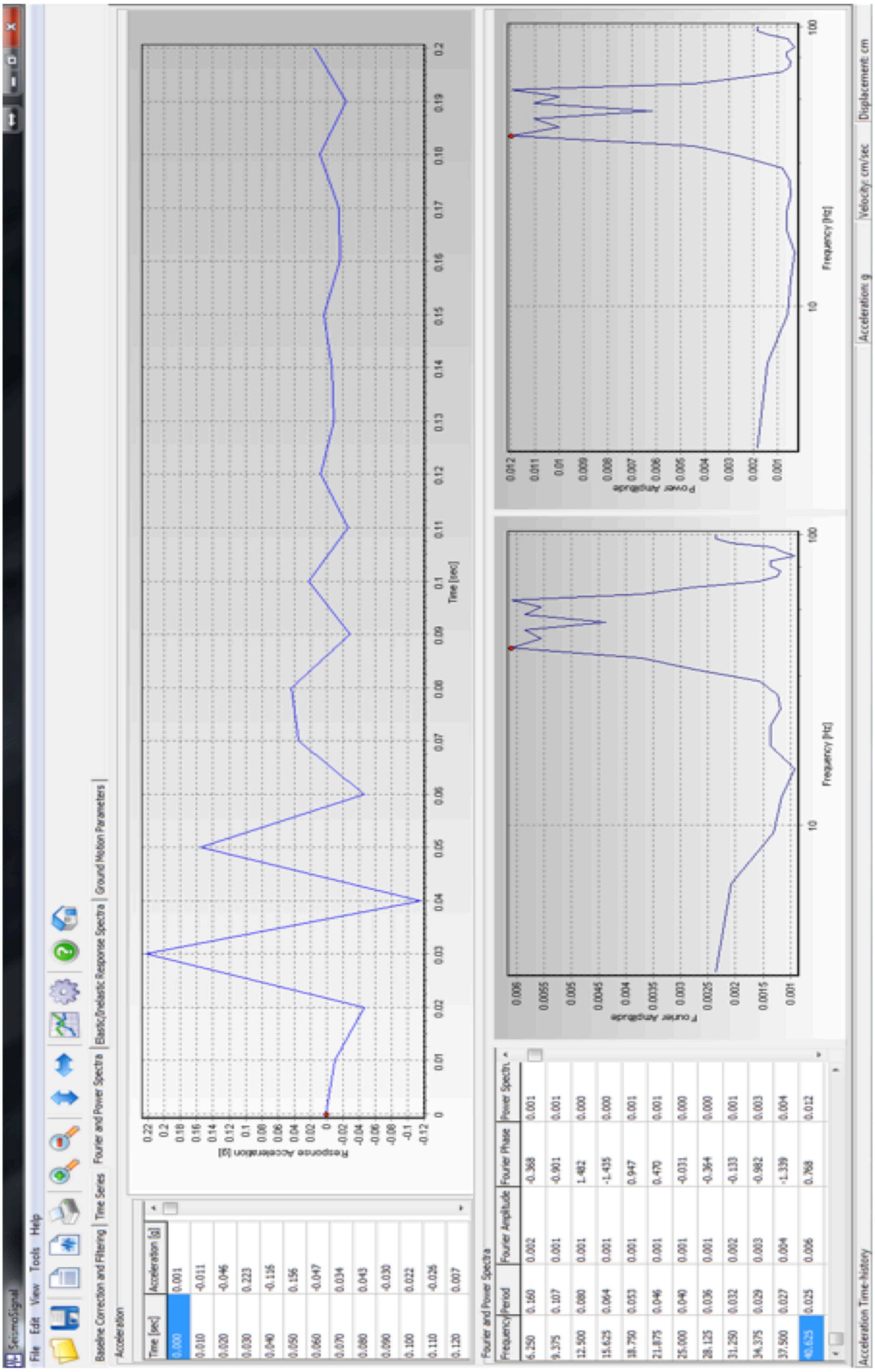


Figure 6.5: Screenshot of signal processing in 'SeismoSignal'

Furthermore, the numerical model developed with Abaqus using micro-modelling technique properties of brick and mortar joints from material testing reveals a natural frequency of 36Hz for R1M2 wallette, which is close to the value of 40 Hz obtained for non-retrofit specimen.

Logarithmic decrement method applied to the response of wallettes from hammer blow reveals a damping ratio of 10% for non-retrofit wallette and 8% for retrofit wallette.

b) Resonance test:

Analysis of resonance test results showed response-to-input ratio of acceleration amplitudes at 30 Hz (Figure 6.6), which was the maximum frequency used for the resonance test. This shows that the natural frequency of the system lies above 30 Hz.

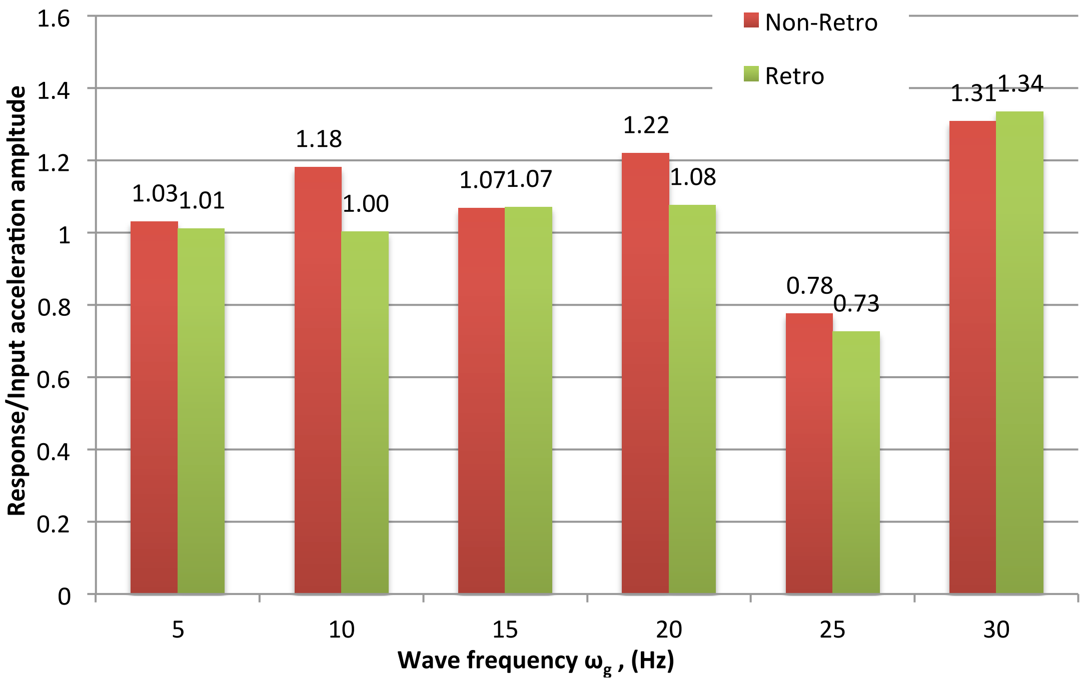


Figure 6.6: Response to input ratio of acceleration amplitudes for different frequencies

<u>Performance Evaluation and Failure Tests:</u>

Table 6.1 shows the sequence and necessary information about the waves that were run for performance evaluation and failure tests.

Table 6.1: Wave sequence used for shake table test of wallettes

S. No.	ω_g (Hz)	A_s (mm)	T_d (sec)	Wave Type	a_{rms} (g)	PGA (g)	I_a (m/s)
1	10	2.2	5	sine	0.2	0.28	3.09
2	10	2.5	5	sine	0.23	0.32	4.07
3	10	3	5	sine	0.3	0.42	6.83
4	10	3.5	5	sine	0.35	0.49	9.40
5	10	4	5	sine	0.40	0.57	12.59
6	10	4.5	5	sine	0.48	0.68	18.00
7	10	5	5	sine	0.52	0.73	20.57
8	10	5.5	5	sine	0.56	0.79	24.03
9	10	6	5	sine	0.64	0.90	31.53
10	10	6.5	5	sine	0.67	0.94	34.42
11	10	7	5	sine	0.74	1.04	41.71
12	10	8	5	sine	0.80	1.14	50.05
13	10	10	5	sine	1.06	1.50	87.21
14	10	10	4	sine	1.05	1.49	68.96
15	10	11	4	sine	1.24	1.76	95.45
16	10	12	4	sine	1.34	1.90	111.43
17	5	15	5	sine	0.61	0.87	29.13
18	5	16	5	sine	0.65	0.92	32.4
19	5	17	5	sine	0.69	0.97	36.50
20	5	18	5	sine	0.72	1.02	40.37
21	5	19	5	sine	0.76	1.08	45.34
22	5	20	5	sine	0.80	1.12	48.85
23	5	21	5	sine	0.83	1.17	52.95
24	5	22	5	sine	0.86	1.22	57.24
25	5	23	5	sine	0.89	1.26	61.53
26	5	24	3.5	sine	0.94	1.32	47.75
27	5	25	2.6	sine	0.96	1.36	37.79

ω_g – Frequency of input wave.

A_s - Amplitude of displacement used as input for generating sine wave.

T_d - Overall duration of time history.

PGA - Peak Ground Acceleration is the maximum acceleration measured.

a_{rms} - Root Mean Square Acceleration is the mean of area under the squared acceleration time history.

$$\boldsymbol{a_{rms}} = \frac{\mathbf{1}}{\boldsymbol{T_d}}\{\int_{\mathbf{0}}^{\boldsymbol{T_d}}(\boldsymbol{a(t)})^2\boldsymbol{dt}\}^{\frac{1}{2}} \qquad \textit{Eq. 6.2}$$

I_a - Arias Intensity is used as a measure of potential destructiveness of ground motion. It is proportional to area under the squared acceleration time history of ground motion [122].

$$\boldsymbol{I_a} = \frac{\boldsymbol{\pi}}{\boldsymbol{2g}}\int_{\mathbf{0}}^{\boldsymbol{T_d}}\{\boldsymbol{a(t)}\}^2\boldsymbol{dt} \qquad \textit{Eq. 6.3}$$

a(t) – Acceleration time history.

g – Acceleration due to gravity i.e. 9.81m/s^2.

t – Time.

It can be seen from Table 6.1 that initially 10Hz waves were used and later the frequency of the waves were reduced to 5Hz. The reason for using higher frequency of 10Hz in the beginning was to accelerate the process of crack initiation. This is because the initial undamaged specimen had higher stiffness and consequently higher natural frequency thus requiring a higher frequency input to resonate the specimen and produce cracks. Once the cracks appear and the stiffness of the specimen deteriorates which in turn reduces the natural frequency of the system, lower frequency input is more capable of producing resonance and accelerate the process of collapse. Furthermore, higher frequency waves at higher intensity was resonating the surrounding structure, and therefore it was advisable to run high amplitude waves at lower frequencies.

Non-retrofit wallette developed visible cracks at wave no. 9 (PGA = 0.09 g) and eventually collapsed at wave no. 12 (PGA = 1.14 g); whereas, retrofit wallette showed cracks at wave no. 13 (PGA = 1.50 g) and collapsed at wave no. 27 (PGA =

1.36 g) as shown in Table 6.2, but sustained maximum PGA of 1.90 g at wave no. 16 (Table 6.1). For non-retrofit sample, horizontal crack appeared closer to base and split the wall in two as shown in Figure 6.7. Red lines on wallette specimens, in Figure 6.7, highlight the location of cracks which otherwise may not be clearly visible. The "Grades" mentioned in the caption of Figure 6.7 are discussed later on in this chapter. With no residual shear strength after the development of cracks, the specimen kept standing due to its self-weight and friction between the layers. Under the action of further vibrations the wall above the crack line displaced and eventually collapsed.

Retrofit specimen had a similar crack orientation as non-retrofit specimen i.e. horizontal and very close to base (Figure 6.7). Post cracking behaviour of retrofit specimen showed better integrity as compared to non-retrofit specimen under higher levels of ground vibrations. The presence of PP-bands kept the specimen intact for longer durations of ground vibration and the collapse was gradual, marked by the failure of PP-bands as shown in Figure 6.8. In terms of peak strength performance, the non-retrofit specimen developed cracks much earlier as compared to retrofit specimen. According to the studies conducted by Mayorca and Meguro [61], PP-bands only contribute to the post peak strength of the masonry i.e. PP-bands come into play once the cracking initiates. However, the increase in peak strength of retrofit specimen is due to the higher stiffness of the specimen as mentioned earlier in results of system identification test. This increase in peak strength of retrofit specimen can be attributed to the pre tensioning of the PP-bands done manually during application to straighten the bands and avoid any looseness after their fixing.

Figure 6.7: Non-retrofit (*top*) and retrofit (*bottom*) specimen at cracking (Grade-3)

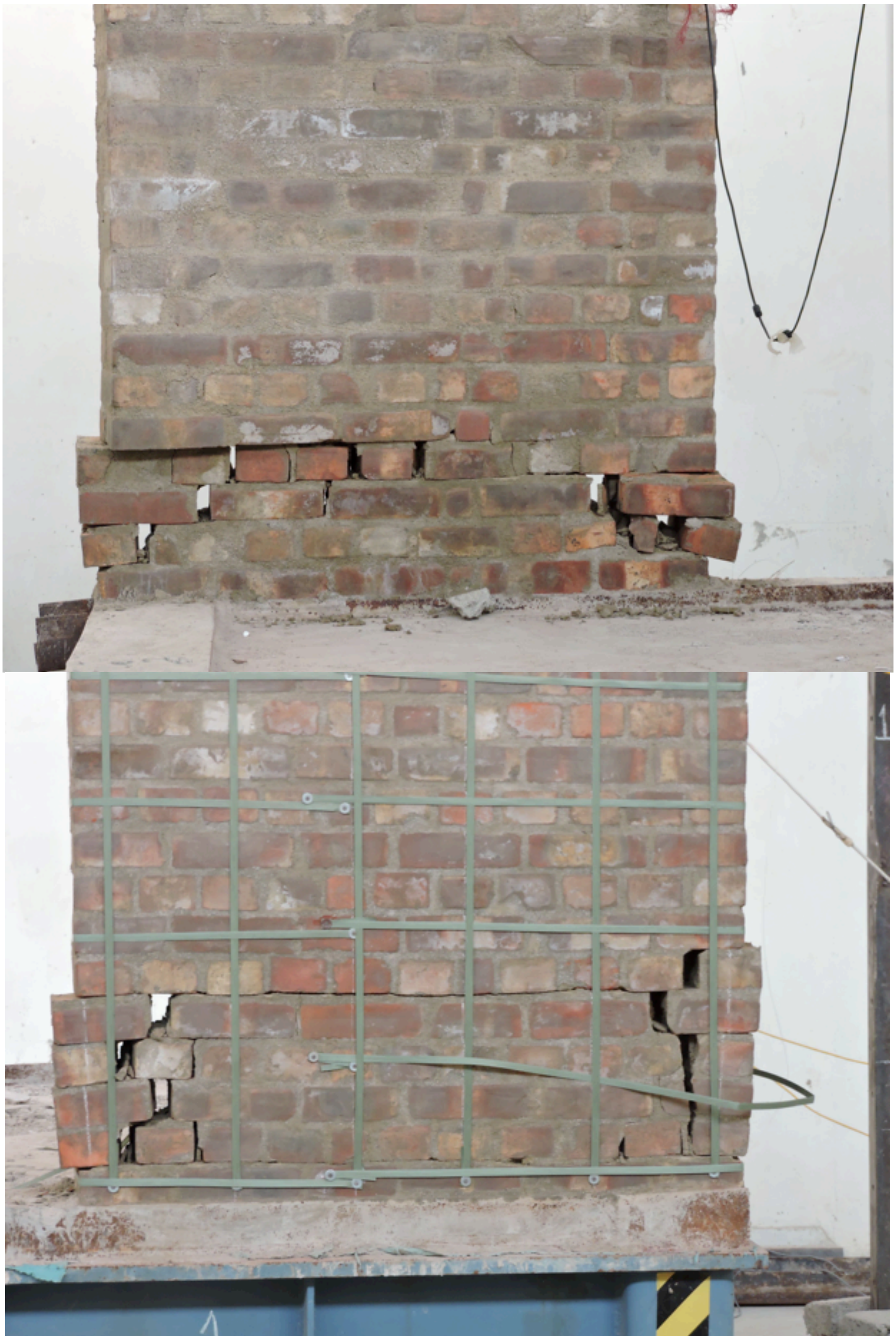


Figure 6.8: Non-retrofit (*top*) and retrofit (*bottom*) specimen before collapse (Grade-4)

Cracks in retrofit specimen appeared after the non-retrofit specimen had already collapsed as shown in Table 6.2. Therefore, comparing the PGA or a_{rms} of individual waves at cracking or collapse is not the true representation of the post peak performance of masonry, as evident from Table 6.1, where the retrofit wallette after having sustained higher PGAs collapse at a value that is close to the one recorded for collapse of non-retrofit wallette. This suggests that each subsequent vibrational wave experienced by the specimen after the initiation of cracks contributes to the further deterioration and eventual collapse of the specimen even if the acceleration amplitudes are kept low.

Post cracking behaviour and strength of the specimen can effectively be quantified by the adding up the damage potential contributed by each wave. Therefore, the arias intensity of each wave sustained by the specimen after the initiation of crack was added up to compare the post cracking behaviour of retrofit specimen against the non-retrofit one. The progressive deterioration and eventual collapse of the wallette specimens due to the accumulation of arias intensity of each subsequent wave is quantified in term of EMS (European Macroseismic Scale 1998) Grades. Figure 6.9 shows the classification of damage for masonry structures based on EMS 98 [123] [124].

	**Grade 1: Negligible to slight damage** **(no structural damage,** **slight non-structural damage)** Hair-line cracks in very few walls. Fall of small pieces of plaster only. Fall of loose stones from upper parts of buildings in very few cases.
	**Grade 2: Moderate damage** **(slight structural damage, moderate non-structural damage)** Cracks in many walls. Fall of fairly large pieces of plaster. Partial collapse of chimneys.
	**Grade 3: Substantial to heavy damage** **(moderate structural damage,** **heavy non-structural damage)** Large and extensive cracks in most walls. Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).
	**Grade 4: Very heavy damage** **(heavy structural damage,** **very heavy non-structural damage)** Serious failure of walls; partial structural failure of roofs and floors.
	**Grade 5: Destruction** **(very heavy structural damage)** Total or near total collapse.

Figure 6.9: Damage classification of masonry buildings based on EMS 98 [123] [124]

Table 6.2: Table showing specimen condition according to EMS Grades

		Cummulative I_a, (m/s)		EMS Grade 1998	
S. No.	I_a (m/s)	Non-Retro	Retro	Non-Retro	Retro
1	3.09	No damage	No damage	No damage	No damage
2	4.07	No damage	No damage	No damage	No damage
3	6.83	No damage	No damage	No damage	No damage
4	9.40	No damage	No damage	No damage	No damage
5	12.59	12.59	No damage	1	No damage
6	18.00	30.59	No damage	1	No damage
7	20.57	51.16	No damage	1	No damage
8	24.03	75.19	No damage	2/3	No damage
9	31.53	106.72	No damage	3	No damage
10	34.42	141.13	No damage	3	No damage
11	41.71	182.84	No damage	4	No damage
12	50.05	232.89	50.05	5	1
13	87.21	Collapsed	137.26	Collapsed	2
14	68.96	Collapsed	206.21	Collapsed	3
15	95.45	Collapsed	301.66	Collapsed	3
16	111.43	Collapsed	413.09	Collapsed	3
17	29.13	Collapsed	442.22	Collapsed	3
18	32.4	Collapsed	474.62	Collapsed	3
19	36.50	Collapsed	511.12	Collapsed	3
20	40.37	Collapsed	551.49	Collapsed	3
21	45.34	Collapsed	596.83	Collapsed	3
22	48.85	Collapsed	645.68	Collapsed	3
23	52.95	Collapsed	698.62	Collapsed	3
24	57.24	Collapsed	755.86	Collapsed	4
25	61.53	Collapsed	817.39	Collapsed	4
26	47.75	Collapsed	865.15	Collapsed	4
27	37.79	Collapsed	902.93	Collapsed	5

Table 6.2 shows the evolution of damage in the specimens based on EMS Grades and the accumulation of arias intensity after the initiation of crack till the point of collapse. Performance evaluation based on arias intensity levels have been carried out by Sathiparan and Meguro for their dynamic testing of PP-band retrofit masonry [125-129]. The sequence of the waves and their ground motion parameters are the same as given in Table 6.1. It can be seen from Table 6.2 that the transition from grade-1 to grade-3 especially in case of non-retrofitted specimen was very sudden with very little time to record grade-2. This is due to the brittle nature of masonry that did not allow enough time to mark the grade-2 condition of the specimen. However, the wave at which the specimen moves from being in grade-1 to grade-3 is attribute to grade-2 condition for the purpose of quantifying analysis.

Figure 6.10 shows the accumulation of arias intensity for each wallette specimen with every subsequent vibration wave they were subjected to after the initiation of crack. Initiation of Grade-1 condition marked by the development of micro-cracks in masonry was hard to visually ascertain and was based on the responses measured by the in-line accelerometers and string pots. Grade-3, however, can be easily determined on visual inspection and is defined as the instance when cracks have completely developed over the entire length of the wall by separating the specimen along the crack line as shown in Figure 6.7.

Grade-3 damage condition is of major interest because this grade marks the complete loss of masonry shear strength offered by the cohesion of the mortar and any further resistance to lateral load now depends on the friction between masonry layers and the presence PP-bands. Figure 6.10 show that retrofit specimen continued to stay in grade-3 for higher levels of arias intensity as compared to the non-retrofit specimen where the specimen soon moved into grade-4 and then to eventual collapse. Thus, retrofit specimen showed better post peak performance and higher structural integrity in comparison to non-retrofit specimen. Furthermore, it can be observed that the transition from grade-1 to grade-5 and eventually to collapse is relatively linear for non-retrofit specimen because of the brittle nature of the masonry and the deterioration process is very quick.

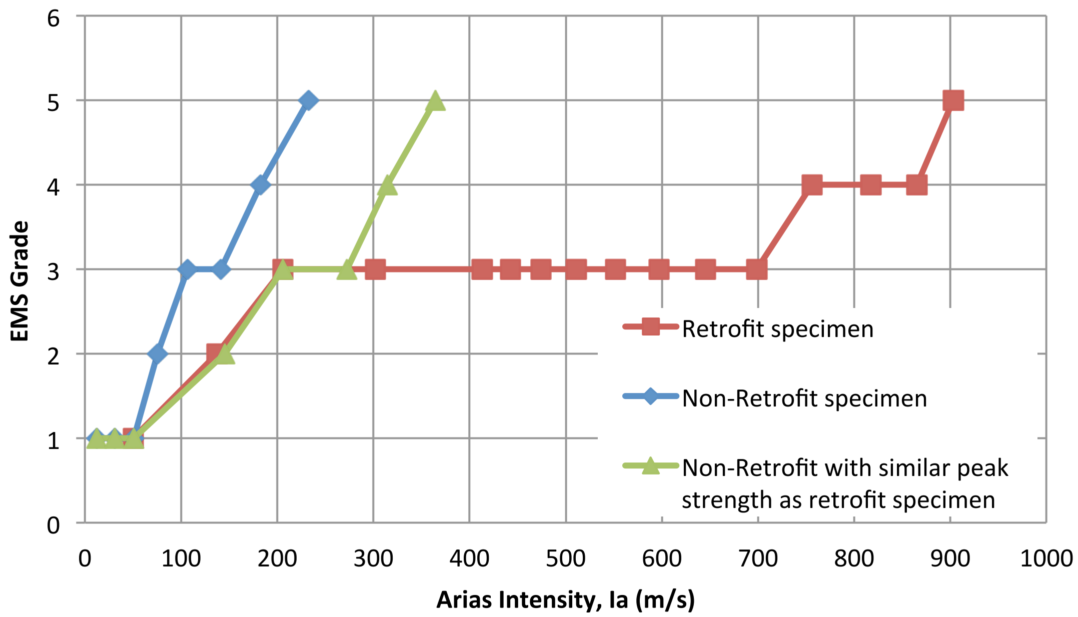


Figure 6.10: EMS Grade vs. Cumulative Arias Intensity

As the post peak behaviour of masonry is of prime interest rather than the peak strength itself, the accumulated arias intensity for non-retrofit specimen at grade-3 was scaled up to match the grade-3 initiation value or the peak strength of retrofit specimen as given by the green curve in Figure 6.10. With the increase in the peak strength of non-retrofit specimen the accumulated arias intensity in grade-3 for non-retrofit specimen was also scaled up based on the multiplier used for scaling up its peak strength. A comparison of cumulative arias intensity of retrofit specimen to non-retrofit specimen in grade-3 suggests 14.3 times higher post peak performance for retrofit specimen and 7.4 times when it is compared to the non-retrofit specimen with peak strength scaled up to match the peak strength of non-retrofit specimen as shown in Figure 6.11. The cumulative arias intensity taken by retrofit specimen in grade-3 is 492.41 m/s where as for non-retrofit specimens predicted using 14.3 and 7.4 factor has 34.42 m/s and 66.5 m/s, respectively.

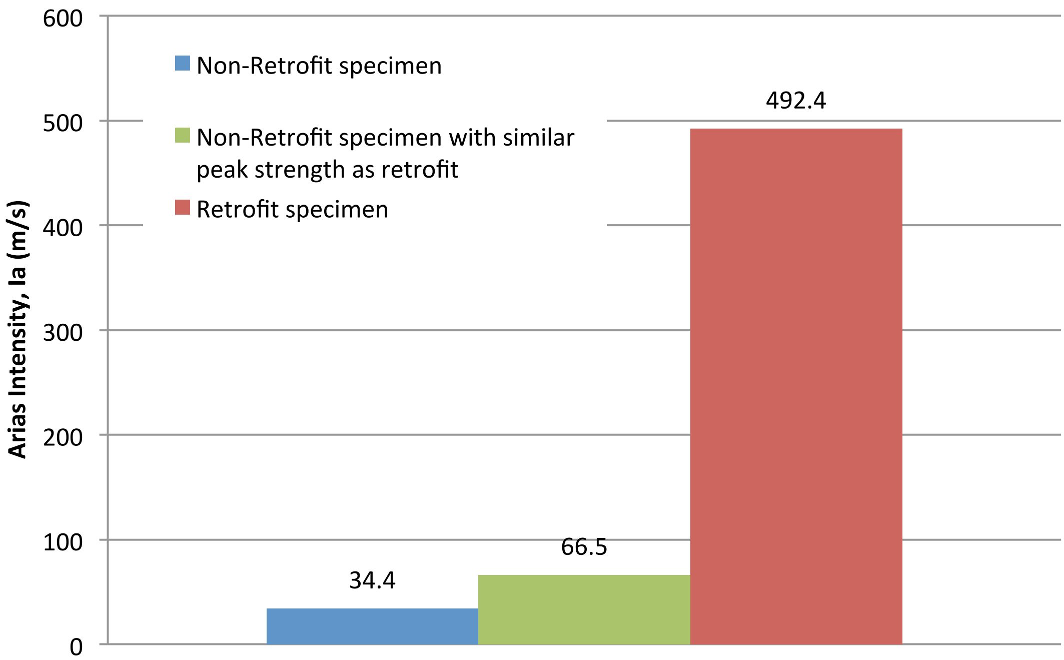


Figure 6.11: Cumulative arias intensity in grade-3 for wallette specimens

6.5 Concluding Remarks

Shake table test of wallette specimens was conducted to compare the performance of PP-band retrofitting for masonry. Both the wallettes although built out of same material had different dynamic properties due to the disturbances created in retrofit specimen for the application of PP-bands especially by the drilling effect. Therefore the retrofit specimen having lesser stiffness and was able to stand for higher levels of ground vibration without showing cracks. Another reason attributed to the high strength of retrofit specimen was the pre-tensioning applied to the bands for getting rid of looseness in the bands. As the interest of study was in the post cracking behaviour and integrity of the wallette, therefore, the performance comparison was based on the levels of arias intensity each wallette endured in grade-3 damage condition before moving into grade-4.

Retrofit specimen showed 14.3 times higher post cracking performance in terms of the arias intensity it endured in grade-3 without deterioration in comparison to the non-retrofit specimen. Another comparison was made based on the assumption that both the wallette specimens developed cracks at the same time for this purpose the values of non-retrofit specimen was scaled up to match the peak strength of retrofit specimen. With this modification the post cracking/peak strength performance of retrofit specimen in comparison to a non-retrofit specimen of equal peak strength was 7.4 times higher. As long as the bands were intact the collapse was prevented and the breaking of bands marked further deterioration of the retrofit specimen.

Hence it can be concluded that the strength of the PP-bands and its end-connections determines when the specimen would move from being in grade-3 damage condition to grade-4. The end connections used in this chapter does not apply the same details as originally proposed by Mayorca et al. [63]. The original retrofit method is more cumbersome as uses steel bars embedded at the ends of the wall for supporting PP-bands and the mesh is prepared before putting it on the structure, while the suggested method is more feasible in terms of application as individual bands can be applied where required and no substantial damage is done to the existing structure. The proposed fixing detail have shown to enhance the seismic performance of masonry, however, putting the screws through the bands caused damage to the bands. Therefore, a different method of fixing PP-bands would be tried in the next chapter that does not damage the band and yet is equally simple to apply to existing structure.

Chapter 7 : Shaking Table Test – Structure

7.1 Introduction

A lot of masonry structures in Pakistan are unreinforced and remain a potential threat to its inhabitants in the event of an earthquake. These structures, especially in rural areas need to be strengthened within a minimum cost that is affordable for the house owner and provides a certain level of safety during earthquakes. Many researches have been conducted to assess the seismic performance of masonry structures using shake table tests. But, due to the variations in masonry based on region (materials and labour) it requires experimental evidences to support the real scale phenomenon.

To make a final recommendation on the use of PP-bands as an effective method for seismic strengthening of masonry, the on-going experimental programme of shake table test had to be extended to test the effectiveness of PP-band retrofit on a real scale structure acted upon by real earthquake time history records and increasing amplitudes of sine waves. The test was performed on a full-scale room built out of English bond masonry and reinforced with PP-bands. The first specimen suffered accidental damage in the initial stages of the test, as a result the entire test had to be performed again on another specimen room. The methodology for the first specimen and the subsequent inferences drawn from its damage and collapse are also presented in this chapter.

7.2 Objective

The objective of this chapter is to assess the performance of PP-band retrofit for real scale masonry structure under seismic loading. The assessment of PP-band retrofit masonry is based on the measure of Arias Intensity endured by the structure after the initiation of cracks i.e when the retrofit bands start to take load. Two different rooms are tested to compare the effects of two different fixing mechanism employed for the application of bands.

7.3 Methodology

Room-1

Specimen Description:

To check the reliability of PP-band retrofit on a real scale structure, a masonry room of size 2.7 m × 2.7 m plan area on wall centre line (corresponding to the size of the platform of shaking table) and 2.7 m height was tested on shake table. The room had no column and comprised of load bearing brick masonry walls constructed using English bond. The two walls parallel to the direction of shaking had a window opening in each of them. The opening for door was provided in one of the wall perpendicular to the direction of shaking while the second wall, perpendicular to the direction of shaking, had no opening. Lintel height was maintained at 1950 mm for all openings and the sill level for windows was at 900 mm. Lintels used are made of reinforced concrete with a cross sectional size of 230 mm x 150 mm and provided only over the openings with 150 mm bearing on either side of the opening. The model follows the same configuration and dimensions as used by Hanzato et al. for their shaking table test on Pakistan masonry [130].

Roof comprised of RCC ring beam with cross-sectional dimension of 380 mm x 150 mm placed on top of all four walls. The opposite beams were connected with 3 steel reinforcement bars to provide a rigid diaphragm effect of an RCC roof slab as shown in Figure 7.2 and Figure 7.4. Complete RCC roof slab was avoided because the collapse of a rigid heavy diaphragm could cause serious damage to the shake table. To account for the weight of complete RCC slab additional weight of 1 tonne in the form of sand bags was evenly distributed on all four walls, while the weight of the beam itself was 1.48 tonnes and the weight of total masonry works accounted for 9.04 tonnes. On the outside the beam was flush with the wall and the width of the beam in excess of wall thickness i.e. 150 mm was provided as overhang inside the room. The type of masonry used for the room was R1M2 as described in the Material Test chapter i.e. machine pressed bricks with 1:8 mortar. Foundation arrangement was made of steel channel sections of width 230 mm similar to the one discussed in wallette test (Chapter 6).

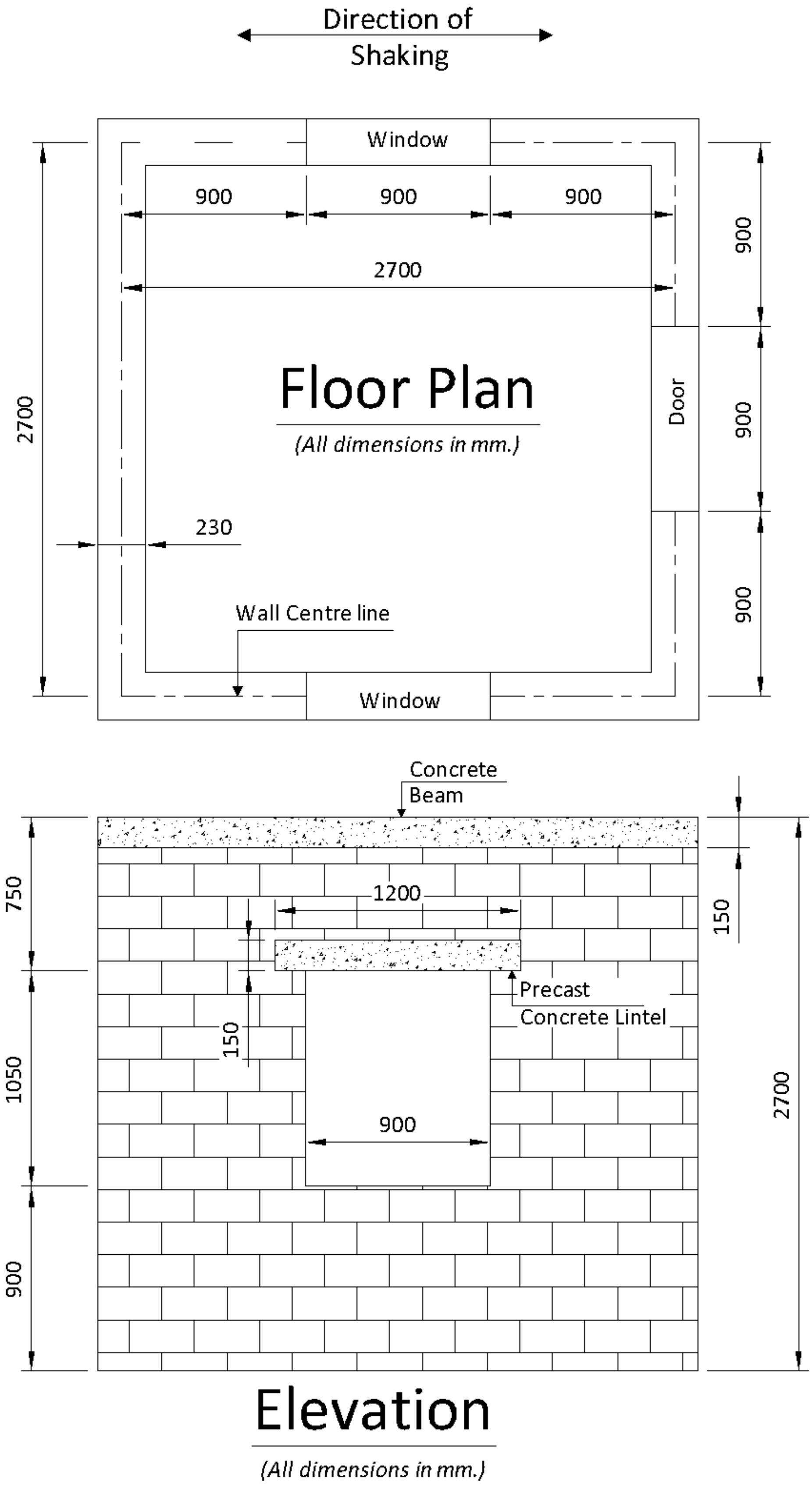
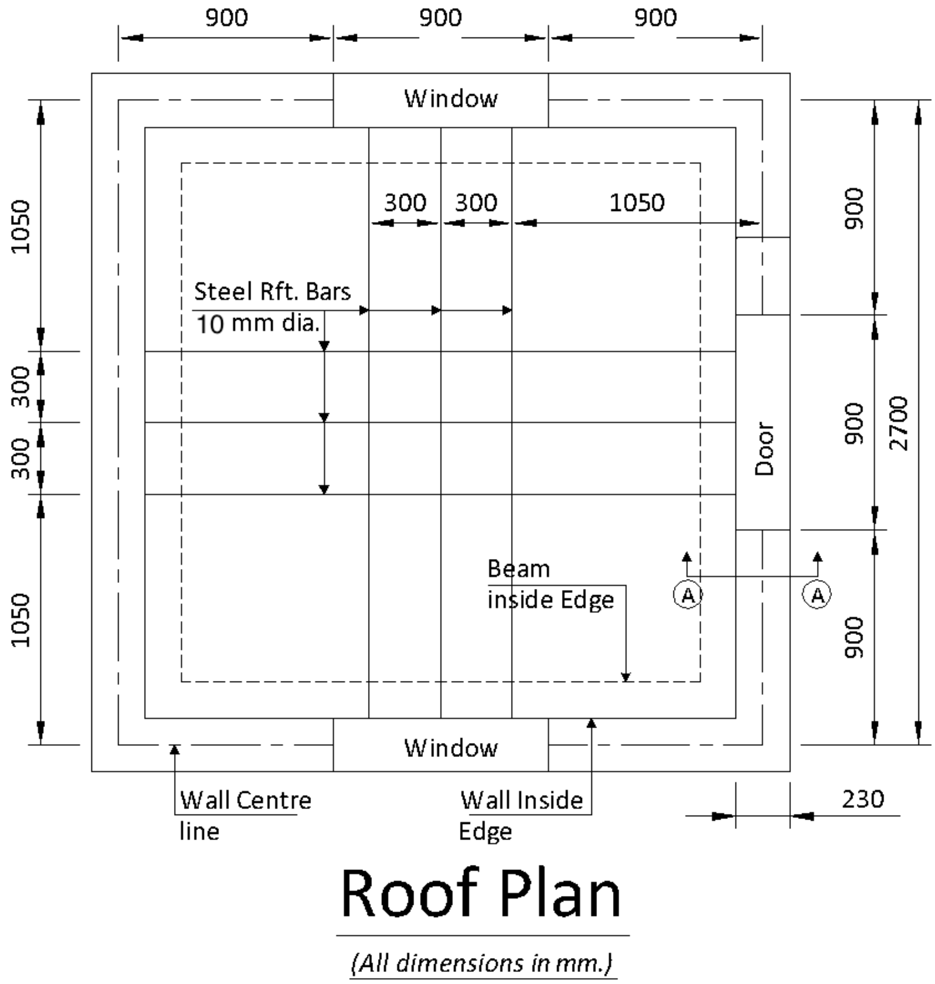


Figure 7.1: Floor Plan and Elevation for model Room-1



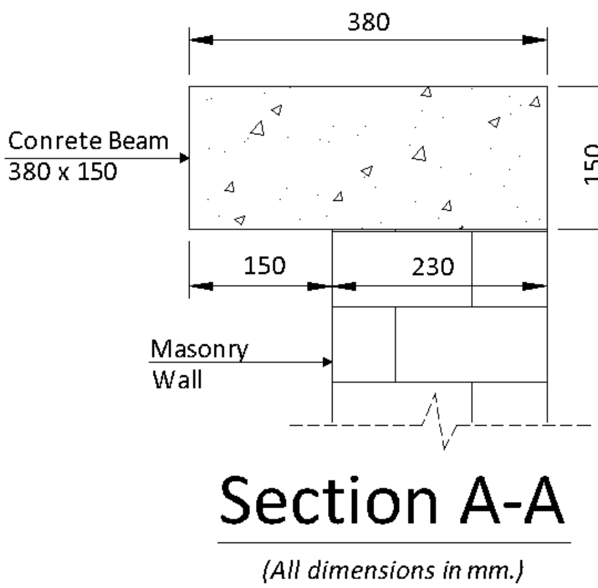


Figure 7.2: Roof Plan and enlarged cross section of roof beam for Room-1

Masonry walls were fitted with PP-band reinforcement spaced at approximately 230 mm vertically and horizontally. Apart from the 230 mm spacing criteria, PP-bands were provided at the edges of all openings and walls. The bands were put on the outside and as well as the inside of the room. As the bands could not circle around the walls, as in case of existing structure, therefore, strips of appropriate length were cut and applied on each wall separately. For the outside horizontal bands, the strips had an extra length of approximately 300 mm on each end, which was used to anchor the strip on the adjacent sidewalls as shown in Figure 7.3. This arrangement was useful as it strengthened the wall corner joints. Whereas, the horizontal bands inside the room run from one end of the wall to the other and did not cover the corner joints by overlapping on to the adjacent side walls. The vertical bands (inside and outside) run from the top most masonry layer to the one near the base. The crossover points of vertical and horizontal bands were attached to the wall using a circular steel washer and a steel screw as shown in Figure 7.3. The type of end-fixing mechanism used for PP-bands was the same as that described in and used for wallette test.

Figure 7.3: Connection detail between horizontal and vertical bands spaced at 230mm (*top*), typical length and arrangement of horizontal PP-bands (*bottom*)

Figure 7.4: Typical roll of PP-band sold in the market (*top*), inside view of the roofing detail showing steel reinforcements connecting opposite beams (*bottom*)

Instrumentation:

Two accelerometers were placed on top of the roof beam at two corners of wall with door opening to measure response accelerations in the direction of shaking. Six string pots were provided at three levels; two at roof level, two at lintel level and two at sill level. The sampling frequency of both string pots and accelerometers was 100 Hz.

Figure 7.5: Instrumentation for Room-1

Performance Evaluation and Failure Tests:

The vibration intensity of the shake table was gradually increased till the point of crack initiation and the progressive collapse of the structure. For evaluating performance, the structure was subjected to two sine waves of frequencies 0.5 Hz and 2 Hz. The initiation of crack was recorded by sensor attached to a ‘Data Acquisition System’ and video cameras while the progressive collapse of the structure was recorded only through video camera because by then the sensors were removed to avoid damage to them.

Room-2

Specimen Description:

Room configuration, masonry type and opening sizes were kept the same as 'Room-1'. The height of the room changed from 2.7 m to 2.6 m. This is due to the change in the roofing system. Previously, 150 mm deep RCC beam was provided on top of the high walls, which gave an overall height of 2.7 m. For Room-2 the roofing system was changed to precast concrete tiles of thickness 50 mm with a lean concrete floor finish of around 20 mm thickness, giving a total height of 2.62 m (Figure 7.6).

The RCC ring beam at roof level, in the 'Room-1' test, proved to be very rigid and was potentially detrimental to the shaking table in the event of structural collapse. Furthermore, the removal and disposal of such heavy and rigid beam was very laborious and time consuming. Therefore, it was deemed appropriate to change the heavy RCC beam roof system with concrete tiles of size 750 mm x 600 mm and 50 mm thickness. These roof tiles were supported on three precast concrete girders and the room walls. The girders were evenly spaced from the wall to divide the 2.7 m centre-to-centre span between two opposite walls into 4 equal spans to support the 750 mm long roof tiles. RCC girders were provided perpendicular to the direction of shaking in order to transfer their load on to walls, which have their stronger plane parallel to the direction of shaking, and to prevent the slipping out of girders in the event of strong ground motion (Figure 7.6 and Figure 7.7). The weight of the roofing system inclusive of girders, roof tiles and roof finish totals to approximately 1.5 tonnes and for masonry walls it was approximately 9 tonnes.

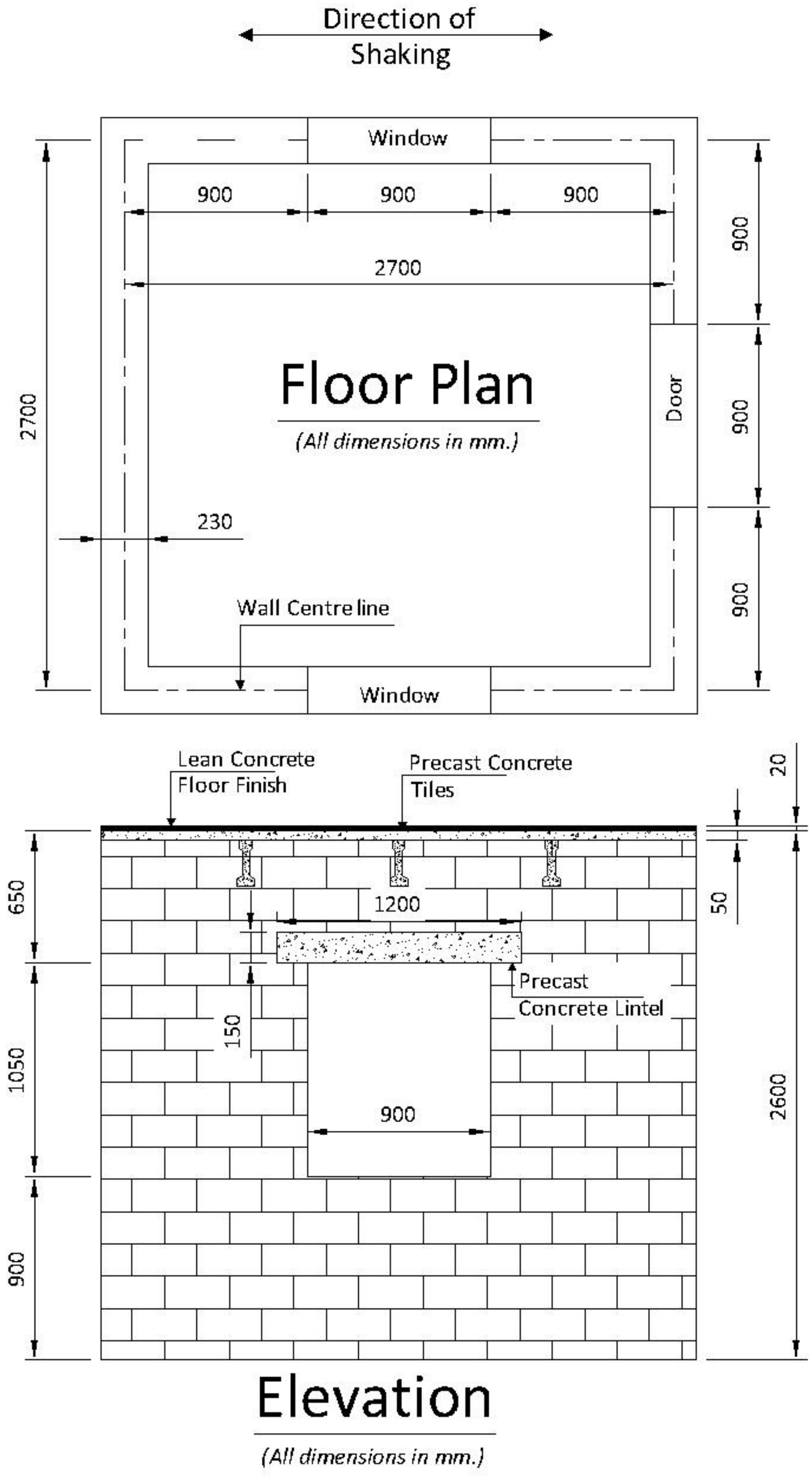
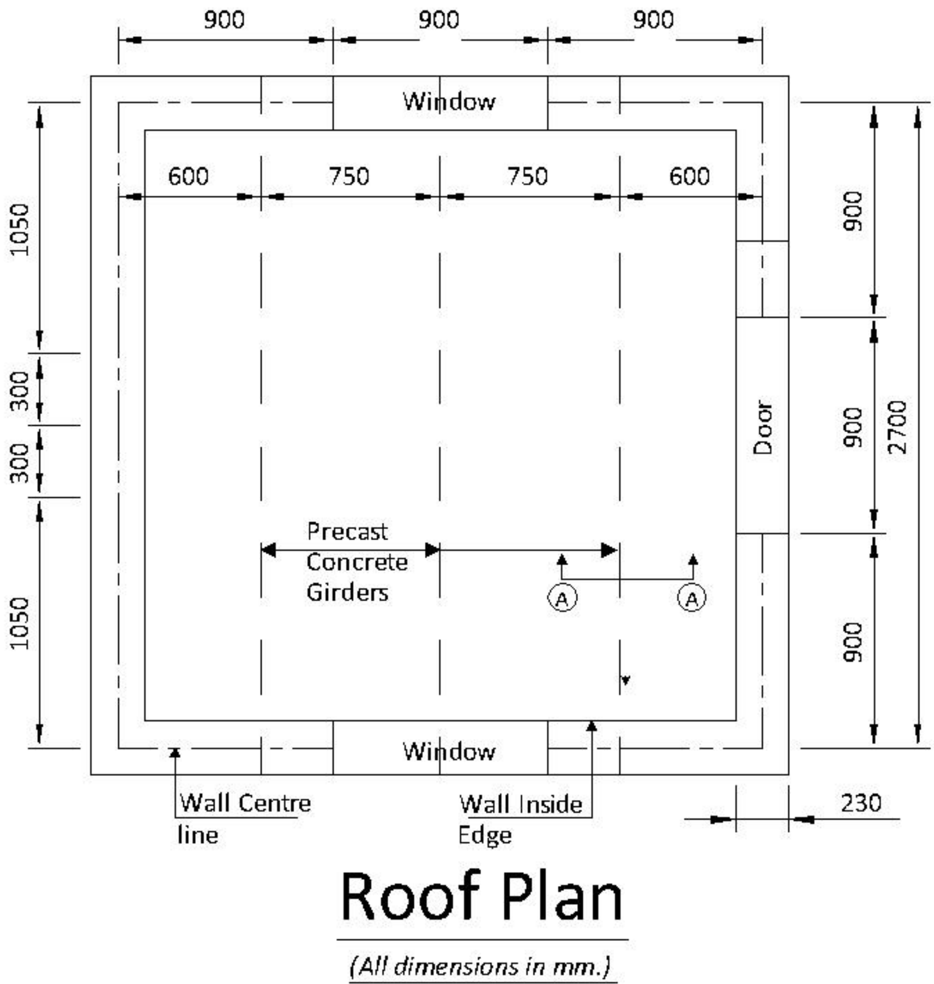


Figure 7.6: Floor Plan and Elevation for model Room-2



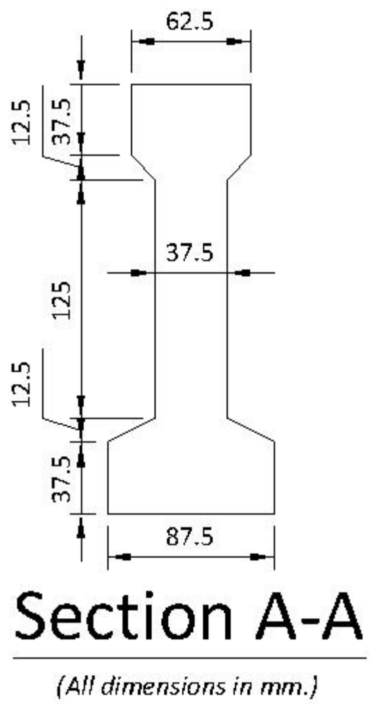


Figure 7.7: Roof Plan and enlarged cross section of roof girder for Room-2

Figure 7.8: Orientation of Room-2 on shake table

Masonry walls were reinforced with PP-bands spaced at approximately 230 mm vertically and horizontally. Apart from the 230 mm spacing criteria, PP-bands were provided at the edges of all openings and walls. The bands were put on the outside and as well as the inside of the room. Typical lengths of horizontal and vertical bands on the outside and inside of the structure are the same as described for Room-1. The observation of collapse process of Room-1 revealed the inefficiency of the connection mechanism. Holes in the PP-bands made by the insertion of steel screw severely affected the strength of bands and in an early failure of the bands at the connections. To avoid such weakness in the bands due to the holes made for fixing purpose, a new fixing detail was worked out.

The new technique involved using a steel strip of approximately 40 mm long with holes made on either ends for inserting screws. *First end* of the band was wrapped on the steel strip between the two holes and attached to the wall using 25 mm long steel screws as shown in Figure 7.9. *Second end* was pulled manually to provide tension in the bands to align them and remove any looseness. The pull was applied manually after passing it under a steel strip already screwed to wall but not tightened to allow space for the band to pass through as shown by Figure 7.10. While keeping the tension in the band it was given a U-turn on to itself and then clamped to the wall using another steel strip a few inches away from the first one (Figure 7.9). This way the requirement to puncture the PP-band, for allowing the steel screw to pass through, was avoided. Cross over points for horizontal and vertical bands were attached to the wall using diagonally placed steel strips of length approximately 70 mm (Figure 7.10).

Figure 7.9: First end (*left*), Second end (*right*) revised connection detail for Room-2

Figure 7.10: Applying tension in PP-bands to remove looseness before fixing Second end

Instrumentation:

Accelerometers to measure the response acceleration were mounted on all four top-corners of the room. String pots to measure response displacement were attached to only one wall, which had its weaker plane parallel to the direction of shaking. Six string pots were provided at three levels; two at roof level, two at lintel level and two at sill level as marked in Figure 7.11. The sampling frequency of string pots was 100 Hz and for accelerometer it was 200 Hz.

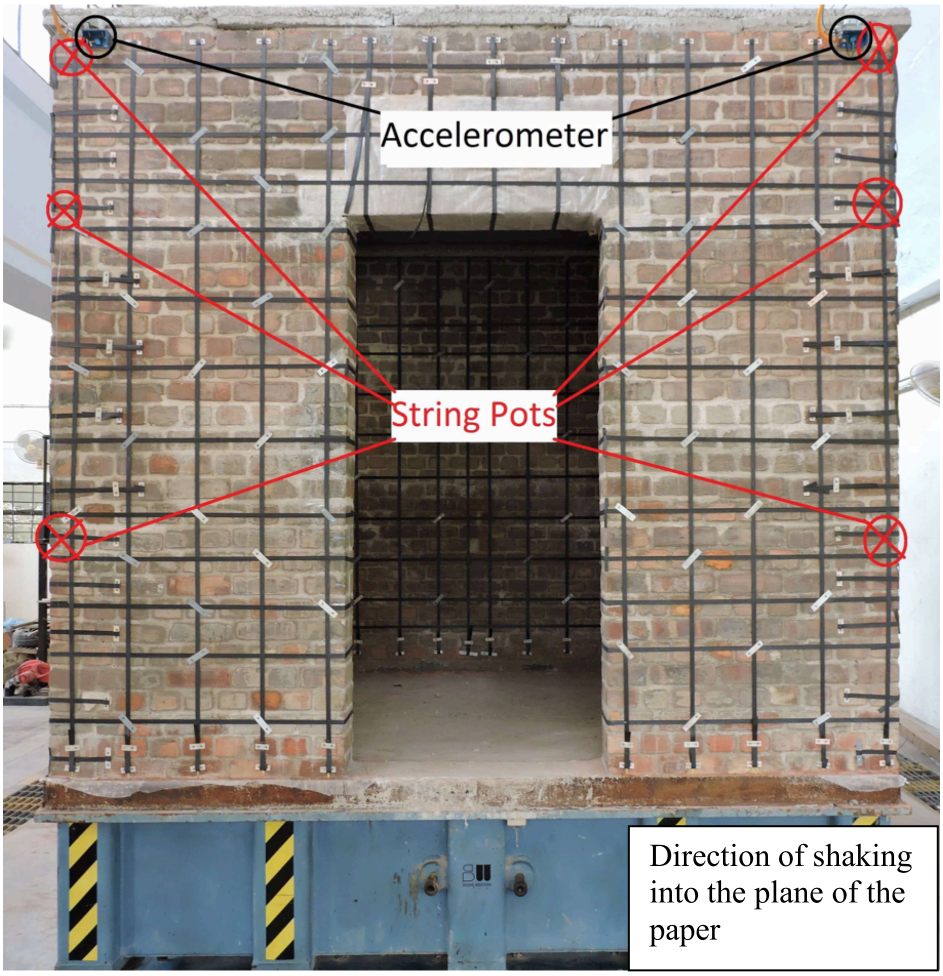


Figure 7.11: Instrumentation for Room-2

System Identification Tests:

Initially, *System Identification Tests* were carried out to determine the dynamic properties of the structure before subjecting it to stronger shake table vibrations to assess the performance of the PP-band retrofit during collapse. System identification tests are non-destructive tests in which the amplitude of the vibration waves was kept low to avoid damage to the specimen. Tests performed to determine the dynamic properties of the structure follow the same procedure as that mentioned for shake table test of wallettes in chapter-6.

Performance Evaluation and Failure Tests:

After performing system identification tests, the vibration intensity of the shake table was gradually increased till the point of crack initiation and the subsequent collapse of structure. For evaluating its performance, the structure was subjected to increasing amplitudes of sine waves at frequencies of 15 Hz and 5 Hz. Elcentro time history records with 50%, 100%, 150% and 200% amplitude magnification were also tried.

7.4 Results

Room-1

Performance Evaluation and Failure Tests

Performance evaluation tests for PP-band retrofitted masonry room reveals the significance of PP-bands in delaying the structural collapse. Due to the premature cracking of the specimen the system identification test could not be performed, however, the collapse mechanism of the room was recorded by video cameras for performance evaluation and the sequence of waves run are given in Table 7.1.

The assessment of masonry performance is based on the accumulation of arias intensity of each successive wave run after the initiation of cracks rather than the peak acceleration amplitudes on individual waves as discussed earlier in the analysis of shake table test for wallettes. Figure 7.12 shows the evolution of damage for PP-band retrofit room specimen with respect to the cumulative arias intensity of waves to which the specimen was subjected to after the initiation of cracks. To give a comparison of the performance of a similar non-retrofitted room tested under similar conditions two further curves (blue and green) are plotted in Figure 7.12. The prediction is based on the factors previously determined when concurrent shake table test study was conducted on retrofit and non-retrofit wallette specimens. The blue curve in Figure 7.12 is based on the actual test results of non-retrofit wallette specimen and the green curve is based on the previously mentioned assumption of equal peak strength and grade-3 initiation values for retrofit and non-retrofit wallette specimens. To achieve the predicted non-retrofit behaviour the cumulative arias intensity values starting from the point of initiation of grade-3 were reduced using the previously determined factors of 14.3 and 7.4 for actual test results of non-retrofit specimen and the scaled up results to match the peak strength of retrofit specimen, respectively. The cumulative arias intensity taken by retrofit specimen in grade-3 is

147.41 m/s where as for non-retrofit specimens predicted using 14.3 and 7.4 factor has 10.3 m/s and 19.92 m/s, respectively.

Table 7.1: Wave sequence for demolition of Room-1

S. No.	ω_g (Hz)	A_s (mm)	T_d (sec)	Wave Type	PGA (g)	a_{rms} (g)	I_a (m/sec)	EMS Grade 1998
1	0.5	50	20	sine	0.43	0.04	0.60	0
2	2	50	5	sine	1.39	0.44	15.19	3
3	15	1.3	6	sine	0.57	0.32	9.40	3
4	15	1.4	6	sine	0.58	0.33	9.84	3
5	15	1.5	6	sine	0.61	0.34	10.86	3
6	15	1.6	3	sine	0.64	0.38	6.88	3
7	15	1.7	3	sine	0.67	0.42	8.12	3
8	15	1.8	3	sine	0.77	0.45	9.44	3
9	15	1.9	3	sine	0.85	0.49	11.07	3
10	15	2	3	sine	0.88	0.52	12.88	3
11	15	2.2	3	sine	0.93	0.57	14.40	3
12	5	3	3	sine	0.16	0.02	0.21	3
13	5	3.5	3	sine	0.32	0.12	0.68	3
14	5	4	3	sine	0.41	0.16	1.12	3
15	5	4.5	3	sine	0.47	0.20	1.78	3
16	5	5.5	3	sine	0.53	0.22	2.22	3
17	5	6	3	sine	0.54	0.25	2.92	3
18	5	7	5	sine	0.55	0.27	5.47	3
19	5	8	5	sine	0.59	0.30	6.80	3
20	5	9	5	sine	0.70	0.33	8.53	3
21	5	10	5	sine	0.75	0.36	10.36	3
22	2	15	5	sine	0.51	0.17	2.27	3
23	N/A	N/A	35	Elcentro-200%	0.91	0.14	12.14	3
24	N/A	N/A	35	Elcentro-200%	0.91	0.14	12.14	4
25	N/A	N/A	35	Elcentro-200%	0.91	0.14	12.14	4
26	N/A	N/A	35	Elcentro-200%	0.91	0.14	12.14	4
27	5	12	5	sine	0.85	0.41	12.76	5
28	5	13	3	sine	0.98	0.49	11.27	5
29	5	13	3	sine	1.10	0.52	12.92	Collapse

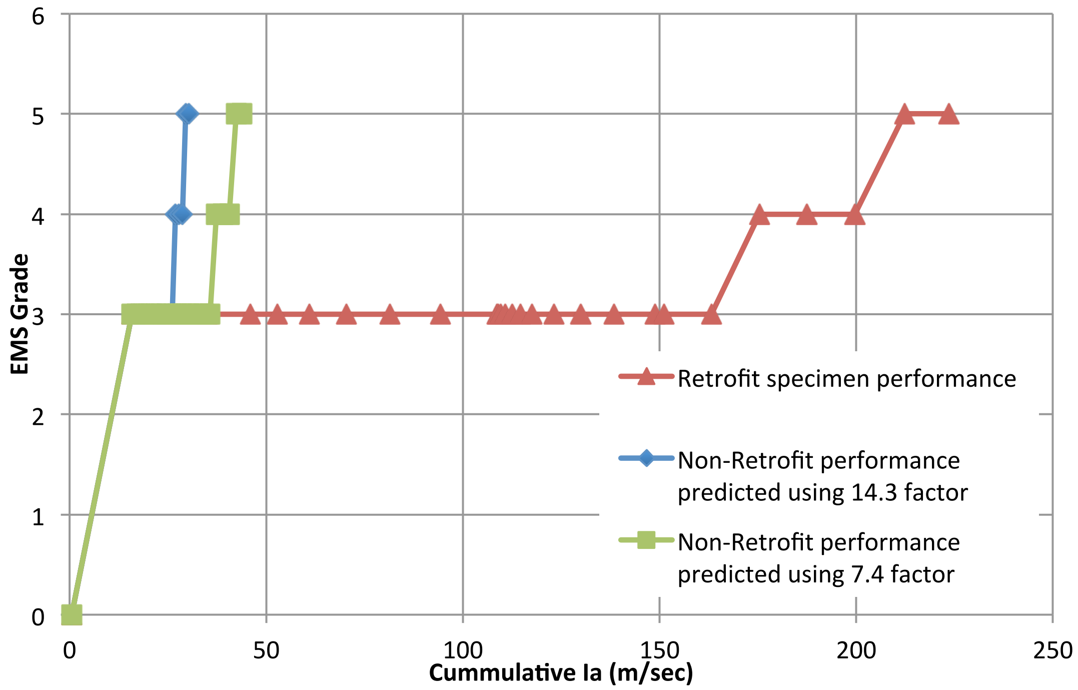


Figure 7.12: EMS Grade vs. Cumulative Arias Intensity for Room-1

The initiation of grade-3 damage was marked by the total loss of structures lateral load carrying capacity due to the cohesion of mortar. The cracks were sudden and from almost no cracks the specimen moved to grade-3 condition in wave-3 where the recorded PGA was 1.39 g. This is due to the brittle nature of masonry but majorly due to the abrupt increase in the PGA from 0.43 g in wave-1 to 1.39 g in wave-2. The cracks were fully developed in wave-2 running from one edge of the wall to another. Walls in plane to the direction of shaking had a window opening, which played a vital role in determining the location and orientation of cracks. Cracks originated from all four corners of the opening and moved diagonally upward in case of top corners and diagonally downward in case of bottom corners of the window (Figure 7.13). The wall with door opening with its stronger plane perpendicular to the direction of shaking showed out of plane flexural cracks with vertical and horizontal orientation (Figure 7.13). Similarly the solid wall opposite to the wall with door opening showed similar out of plane flexural crack pattern along with evident indication of masonry slightly being pushed out (Figure 7.14).

Figure 7.13: Cracks in Room-1 after the initiation of Grade-3 damage in wave-2

Figure 7.14: Cracks in solid wall of Room-2 at initiation of grade-3 damage

The initiation of cracks in wave-2 also had damaging effects on the PP-bands due to the shortcomings in the connection technique selected during application. Figure 7.15 gives a picture of PP-bands rupture, and it can be clearly seen that due to the insertion of steel screws through the band to tighten them against wall had affected the strength and performance of the bands during dynamic vibrations. Holes created in the bands by the insertion of steel screws created a stress concentration zone with potential weakness for rupture during lateral vibrations. This shortcoming observed in the performance of PP-bands urged the need to have a different connection detail where the performance of the bands is not hindered by the connection method. Had the PP-bands been allowed to perform at their material yielding capacity without sustaining any ruptures, the overall integrity of the structure would have been well maintained to higher levels of ground vibrations.

Figure 7.15: Rupture of PP-bands due to the selected connection technique

Even with the flaw in connection detail the PP-bands did provide a sufficient level of integrity to the structure by delaying its collapse. Pictures of structure progressive damage starting from wave-23 of Table 7.1 are given in appendix.

<u>Room-2</u>

<u>System Identification:</u>

a) *Hammer test:*

The power spectral density plots of the specimen response measured by the inline response monitoring sensors reveal a frequency of around 35Hz for Room-2. The value for natural frequency is based on the average of the three hammer trials and the response measurements from all four accelerometers. Logarithmic decrement method applied to the free vibration response from hammer blow reveals a damping ratio of 9% for Room-2.

b) *Resonance test:*

Sine waves run on the structure to record responses showed no resonance below 20 Hz. First resonance recorded was at 20 Hz where the response amplitude was twice the input wave acceleration amplitude. 25 Hz recorded lesser increase in the response acceleration amplitudes as compared to the 20 Hz, but then again at 30 Hz the increase of response acceleration amplitude was again twice the input acceleration amplitudes. Therefore, it was not possible to accurately judge the natural frequency of the structure but it gives an idea of the range of frequencies where the structures natural frequency may lie, which is above 20 Hz and around 30 Hz.

Performance Evaluation and Failure Tests:

In terms of performance the structure itself was quite strong and sustained vibrations of up to 0.91 g PGA without showing any cracks due to the box action of 230 mm thick masonry walls. At a PGA of 0.99 g, which is the 1st wave in the table, cracks started to appear in the walls marked as Grade-1/2 on EMS damage scale. Even after the occurrence of cracks the structure remained completely intact when subjected to Elcentro record with 200% magnification. Movie recordings of the test reveal the action of subsequent waves trying to widen the cracks in masonry which was held together by the presence of PP-bands, thus preventing crack widening and delaying the subsequent collapse of the structure or any part thereof.

Table 7.2 shows the sequence and necessary information about the waves that were run for performance evaluation and failure tests. Waves starting from the crack initiation are shown in Table 7.2 and the ones tried before those are omitted for the conciseness of table.

Table 7.2: Wave sequence for shake table test of Room-2

S. No.	ω_g (Hz)	A_s (mm)	T_d (sec)	Wave Type	PGA (g)	a_{rms} (g)	I_a (m/sec)	EMS Grade 1998
1	5	8	5	Sine	0.99	0.41	12.94	1/2
2	N/A	N/A	35	Elcentro-200%	0.91	0.14	12.14	3
3	5	8	5	Sine	0.78	0.37	10.43	3
4	5	10	5	Sine	0.84	0.38	11.39	3
5	5	12	5	Sine	1.20	0.48	17.97	3
6	5	14	5	Sine	1.18	0.56	24.06	3
7	5	16	5	Sine	1.35	0.62	29.39	3
8	5	18	5	Sine	1.59	0.71	39.24	3
9	5	20	5	Sine	1.77	0.82	51.39	3
10	5	22	5	Sine	1.8	0.86	56.88	3
11	5	20	5	Sine	1.65	0.78	46.62	3
12	5	25	5	Sine	2.00	0.96	71.97	4
13	5	40	5	Sine	3.19	1.43	157.95	5
14	5	50	5	Sine	3.85	1.61	166.06	Collapse

The ground motion parameters and their notations used in Table 7.2 are the same as those explained in Chapter-6. Here again the focus of analysis would be based on the arias intensity parameter rather than the acceleration amplitudes. It should be mentioned here that due to extra ordinary performance of PP-bands in preventing the structure collapse the bands had to be cut at the end of the day to speed up the collapse. The bands were cut after wave-10 of Table 7.2.and with subsequent waves the structure's damage condition on EMS scale increased and eventually the structure collapsed. The next wave after cutting the bands, which is wave-11, had a lesser acceleration amplitude as compared to the last wave. This was done to remove any residual stresses in masonry due to the presence of PP-bands.

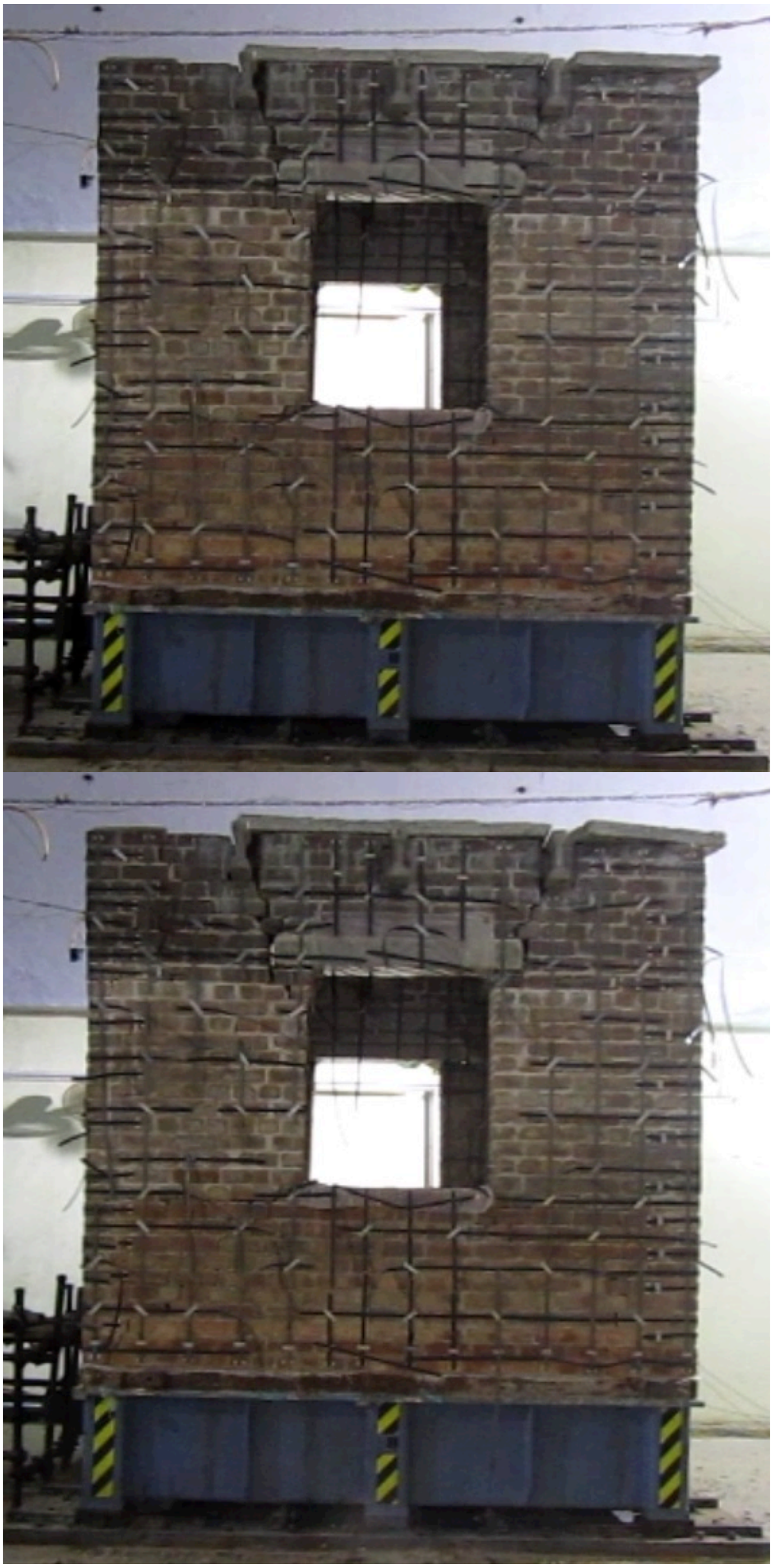


Figure 7.16: Room-2 before (*top*) and after (*bottom*) wave-11

Although the structure stayed in grade-3 on wave-11 but the screen shots of video recording shows slight increase in the previously present crack widths. A careful look at Figure 7.16 shows that the left edge of the room has slightly moved away after being subjected to wave-11. Also the two cracks propagating from the two ends of the window lintel towards the roof have greater width evident from a darker crack line in the left figure of Figure 7.16. This shows that although the intensity of shaking was low but due to the destructive potential of the input wave characterized by the arias intensity had contributed its effect in damaging the structure and also how the PP-bands have been delaying this effect previously.

Accumulation of arias intensity of the input waves against EMS grade is shown in Figure 7.17. The graph along with the values of tested retrofit specimen also gives predicted performance curves for non-retrofit room if tested under similar conditions. The two separate lines for predicted non-retrofit room specimen are due to the two different factors worked out and explained previously in shake table test of wallettes. The method for obtaining the two predicted curves is the same as mentioned previously for Room-1. The cumulative arias intensity taken by retrofit specimen in grade-3 is 287.36 m/s where as for non-retrofit specimens predicted using 14.3 and 7.4 factor has 20.1 m/s and 38.83 m/s, respectively.

In comparison to the results of Hanazato et al. the structure with PP-band showed remarkable integrity in post cracking stage. Test conducted by Hanazato et al. had crack widening in the very next wave after crack initiation and collapsed in the third wave [130]. Their structure initiated cracks at 1.7g of pulse shocks and sustained accelerations of 1.5-2g with heavy damage and eventual collapse. Whereas, the structure retrofit with PP-bands although initiated cracks at 0.9g due to weaker bond strength but sustained waves of 2g without any crack widening or structure deterioration.

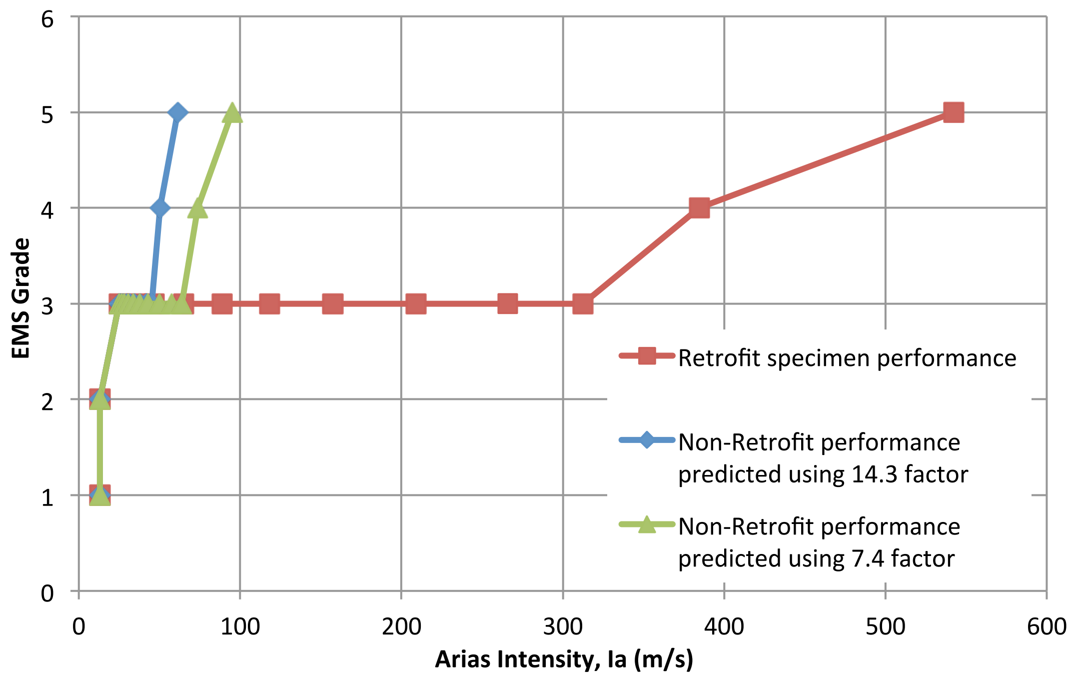


Figure 7.17: EMS grade vs. Cumulative arias intensity for Room-2

7.6 Concluding Remarks

Shake table test of real scale masonry room was conducted to demonstrate the performance of PP-bands in prolonging the collapse mechanism of a real structure. Although the wallette specimens were built using real scale masonry units with English bond but they do not represent the actual behaviour of a real structure because of the difference in mass and size, the cracks in wallettes were mostly flexural or shear slips, whereas, in real masonry structures shear diagonal cracks dominate the crack pattern of in-plane walls. Also, testing a complete structure would give the performance of wall joints and the out-of-plane walls.

Two separate retrofit rooms tested on shake table are presented in this chapter because the second wave had very high acceleration intensity and created a premature grade-3 condition in the first room before any system identification tests could be carried out. But, majorly the test had to be repeated because the increase in the acceleration amplitude was sudden as opposed to FEMA guidelines where the increase in acceleration amplitude should be gradual and should be approximately 25% of the last run wave amplitude [120]. Therefore, grade-1 and grade-2 were not achieved and the structure abruptly moved to grade-3 condition. Still subsequent waves had to run to cause collapse of the structure and the results presented are based on these waves.

Major focus of this test was again on the performance of PP-bands, which was recorded by video cameras. Room-1 test revealed the weakness in the connection detail used for fixing PP-bands on the wall. The main source of this weakness was due to the insertion of steel screws through the centre of the band, which created a hole in the band and provided a potential weak zone of high stress concentrations. As PP-bands are made of polypropylene fibres stacked together in the longitudinal direction therefore any hole within the band caused the fibres to split length wise under the action of lateral loads and thus reducing the efficiency of the bands. Even with this drawback the structure sustained very high levels of ground vibration such as four El Centro records with acceleration amplitudes magnified to 200%. The intermediate connections provided at the intersection of horizontal and vertical bands played an important part in prolonging the collapse of the structure.

Room-2 was constructed to test the performance of PP-bands but with a different connection detail that should not hinder the efficiency of bands. The bands eventually had to be cut to bring the structure to gradual collapse. Performance of the PP-bands had been proven to maintain structural integrity to very high levels of ground vibrations of up to 2g, which is a rare occurrence in reality. Therefore, it was deemed appropriate to cut the bands and witness the subsequent collapse. After the bands were cut the cracks in the masonry started to widen and the structure moved to grade-4 damage condition. The last two waves were run at high acceleration amplitudes to speed up the collapse due to the excessive heating of shake table actuator. The new connection detail registered higher performance for the Room-2 with accumulated arias intensity in grade-3 being 287 m/s while for Room-1 it was 147 m/s. The study from shake table test will help future masonry to effectively implement PP-band retrofit against seismic hazards. The new connection detail used for model masonry structure showed remarkable performance where the bands stayed intact up to significantly high levels of ground accelerations and thus structural integrity was maintained.

Chapter 8 : Numerical Study

8.1 Numerical Modelling

Wall specimens tested in the lab (*Chapter-3)* were modelled in '*Abaqus*' (Finite Element Solver) using Python scripting. '*Python*' [131] is a general-purpose high level programming language used for developing Abaqus. Using Python helps to save considerable time on tasks that are time consuming and repetitive especially in case of meso-modelling where a large number of bricks with their corresponding interactions with adjacent bricks had to be defined. With the use of algorithms in Python the time for modelling was reduced considerably. Python also allows to randomly distribute weaker mortar joints across the wall to replicate actual masonry conditions. Two models have been attempted – one a macroscopic model and another microscopic model.

8.1.1 Macroscopic Model

In the macroscopic model the wall was modelled as a homogenous shell element with material properties of MDF. A Python code was developed which modelled the test setup discussed earlier in Chapter-3. The model assumed no damage taking place and did not account for plastic strains.

a) Model Properties

The details of wall model for small scale tests of Chapter-3 are given below:

Wall Size = 480 mm × 288 mm

Brick material – 'MDF'

E_{MDF} = 4 GPa

ν_{MDF} = 0.25

ρ_{MDF} = 750 kg/m^3

Applied *Push* = 1 mm/sec

Mesh type = CPS4 (continuum plane stress 4-noded full integration)

Element shape = QUAD (Quadrilateral)

Normal Interaction property with base or restraining bricks:

Interaction Type = "Normal Behaviour"

Pressure Overclosure = "Hard Contact"

b) Model Description

The macroscopic model in the current study runs within the material elastic limit. To achieve the cracking of wall, material properties need to be modified with the introduction of a damage model for MDF based on traction separation law used in mesoscopic models of *Section 8.1.2*. Figure 8.1 shows the screenshot of macroscopic numerical model from 'Abaqus' developed to replicate the test setup of Chapter-3. Brick provided at the bottom-left corner of the wall, named *'Push Brick'*, is restrained for any rotation and vertical translation, and moves towards right at a speed of 1 mm/s for 8 seconds to apply displacement push to the wall. Bricks named *'Fixed Restraints'* provided at the top-right corner of the wall assembly are restrained in all six degrees of freedom and restrict wall motion in vertical and horizontal directions. *'Fixed Base'* is provided as the base for the wall and is rigidly restrained in all six degrees of freedom. Normal *'hard contact'* free of any cohesion or friction is provided between the wall and the Push Brick, Fixed Restraints and Fixed Base.

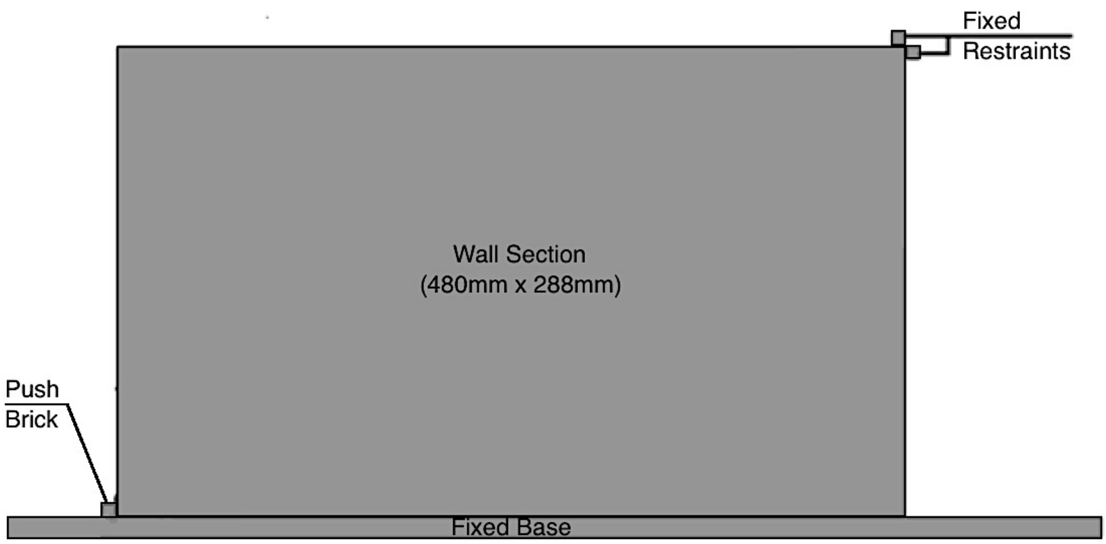


Figure 8.1: Screen shot from Abaqus for macro-model of wall

c) Mesh Sensitivity Analysis

For mesh sensitivity analysis four mesh sizes were tried, which are 6 mm, 3 mm, 2 mm and 1 mm. The resultant reactions of the restraining bricks at the top right corner for various mesh sizes are shown in Figure 8.2. Comparison of reaction forces suggests no significant effect on the results with regards to mesh size.

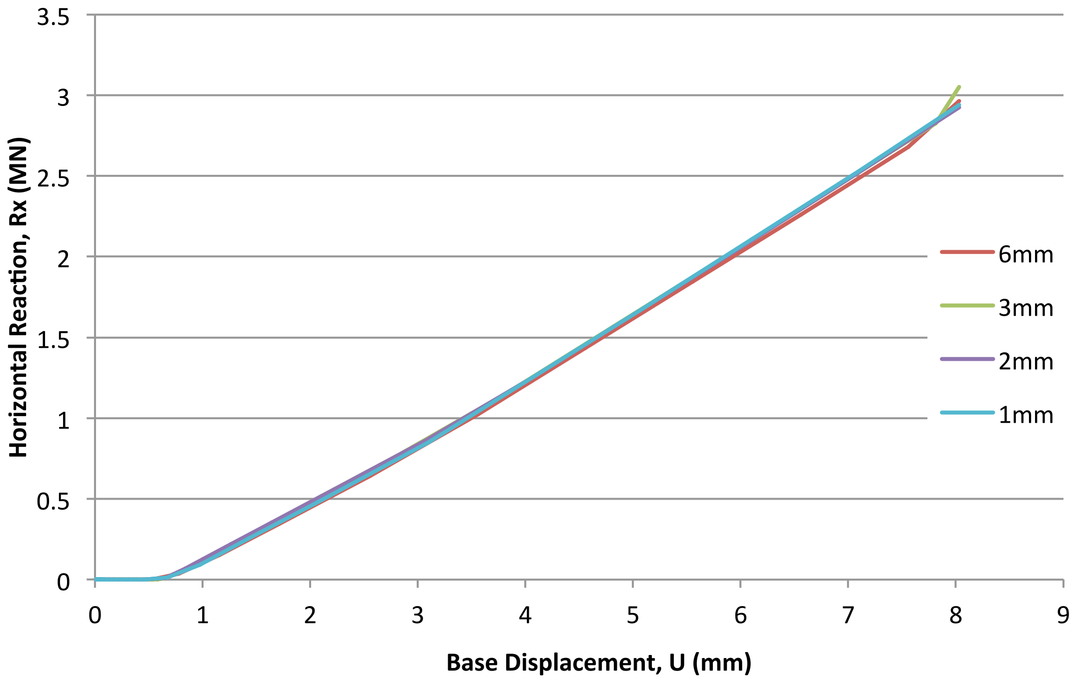


Figure 8.2: Reaction vs. base displacement for various mesh sizes in macro-model

A comparison of the *System Time*, which is the time taken to execute the kernel code by the system, for different mesh sizes is represented by bar chart in Figure 8.3. Computational times for analysis increase logarithmically with increasing number of elements in the wall.

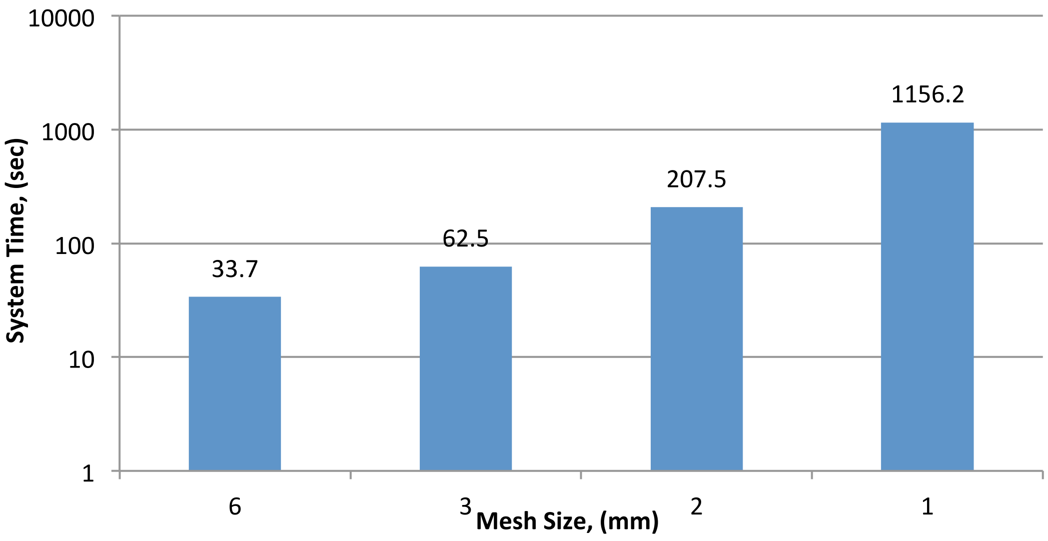


Figure 8.3: System time for various mesh sizes in macro-model

Von misses stresses measured at the centre of the wall for all mesh sizes is given in Figure 8.4. Stress values increase with finer mesh size. Assuming that finer mesh size gives more accurate results then the result of 1mm mesh size should be the most desirable. However, due to the high computation cost of 1 mm mesh it is not advisable to use this mesh size. The stress values for 2 mm and 3 mm mesh size are almost equal and it should be advisable to use 3 mm mesh due to lesser computational time.

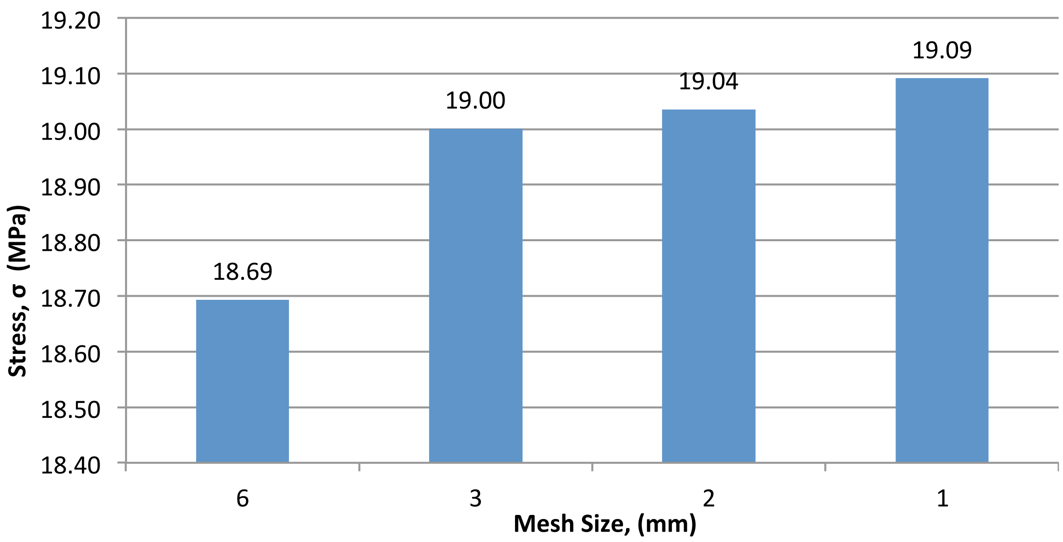


Figure 8.4: Von misses stresses at the centre of the wall in macro-model

Point displacements measured for different mesh sizes at bottom-left corner of the wall towards the end of 8 mm push analysis are given in Figure 8.5. As the total displacement applied at the bottom left corner by push-brick is 8 mm, the mesh size of 1 mm gives the closest value to 8 mm.

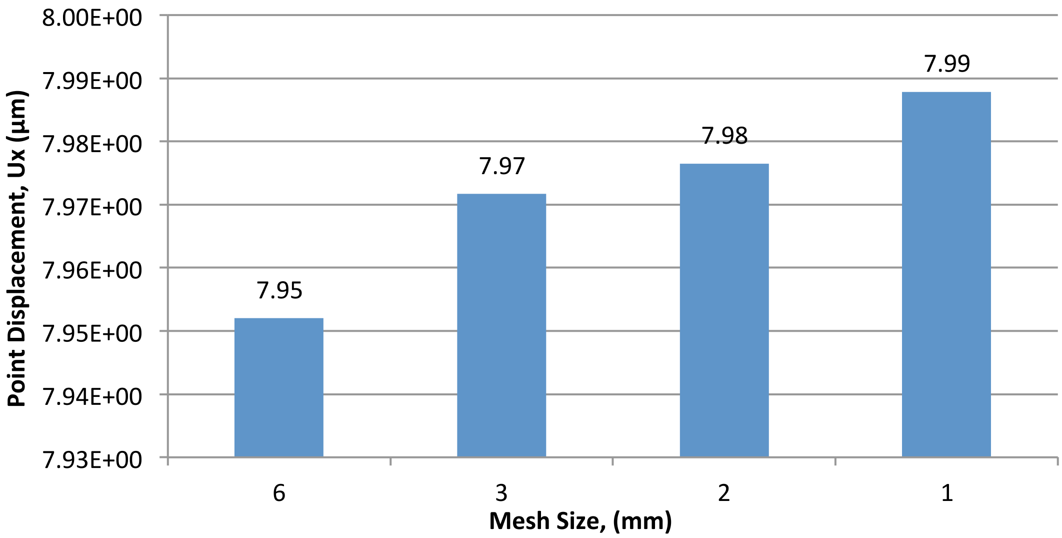


Figure 8.5: Horizontal Displacement at bottom left corner of wall in macro-model

The percentage difference of 3 mm and 2 mm mesh size value for displacement of bottom-left corner in comparison to 8 mm base push is 0.35% and 0.3%, respectively. The percentage increase of computational time from 3 mm to 2 mm mesh size is 232%. Thus it can be concluded that a mesh size of 3 mm would be most efficient in terms of computational cost and analysis results.

8.1.2 Mesoscopic Model

A 'Python' script was written to generate the mesoscopic model of the wall specimens tested in lab. The script generates the entire wall model with all the appropriate interactions assigned between the contacting surfaces of masonry units. The Python Script used for assigning interactions in meso-models along with necessary data input is given in 'Appendix-F'.

a) Model Properties

Mechanical values for Medium Density Fibreboard (MDF) taken from literature [132].

Wall Size = 480 mm × 288 mm

Brick Length = 24 mm

Brick Height = 12 mm

Brick Width = 12 mm

Brick Material = ‘MDF’

E_{MDF} = 18.9 N/mm^2

ν_{MDF} = 0.24

ρ_{MDF} = 650 kg/m^3

Application of *Push*= 1 mm/sec

Mesh type = CPS4R (Continuum Plane Stress 4-element Reduced)

Element shape = QUAD (Quadrilateral)

Interaction between bricks = “Cohesive+Friction+Damage”

Interaction between bricks and restraining elements = “Normal”

Friction Interaction property:

Friction formulation = PENALTY

Directionality = ISOTROPIC

Shear stress limit = none

Friction coefficient = 0.1

Cohesive Interaction property:

Normal Stiffness, K_{nn} = 10^{10} N/m (normal)

Shear Stiffness, K_{ss} = 10^{10} N/m (local element direction-1)

Tangential Stiffness, K_{tt} = 10^{10} N/m (local element direction-2)

Slave nodes (nodes of the deformable body, in this case, bricks) initially in contact

Damage Interaction property:

Initiation Criteria = “Quadratic Traction”

Maximum Nominal Stress – Normal ($\sigma°_n$) = 10^6 N/m^2

Maximum Nominal Stress – Shear 1 ($\sigma°_s$) = 10^6 N/m^2

Maximum Nominal Stress – Shear 2 ($\sigma°_t$) = 10^6 N/m^2

Damage Evolution Type = "Displacement"

Softening = "Linear"

Total/Plastic Displacement = 0.0002 m

Stabilization viscosity = 0.002 (viscosity Coefficient)

Eligible Slave Nodes = "Only slave nodes initially in contact"

Traction Separation Behaviour = "Specified Stiffness" – "Uncoupled"

Normal Interaction property:

Pressure Overclosure = "Hard Contact"

Constraint Enforcement Method = "Penalty"

Separation after Contact = "Allowed"

b) Cohesive and Damage Interaction

Interaction properties used are based on literature [132] and sensitivity analysis (Appendix-E) to determine values that give the most favourable behaviour of the model. To achieve a brittle contact between the brick surfaces, as observed during lab experiments, the stiffness of the interaction was kept higher than the young's modulus of the bulk material to ensure brittle crack opening in joints without excessive deformation [93].

The stiffness matrices in the two following equations of "*Traction-Separation*" show the difference between coupled and uncoupled stiffness.

$$\begin{Bmatrix} \sigma_n \\ \sigma_s \\ \sigma_t \end{Bmatrix} = \begin{bmatrix} K_{nn} & K_{ns} & K_{nt} \\ K_{ns} & K_{ss} & K_{st} \\ K_{nt} & K_{st} & K_{tt} \end{bmatrix} . \begin{Bmatrix} \delta_n \\ \delta_s \\ \delta_t \end{Bmatrix} \qquad Eq.\ 8.1$$

$$\begin{Bmatrix} \sigma_n \\ \sigma_s \\ \sigma_t \end{Bmatrix} = \begin{bmatrix} K_{nn} & 0 & 0 \\ 0 & K_{ss} & 0 \\ 0 & 0 & K_{tt} \end{bmatrix} . \begin{Bmatrix} \delta_n \\ \delta_s \\ \delta_t \end{Bmatrix} \qquad Eq.\ 8.2$$

where, *n, s, t* represent normal, shear and tangential directions respectively, "σ" is the traction stress, "*K*" is the interface stiffness or *Penalty Stiffness* in Abaqus and "δ " is the separation. Coupled stiffness means that traction-separation in one direction is also affected by the stiffness in the other two directions as well. However, in the current study uncoupled stiffness is considered, which means that traction in one direction is not affected by the traction in other two directions.

Damage initiation for cohesive interactions is based on the *quadratic separation criterion* which is as given by the following expression:

$$\left[\frac{\langle\sigma_n\rangle}{\sigma^\circ{}_n}\right]^2 + \left[\frac{\sigma_s}{\sigma^\circ{}_s}\right]^2 + \left[\frac{\sigma_t}{\sigma^\circ{}_t}\right]^2 = \mathbf{1} \qquad \textit{Eq. 8.3}$$

where, parenthesis on 'σ_n' is called Macaulay brackets used to ignore the values in compressive stress and 'σ°' represent the maximum nominal traction stress. The damage is initiates when the sum of the terms on right hand side reaches or exceeds unity.

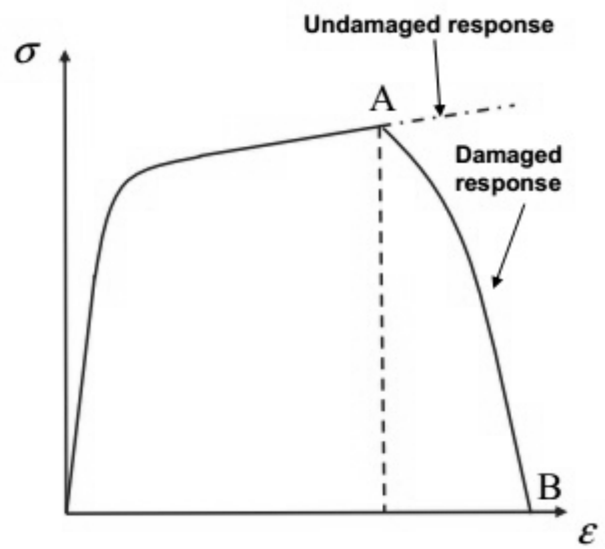


Figure 8.6: Progressive damage of a material [133]

Figure 8.6 defines the concept of damage initiation and damage evolution. The graph till point-A is normal stress-strain curve for elastic-plastic material and point-A represent is the point of damage initiation. The part of curve from point-A to point-B is damage evolution or also known as softening, which could either be linear, exponential or user defined in Abaqus [134]. The damage evolution progresses from the point of maximum load taken by the material to the point where it can no longer take any load, in other words the degradation factor "*D*" goes from 0 to 1 in the following expression [134]:

$$\sigma = (1 - D)\bar{\sigma} \qquad \textit{Eq. 8.4}$$

where, 'σ' is the stress in material due to damaged response and '$\bar{\sigma}$' is the stress due to undamaged response.

The evolution of damage under a combination of normal and shear separations is defined through an *effective separation* given by the following expression:

$$\delta_m = \sqrt{(\delta_n)^2 + {\delta_s}^2 + {\delta_t}^2} \qquad \textit{Eq. 8.5}$$

Degradation factor "*D*" for linear softening at any point after the damage initiation is calculated using following expression:

$$D = \frac{\delta_m^f(\delta_m^{max} - \delta_m^{\circ})}{\delta_m^{max}(\delta_m^f - \delta_m^{\circ})} \qquad \textit{Eq. 8.6}\ [134]$$

Where, 'δ_m^{max}' is the maximum separation experienced during the loading history, and 'δ_m°' and 'δ_m^f' are the effective separations at damage initiation and failure as shown in Figure 8.7.

"Total/plastic displacement" mentioned earlier while defining damage properties is 'total displacement for elastic materials, and plastic displacement for bulk elastic-plastic materials'. In the current study total displacement 'δ^f' will be used for the damage evolution parameter. For finding total displacement 'δ^o' is calculated using equation 8.6 and then increase it to account for the softening curve (A-B).

$$\sigma^{\circ} = k\delta^{o} \qquad \textit{Eq. 8.7}$$

Total displacement obtained from equation 8.7 is 0.0001 m, which was doubled to 0.0002 m to account for the damage evolution curve (A-B) in Abaqus. Also this value for total/plastic displacement was based on sensitivity analysis to check which value gives appropriate crack pattern.

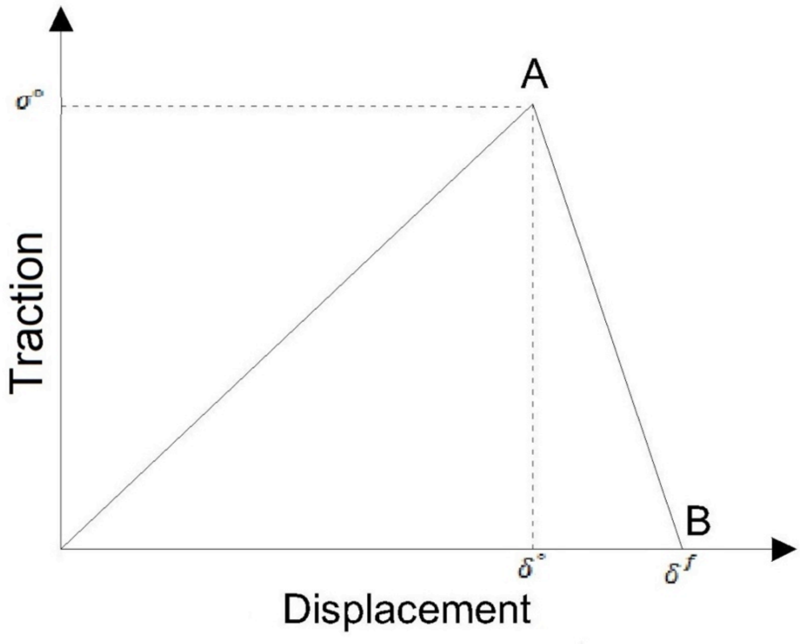


Figure 8.7: Explanation for *Total Displacement* value

c) Model Description

Brick units were modelled distinctively using deformable shell element. The model was prepared in two-dimensional plane and the out-of-plane direction was neglected for increasing the computational cost and efficiency of the model. Cohesion interaction was defined to imitate the effect of glue between the bricks. For every brick unit separate interaction was assigned for its contact with every other adjacent brick to avoid errors in computation as shown in Figure 8.8, where notation '*I*' represent interactions between bricks along with the subscript used to assign the number of interacting surfaces.

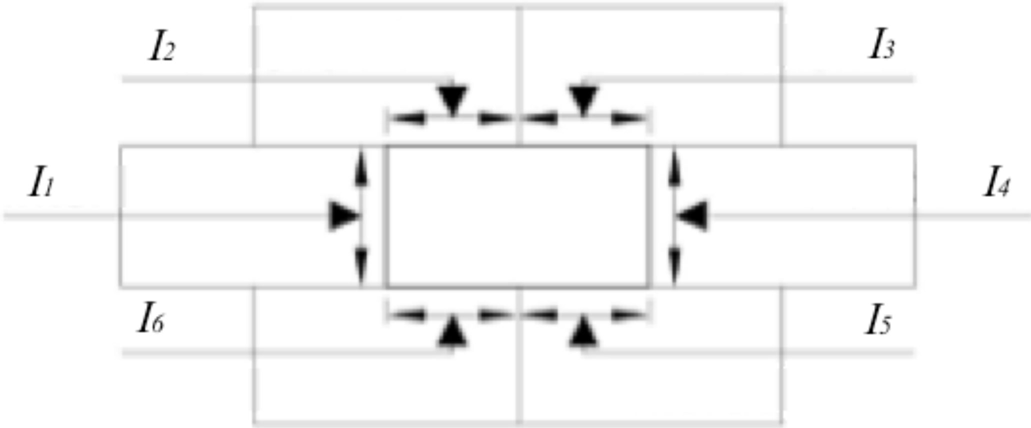


Figure 8.8: Assigning interactions for a single brick unit with adjacent bricks

Friction between brick joints is assigned to model the post failure sliding of bricks over one another. "*Normal interaction*" property is used to define the interaction between non-glued surfaces, where all default options are selected for hard type contact to prevent the penetration of elements into each other.

In reality, all the mortar joints in masonry do not have the same strength over the entire wall in fact there are material discrepancies randomly spread throughout the wall. Lab tests a unique crack pattern was observed every time, which was governed by the spread of the discrepancies in the joint strength. To account for such discrepancies in the joints two further damage interaction properties were defined. The original interaction property had 100% strength, whose properties are mentioned earlier in the properties section. Whereas, the others had 75% and 125% strength, which means that the values of maximum nominal strength for damage initiation and total/plastic displacement for damage evolution, mentioned in the properties section for 100% damage interaction, were altered accordingly. These interaction properties were randomly chosen while assigning the interaction between the brick surfaces, this allowed the discrepancy in joint strength to be spread randomly throughout the wall model. Every time the script was loaded in Abaqus the spread of these discrepancies was unique due to the random function and a unique crack pattern was observed every time for the particular random damage property choice.

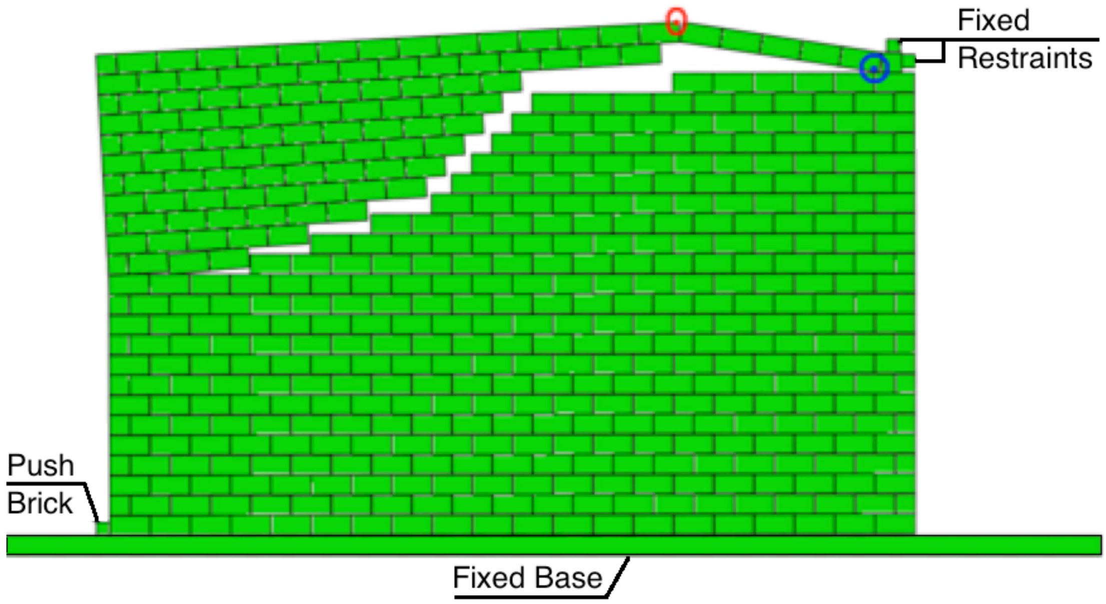


Figure 8.9: Screenshot of the mesoscopic wall model from Abaqus showing location of Point-1 (*Red*) and 2 (*Blue*)

Figure 8.9 shows the screenshot of microscopic numerical model from 'Abaqus' developed to replicate the test setup of Chapter-3. Brick provided at the bottom-left corner of the wall named *'Push Brick'*, is restrained for any rotation and vertical translation, and moves towards right at a speed of 1 mm/s for 8 seconds to apply displacement push to the wall. Bricks named *'Fixed Restraints'* provided at the top-right corner of the wall assembly are restrained in all six degrees of freedom and restrict wall motion in vertical and horizontal directions. *'Fixed Base'* is provided as the base for the wall and is rigidly restrained in all six degrees of freedom. Normal *'hard contact'* free of any cohesion or friction is provided between the wall and the Push Brick, Fixed Restraints and Fixed Base. Figure 8.9 also shows the location of point-1 and 2 that are later used in mesh sensitivity analysis to record and compare the displacements.

d) Mesh Sensitivity Analysis

Glued solid wall model is used for mesh sensitivity analysis to determine the optimum mesh size. Mesh sizes greater than 12 mm is not possible for brick elements. This is because the full-brick length is 24 mm and these bricks are further partitioned at the centre for defining interaction surfaces with the overlapping bricks in the top and bottom layers, as shown in Figure 8.8. Altogether four mesh sizes

(12 mm, 6 mm, 3 mm and 1.5 mm) were analysed and the results are presented for comparative study.

A review of the resultant force in the top right corner in Figure 8.10 and Figure 8.11 show that load-displacement curves for different mesh sizes closely follow one another up until the peak strength of the wall after which they start to diverge during the crack formation period but then gradually converge at the loss of walls load carrying capacity. 6 mm and 3 mm curves closely follow each other whereas the 12 mm mesh curve has slightly lower values and the 1.5 mm mesh curve has the least. However, at 5 mm base displacement these curves converge which means that mesh size affects the peak failure load for the wall and post failure behaviour but the loss of shear-strength is similar for all mesh sizes.

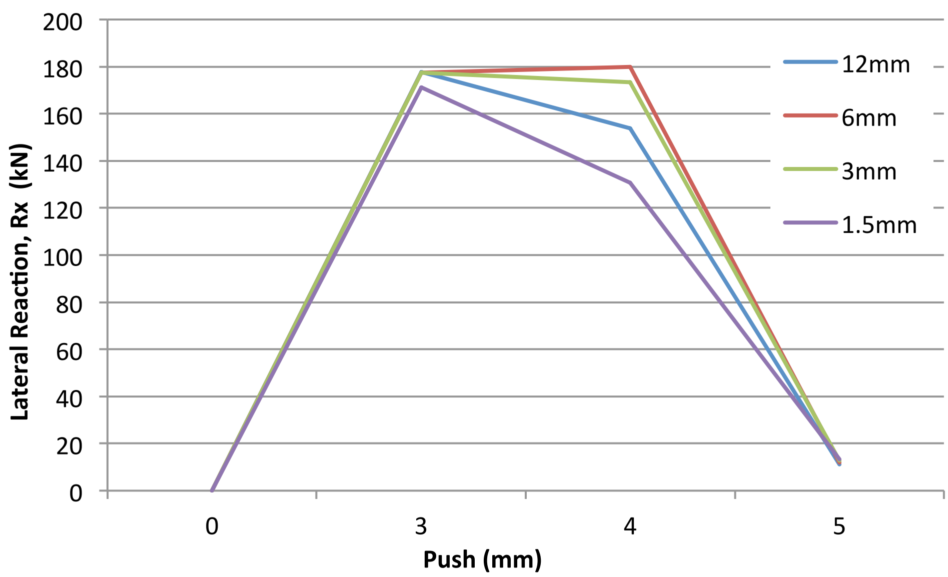


Figure 8.10: Lateral reactions for various mesh sizes in Mesoscopic Model

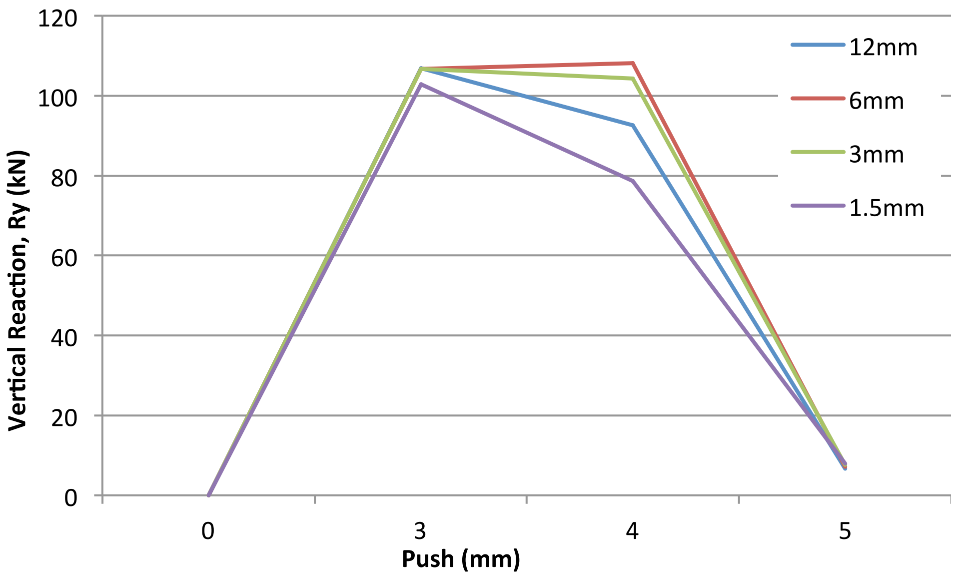


Figure 8.11: Vertical reactions for various mesh sizes in Mesoscopic Model

A comparison of computational times for different mesh sizes is shown in Figure 8.12. The bar chart suggests that mesh size less than 3 mm is computationally expensive especially if this modelling technique is to be extended to more complex geometries.

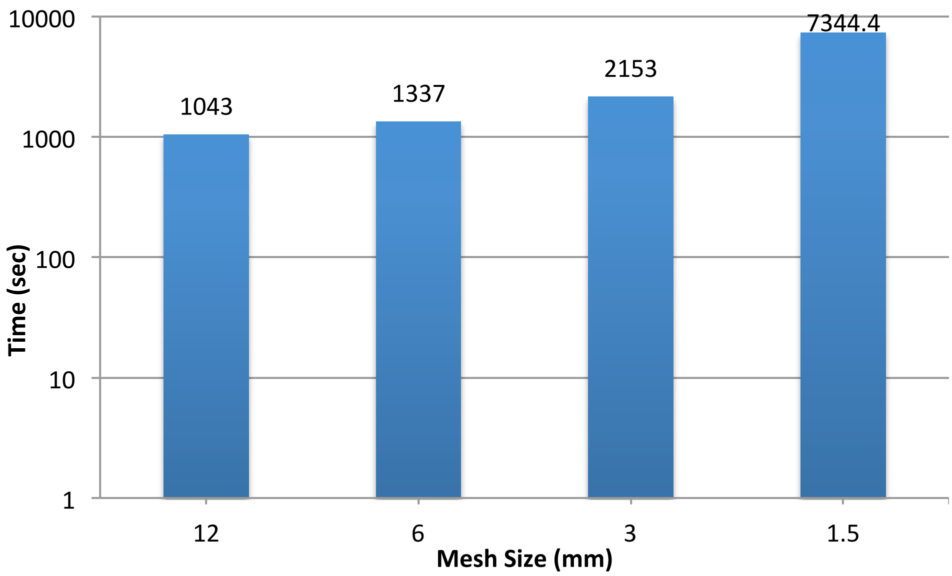


Figure 8.12: System Time for various mesh sizes in Mesoscopic Model

A comparison of lateral displacement for Point-1 and vertical displacement for point-2 is shown in Figure 8.13 and Figure 8.14. The graph is plotted for different mesh sizes as shown in the legend. The graph plots the horizontal and vertical displacement experienced by points 1 and 2 towards the end of analysis.

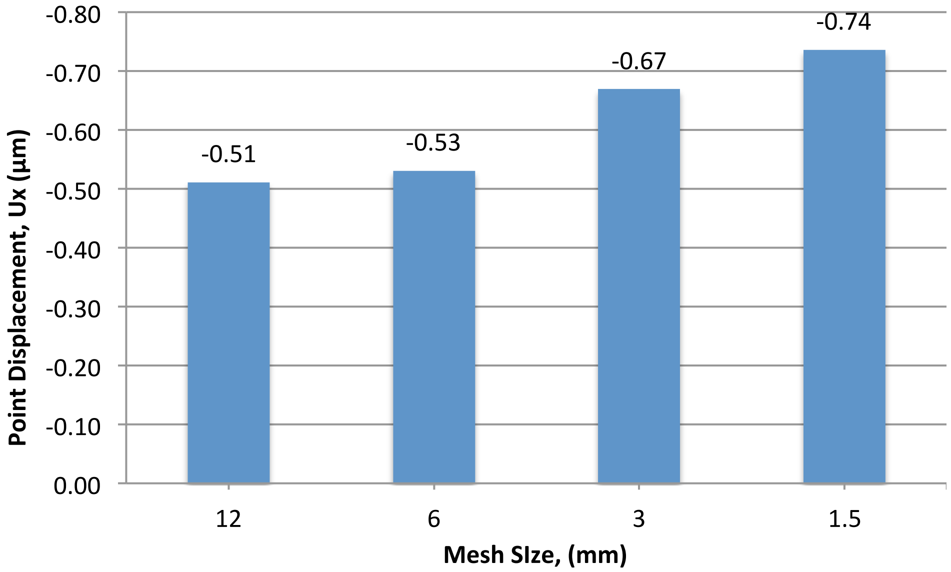


Figure 8.13: Lateral Displacement of Point-1 for various mesh sizes in Mesoscopic Model

The comparison of displacement suggests that finer mesh sizes yields higher values. To have accuracy in results a finer mesh need to be adopted but keeping in view the computational cost. Therefore, mesh size of 3 mm would yield the most accurate results in terms of displacement values and mesh size lesser than 3 mm is computationally expensive.

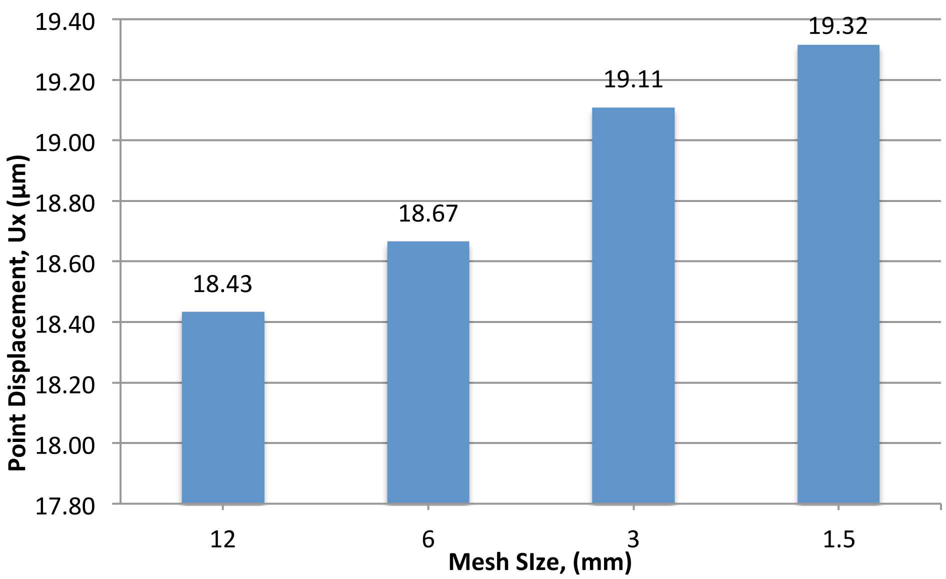


Figure 8.14: Vertical Displacement of Point-2 for various mesh sizes in Mesoscopic Model

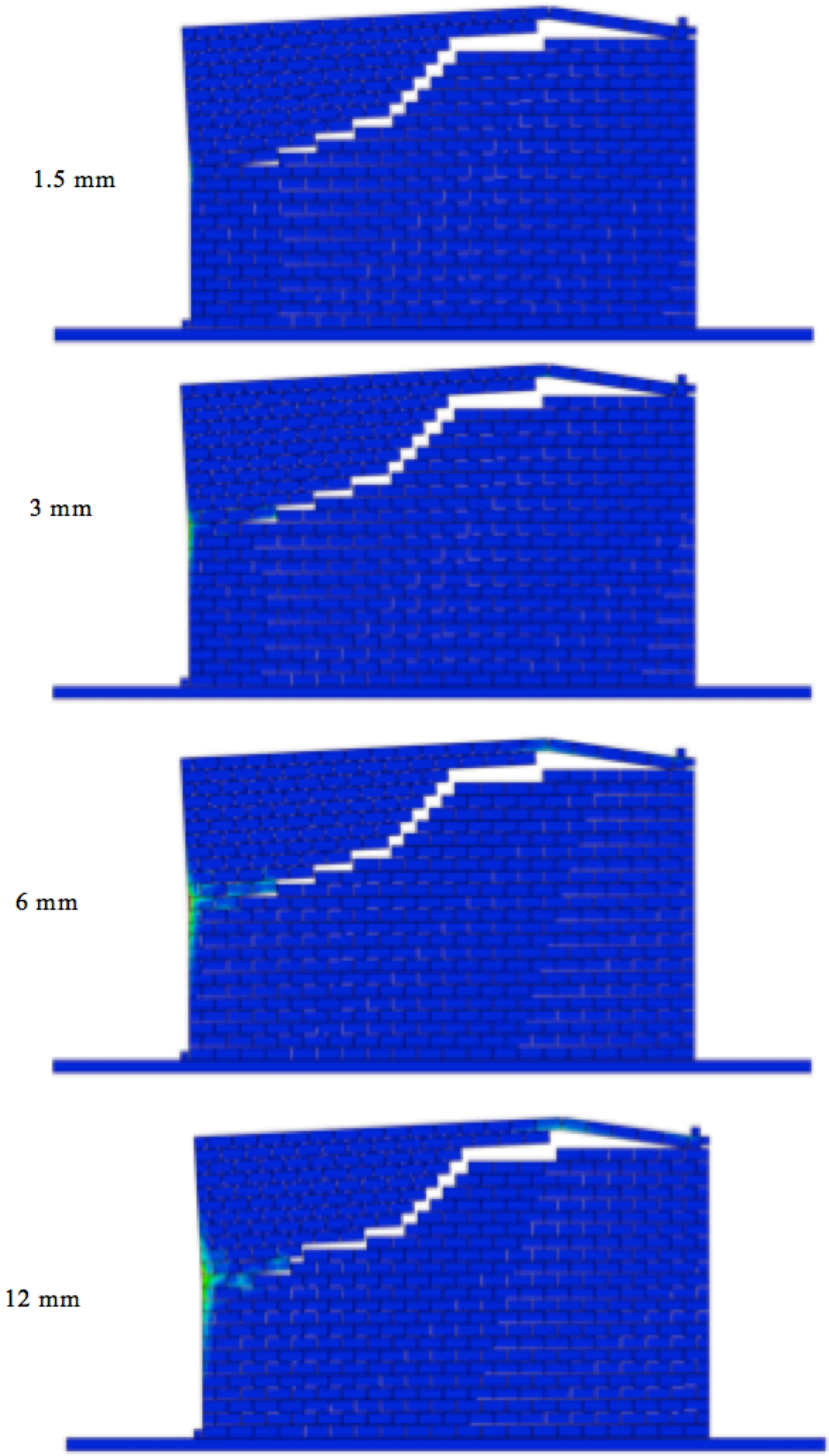


Figure 8.15: Similar crack patterns for shear wall even with different mesh sizes

The instance and location of crack occurrence and propagation show no sensitivity to the mesh sizes (Figure 8.15). This is due to the meso-modelling technique, where crack path is already defined in terms of interactions between the masonry units and geometric discontinuity is created and propagated in the masonry. To assess the location and propagation of cracks, mesh size of 12 mm would be a more viable option considering it has the least computational time of all mesh sizes tried. Whereas, for studying the stresses and point displacements in the wall, mesh size of 3 mm should be most efficient in terms of computational times and accuracy of results.

8.2 Validation

8.2.1 Macroscopic Model

Macroscopic models were prepared to understand the evolution of crack in shear wall based on stress concentrations. These models can help predict the weak zones based on stress concentrations. Figure 8.16 shows the Von Misses stress contours plotted by Abaqus, where the stress concentrations are found towards the location of boundary conditions. The stress contours follow the path of load transfer from the application of push at bottom-left corner to the counter acting restraints at top-right corner.

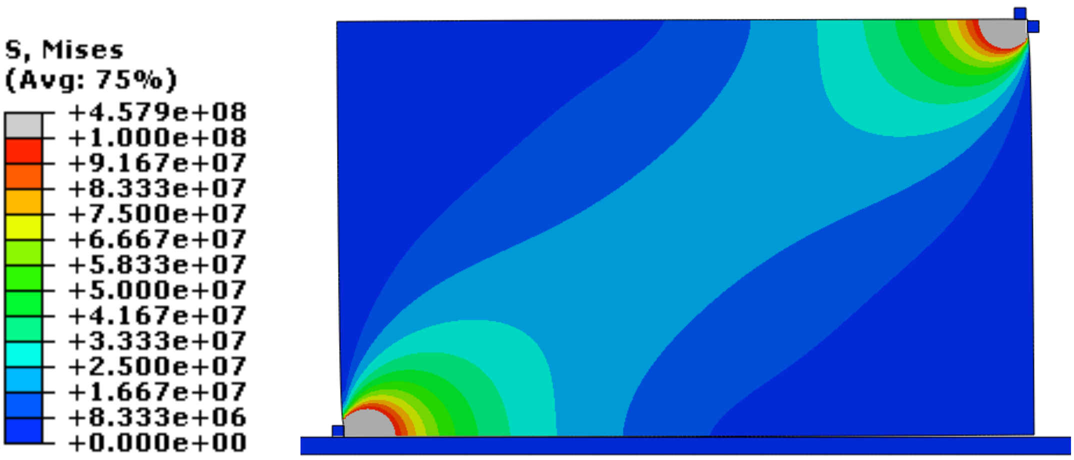


Figure 8.16: Von Misses stress contours for macro-model at 8 mm base push (*stress in N/m²*)

This path of load transfer determines the location for cracks. Thus for solid wall cracks prediction suggests a diagonal cracks from top-right to bottom-left corner of the wall which is the most idealistic crack orientation. Lab experiments discussed in Chapter-3 showed diagonal cracks as suggested by the macro model stress concentration with little variation as shown in Figure A.3.

Figure 8.17 shows the Von Misses stress contours plotted by Abaqus for wall with opening. Stress contours suggest crack orientation in diagonal direction from top-right corner of wall to the bottom-left corner of wall and progressing through the window opening. This cracks orientation was shown by specimen 5 and 6 of lab experiments in Figure A.9. The rest of the specimens showed sporadic and varied crack pattern as that suggested by the macro-model.

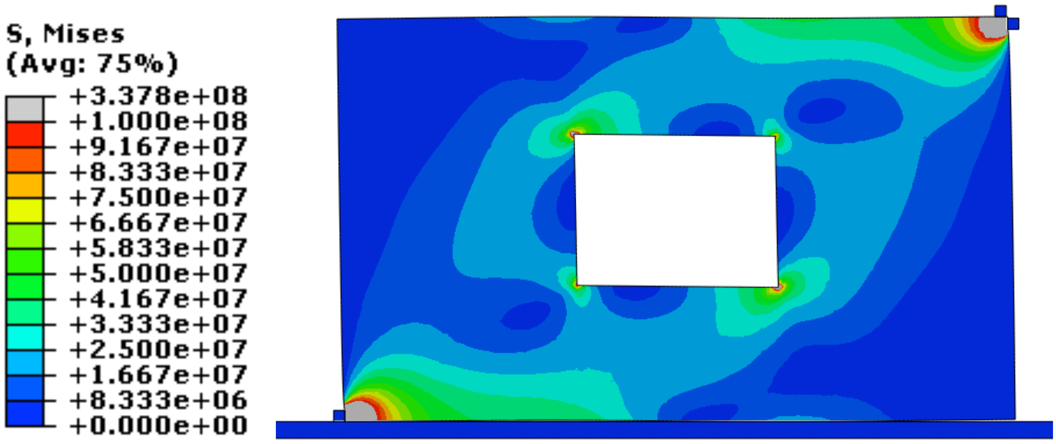


Figure 8.17: Von Misses stress contours for macro-model of wall with opening at 8 mm base push (*stress in N/m*^{2})

8.2.2 Mesoscopic Model

Numerical modelling of the small-scale experimental wall specimens were prepared to compliment the behaviour of glued solid wall under lateral loads observed during experimental study. These models would determine the validity of modelling technique and could be used for various masonry types.

For Glued solid wall total 8 models with unique random distribution of weaker cohesive interactions were tried and the resulting deformed geometries at the end of 8 mm base displacement are shown in Figure 8.19. Figure A.3 show similar crack patterns and deformed geometries observed in lab for solid glued wall specimens with the exception of Specimen-1. For lab specimen-1 the deformed shape shown in Figure A.3 is at the initiation of crack opening and not at the end of the experiment when the wall is no longer able to resist lateral loads. Looking at Figure 8.18, it can be seen that towards the end of loading the top portion of the wall snaps back under the excessive pressure build up and hence, resembles the behaviour shown by numerical Model-3 in Figure 8.19 with the exception of extra cracks developing in the test specimen.

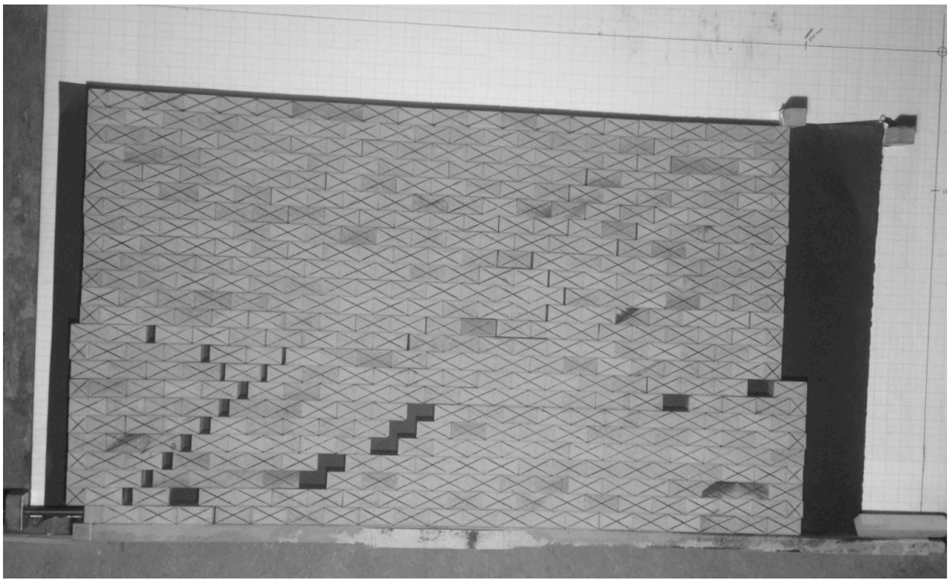


Figure 8.18: Specimen-1 of lab experiment at the end of test

For specimen 2 to 4 of lab experiments (Figure A.3), numerical model-6 offers the best match (Figure 8.19). To achieve the exact cracking pattern as the lab experiments we would have to assign weaker interactions in the numerical models at the exact locations. But, this would render the entire modelling approach of predicting the crack patterns for masonry shear walls as difficult. The results of numerical models are all possible crack patterns that might occur due to lateral loading on the wall under consideration. With the limited wall specimens tried in the lab, the crack patterns of Model-3 and Model-6 from Figure 8.19 gives the best match.

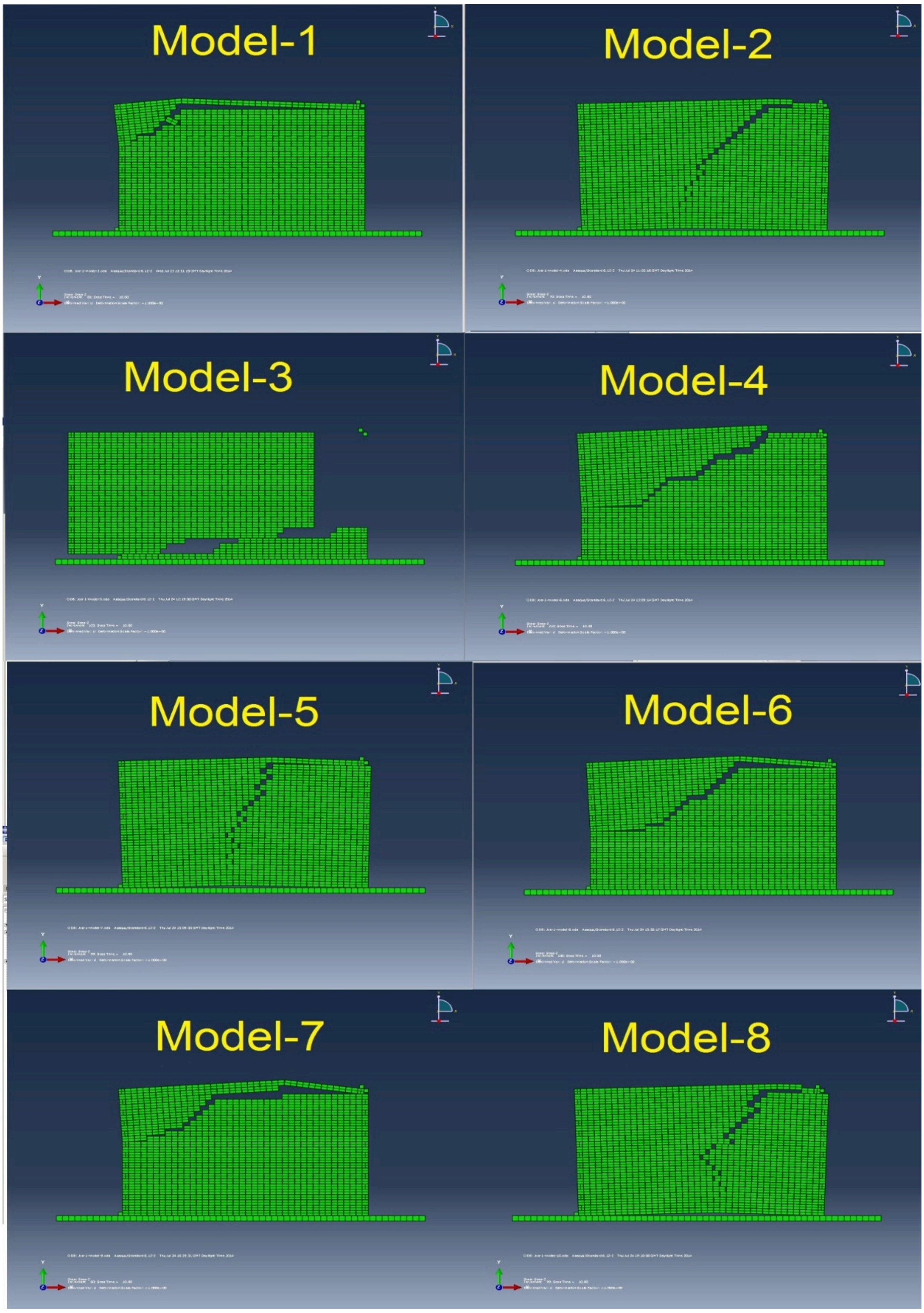


Figure 8.19: Numerical models for glued Solid wall (deformed geometry)

The cracks in numerical modelling occur at approximately 3.72 mm, which is fairly close to the average base displacement values of lab experiments for solid glued wall specimens i.e. 4.17 mm. This proves that the cohesive and damage values selected, after a series of hit and trial, for numerical model matches to that of the lab experiments. Load-displacement graph for the numerical simulation of wall model are shown in Figure 8.20, where the reaction values suddenly drops to zero or in some cases to 10% of the peak value after the initiation of cracks. This marks the total failure of wall to resist any further laterally applied loads and the average value from numerical simulations is around 4.5 mm, while lab tests gave an average value of approximately 5.3 mm. The difference in the values of base displacement at crack initiation and collapse are fairly close to the findings of the lab test i.e. approximately 15%.

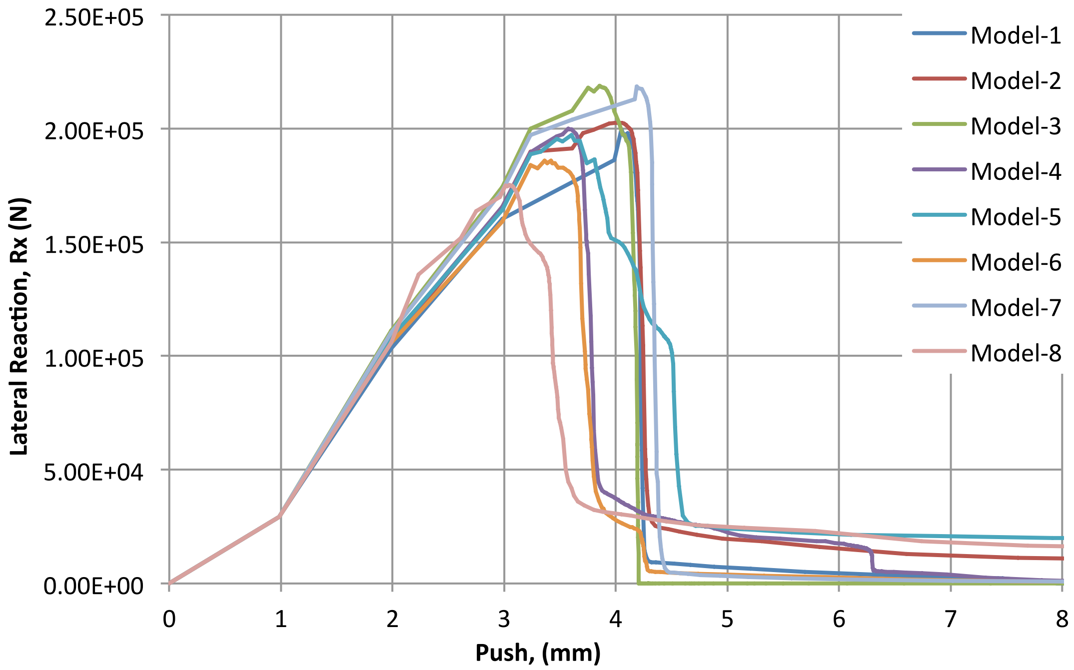


Figure 8.20: Load-displacement graph for numerical model of glued solid wall

To further test the validity of numerical simulation, wall movement and rotation after cracking is compared with lab results using displacement vectors. Figure 8.21 shows the same orientation for displacement vectors in numerical model and the experimental works mentioned in Chapter-3 (**Error! Reference source not found.** to **Error! Reference source not found.**). The top left portion shows counter-clockwise rotation where as the bottom portion continues to move in the direction of push.

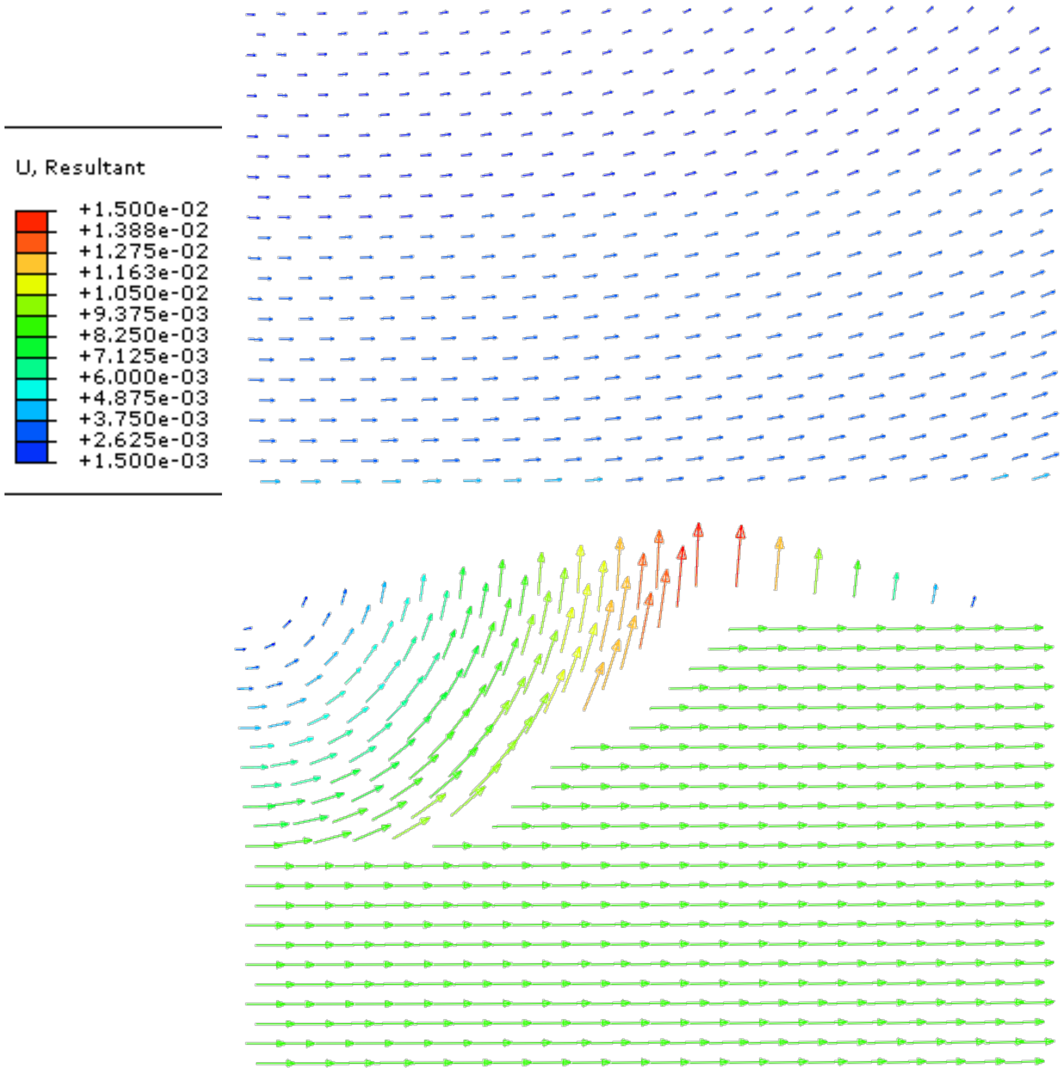


Figure 8.21: Displacement vector for glued solid wall from numerical analysis; pre cracking (*top*), post cracking (*bottom*) (*Dimensions in mm.*)

Similar microscopic models were analysed for wall with opening using random interaction distribution. Crack patterns observed in these models are given in Figure 8.22. These crack patterns are the same as discussed in the previous section of mesoscopic models, that is, diagonal cracks from the top-right corner of wall to the bottom-left corner of wall and through the window opening. This orientation follows the load transfer path in case of shear wall and the lab experiments showed similar crack orientations (Figure A.9). Cracks in the wall occur at approximately 2.8 mm and the total loss of load carrying capacity of wall is around 4 mm in comparison to the experimental values of 3 and 5 mm, respectively. The percentage difference of numerical to experimental values is within acceptable range of 20%.

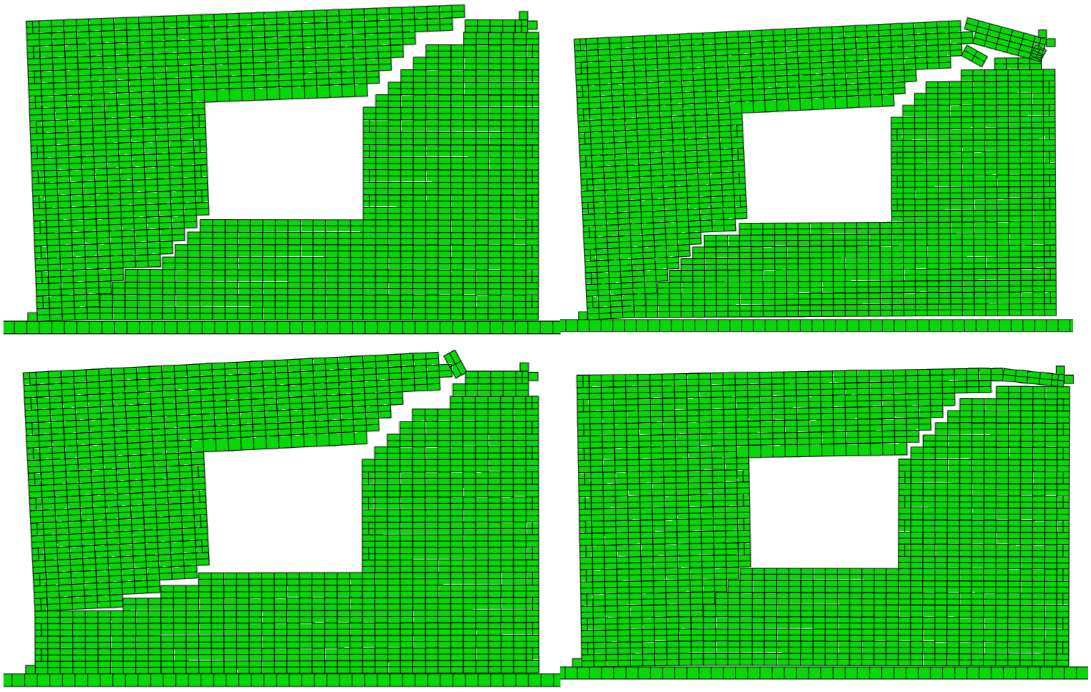


Figure 8.22: Numerical models for glued wall with opening (deformed geometry)

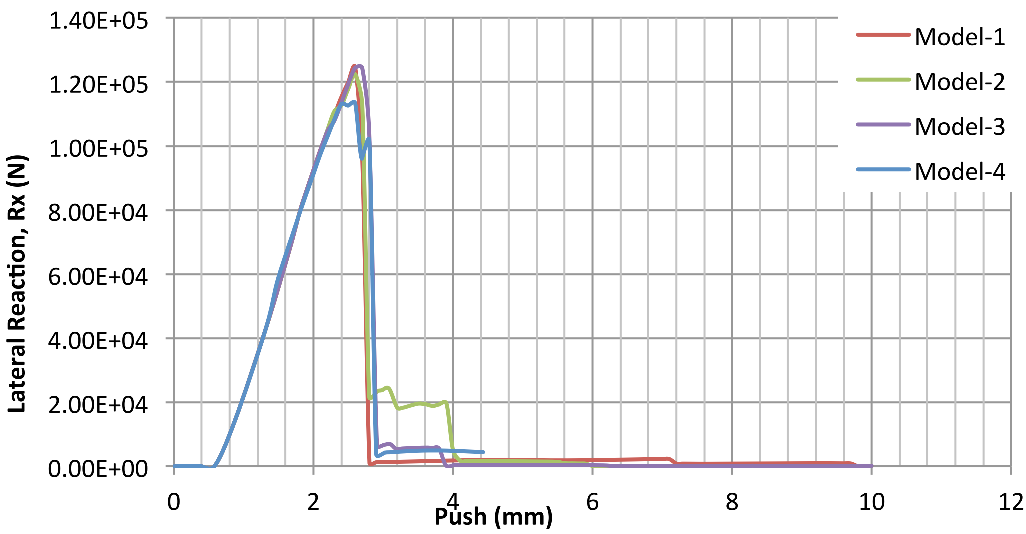


Figure 8.23: Load-Displacement graph for numerical model of glued wall with opening

8.3 Alternate Numerical Model based on ASTM E519/519M-15

The initial shear strength of masonry as prescribed by BS-EN [18] gives the shear strength of mortar joints. On the other hand the ASTM standards prescribe a different test setup in ASTM E519/519M-15 [21] called “Diagonal Compression Test” where the shear strength is calculated by applying a diagonal compressive load on the two opposing corners of a 1200 mm x 1200 mm wallette. This test gives the shear and tensile strength of masonry and can be used to determine the shear modulus of masonry by measuring the vertical and horizontal strain.

The test setup of ASTM E519 is performed numerically through Finite Element software using the mechanical properties of masonry units and mortar joints obtained previously through experiments. The model consists of masonry units modelled as shell elements with Modulus of Elasticity determined through Compression strength tests of masonry units. The size of masonry units for numerical modelling is altered to include the thickness of mortar joints. Joints between masonry units are modelled as cohesive contacts with normal and shear stiffness values derived from the load-displacement curves of Initial Shear Strength Tests. The contact also includes the damage parameters to allow the failure of joints when the maximum stress limit is reached. The maximum values for normal and shear strength are obtained from the results of Bond Wrench and Initial Shear Strength Tests, respectively. Basic Coulomb friction model is used to assign the friction coefficient between contacting surfaces of masonry units, the values for friction coefficient are obtained from Initial Shear Strength Test results. This friction model incorporates the effect of contact pressure from the overlying masonry and generates frictional resistance between the contacting surfaces of masonry units after the initiation of damage in cohesive model.

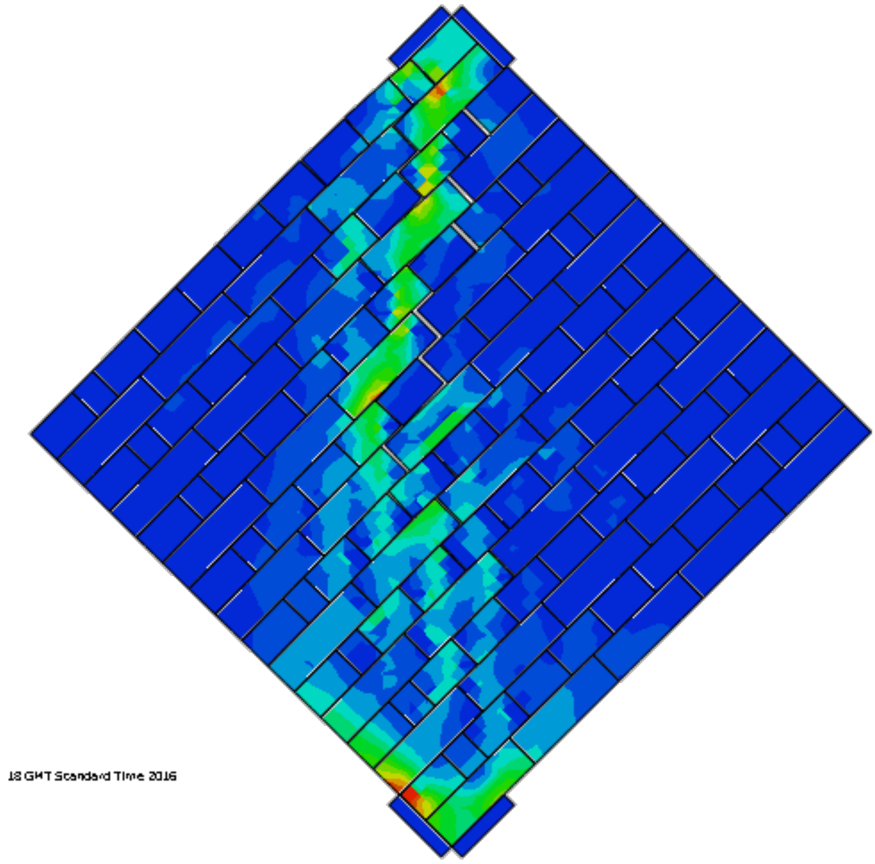


Figure 8.24: Screenshot of diagonal compression test non-retrofit R1M2 masonry in Abaqus

The results of diagonal compression test simulations are used to assess the shear strength of masonry (τ_0) in comparison to the Initial Shear Strength (f_{vo}) performed in accordance of BS EN 1052-3:2002. Apart from shear strength of masonry, young's modulus (E) and shear modulus (G) were also calculated using the equations given below.

$$\tau_0 = \frac{0.707 \times P_{max}}{A_n} \qquad \textit{Eq. 8.8}$$

$$A_n = \left(\frac{w+h}{2}\right) \times t \times n \qquad \textit{Eq. 8.9}$$

$$G = \frac{\tau}{\gamma} \qquad \textit{Eq. 8.10}$$

$$\gamma = \frac{\Delta v}{g_v} + \frac{\Delta h}{g_h} \qquad \textit{Eq. 8.11}$$

Where, P_{max} is maximum applied load to the specimen, A_n is net area of panel, w is width of panel, h is height of panel, t is the total thickness of the panel and n is percent of the unit's gross area that is solid, expressed as a decimal. In the present work n = 1 was adopted. γ is shear strain, Δv is the vertical shortening of the panel, Δh is the horizontal elongation of the panel, and g_v and g_h are the original vertical and horizontal lengths, respectively [135]. For calculating modulus of elasticity Eq. 8.5 is used with a simple design assumption of Poisson's ratio 0.25 [136].

$$G = \frac{E}{2(1+v)} \qquad \textit{Eq. 8.12}$$

Table 8.1: Diagonal compression results from Numerical simulations

Masonry type	Numerical model τ_0 (MPa)	Percentage difference from f_{v0} (%)	Modulus of Elasticity, E (MPa)	Shear Modulus, G (MPa)
R1M1	0.25	-30	131	52
R1M2	0.12	-6	108	43
R2M1	0.13	6	83	33
R2M2	0.10	35	14	5

Table 8.1 lists the values of shear strength for redbrick masonry specimens obtained through numerical modelling of masonry wallettes in Abaqus to replicate diagonal compression test of ASTM and compared with the values of lab tests carried in accordance with BS-EN. Also the calculated values of modulus of elasticity and shear modulus on the basis of Eq.8.1-8.5 are given. The values of shear strength (τ_0) digress from initial shear strength (f_{v0}) by only 6% for R1M2 and R2M1 specimens but the difference is more than 30% for R1M1 and R2M2 masonry. The results show that the numerical model has more accuracy in predicting shear values around 0.125 MPa and as the strength values move away from that mark the difference increases.

Table 8.2: Values of non-retro compared to retrofit wallette

Masonry type	**Numerical model τ_0 (MPa)**	**Modulus of Elasticity, E (MPa)**	**Modulus of Rigidity, G (MPa)**
R1M2	0.12	108	43
R1M2-retro	0.13	108	43

Table 8.2 lists the values of shear strength, modulus of elasticity and shear modulus for R1M2 non-retrofit masonry and R1M2 retrofit masonry. Six PP-bands were provided in each direction and distributed appropriately along the width of wallette to have fairly equal spacing of 230 mm. It is evident that providing PP-band retrofit had no effect on the modulus of elasticity and shear modulus of the specimen as these values were calculated from the pre-cracking load-displacement graph of the specimen when the effect of retrofit is non-existent. Peak strength of the specimen shows a slight increase in value by approximately 8%. But do confirm this effect of PP-band retrofit on the peak strength of masonry needs studying the load-displacement curves of individual models given subsequently in Figure 8.26 to Figure 8.29.

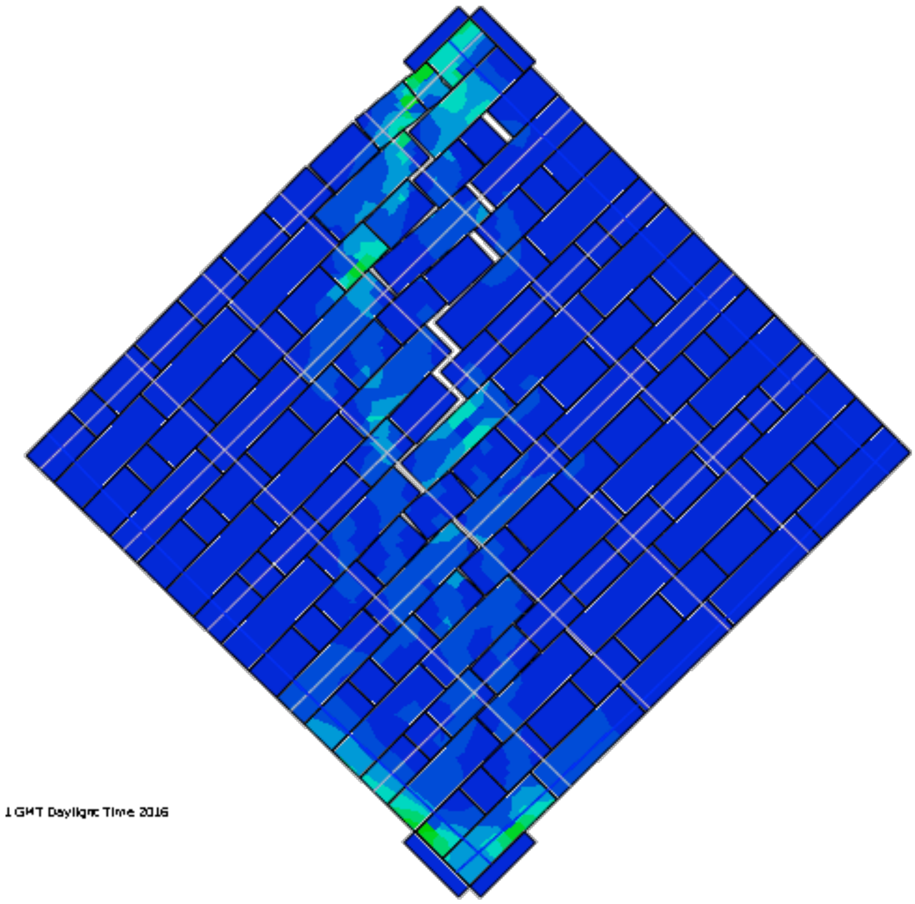


Figure 8.25: Screenshot of diagonal compression test retrofit R1M2 masonry in Abaqus

Figure 8.26 to Figure 8.29 gives the load-displacement graphs of the numerical simulations conducted on R1M2 masonry panels with and without retrofit. As the loading plate made contact with the specimen after 1.2 mm of downward displacement due to the initial clearance provided in model, therefore, the load-displacement graphs show no increase in the load initially up to 1.2 mm of Push. The graphs show similar peak strengths for non-retrofit and retrofit specimens in Figure 8.26 and Figure 8.27. In case of model-3 (Figure 8.28) the retrofit specimen has 20% lower peak strength as compared to non-retrofit specimen. While in Figure 8.29 the non-retrofit specimen has 20% lower peak strength as compared to retrofit specimen. Thus, it can be concluded that PP-band retrofit has no decisive effects on the peak strength of masonry.

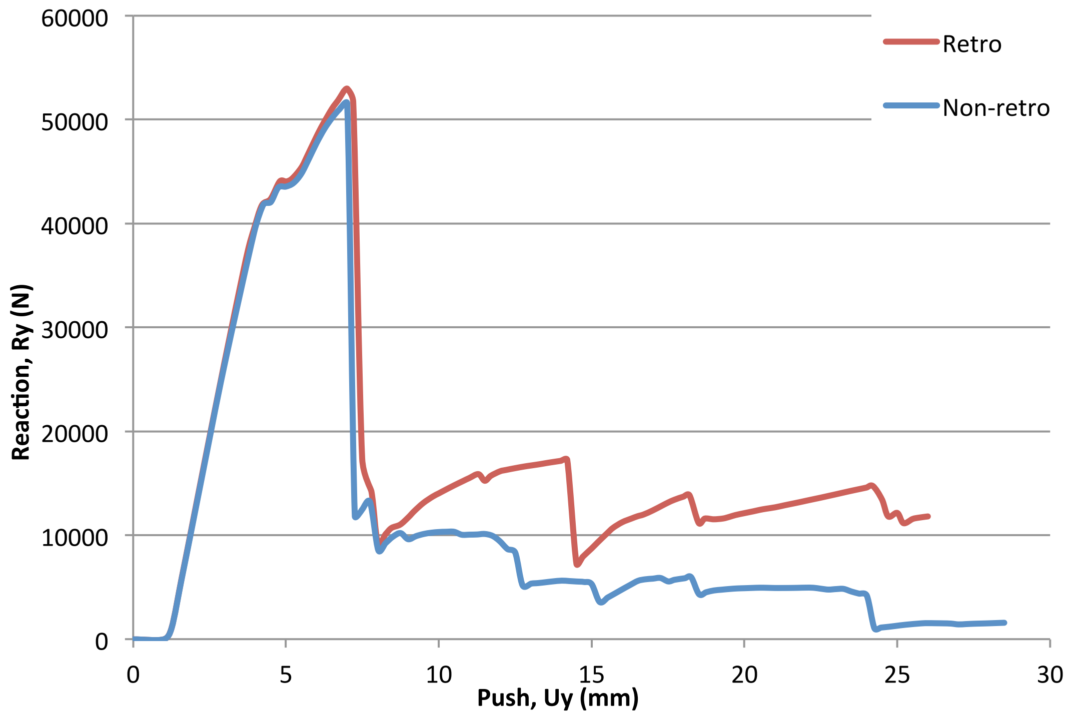


Figure 8.26: Load-Displacement for retrofit and non-retrofit simulation of model-1

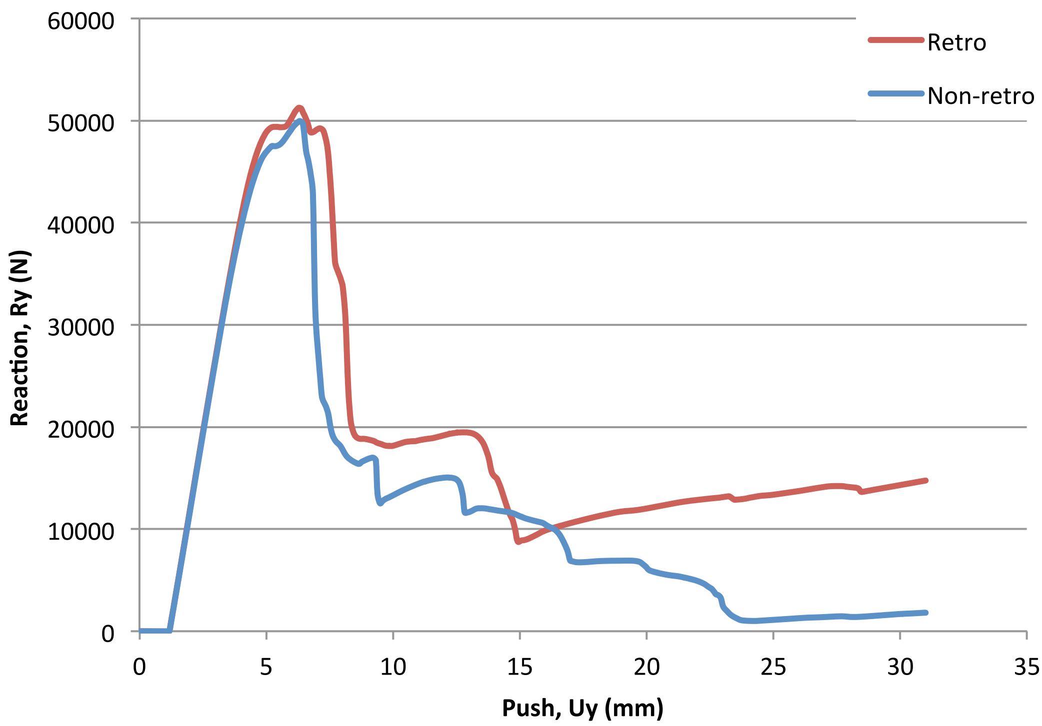


Figure 8.27: Load-Displacement for retrofit and non-retrofit simulation of model-2

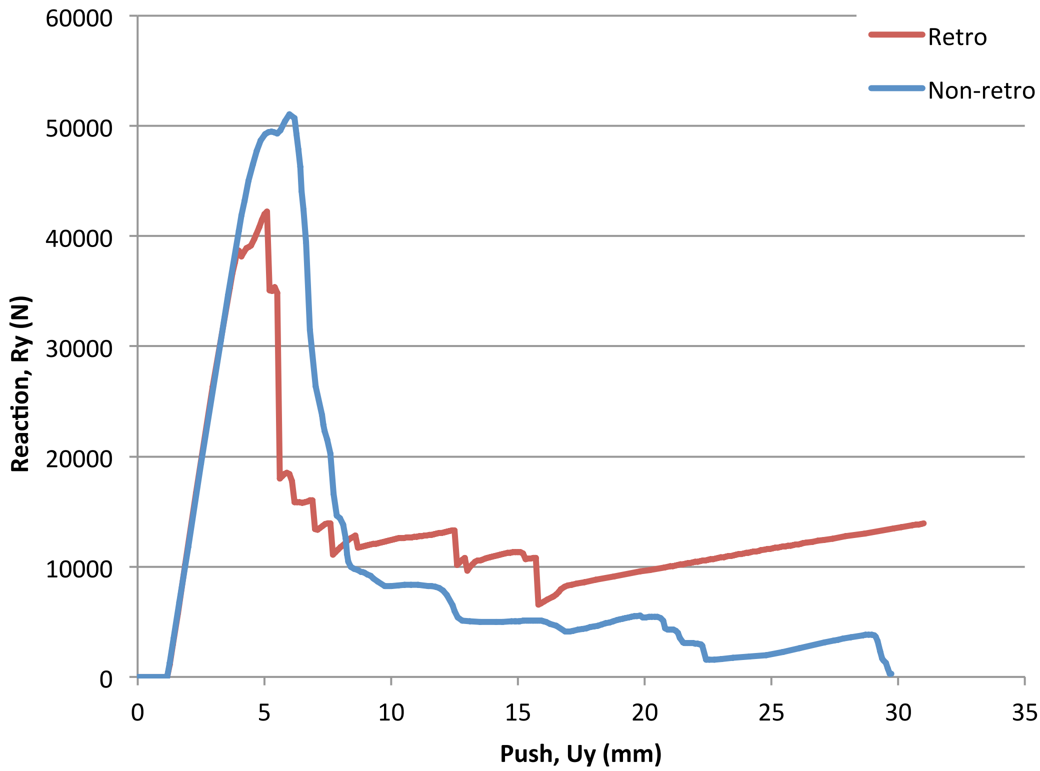


Figure 8.28: Load-Displacement for retrofit and non-retrofit simulation of model-3

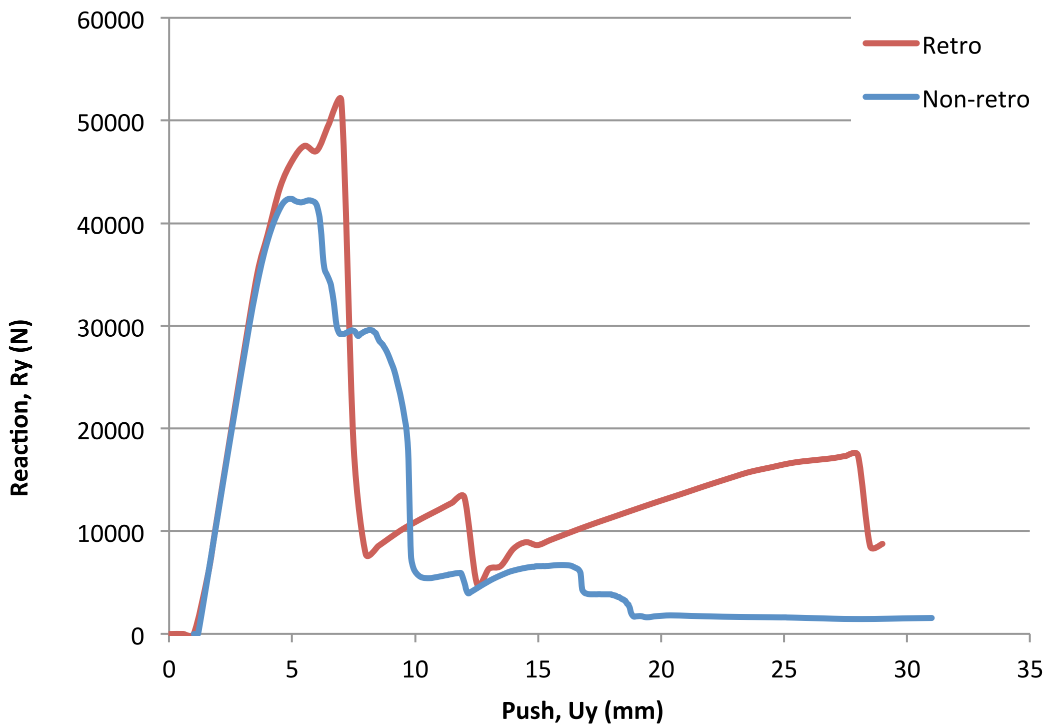


Figure 8.29: Load-Displacement for retrofit and non-retrofit simulation of model-4

The load-displacement curves for retrofit specimens show higher post peak strength as compared to non-retrofit specimens. For retrofit specimens the post peak strength is around 30% of the peak strength and for non-retrofit specimen it drops below 20% of peak strength and then continues towards zero. Whereas, for retrofit specimens the post peak strength gradually moves upwards due to the stiffening of PP-bands. Therefore, the post peak behaviour of the masonry panels retrofitted with PP-bands show better integrity and load carrying capacity for longer durations of loading.

8.4 Concluding Remarks

Numerical models developed as a part of this research have shown good agreement with the experimental results. The formation of crack and its progression can be predicted with micro-modelling approach of current research. Macro-models help predict the idealised crack pattern in masonry through stress concentrations where as micro-models allow for strength discrepancies in the model and generate unique crack pattern each time.

The modelling technique is further extended to model diagonal compression test setup for masonry as prescribed by ASTM E519/519M-15 [21]. The values for material and joint properties are taken from the findings of Chapter-5. The shear strength (τ_0) obtained from numerical modelling of wallette according to ASTM is validated against the initial shear strength ($f_{v0)}$ values obtained through lab tests according to BS-EN. The differences in shear strengths obtained from BS-EN and ASTM range from 6% to 35%.

Furthermore, wallettes were retrofitted with PP-bands to study the affects of retrofitting. The initial load displacement curve for retrofit and non-retrofit wallettes follow one another up to the point of crack initiation after which the retrofit models show residual strength of 30% of peak strength while for non-retrofit specimen it goes down to zero.

Chapter 9 : Summary, Conclusions and Future Recommendations

9.1 Summary

Masonry has especially been problematic in seismic conditions with its highest application in poorer communities. Constructions in these areas are majorly non-engineered and use non-standard local material, which costs cheaper. Most retrofitting/strengthening methods suggested for masonry construction are unfeasible for the rural communities in terms of economy and material availability. Literature research has been carried out in Chapter-2 to familiarize with masonry performance and failure patterns under seismic loads. All available retrofits for masonry were studied to outline their advantages and disadvantages with respect to economy, material availability, ease of application, aesthetics and sustainability. To better estimate the applicability of retrofit, Kashmir region was selected as a case study and the past earthquakes in the region were researched and documented. Based on the selected region involving its topography and economy, PP-band retrofit was selected as the most viable solution.

Further the research involves experiments on small-scale freestanding shear walls to understand masonry behaviour and the subsequent crack propagation. The test also revealed the effects of retrofit on wall behaviour and crack propagation. Finite element software package 'Abaqus' was used to develop numerical models for the test setup.

Field survey of the selected region was carried out to understand the common practices and the problem faced by the local construction industry. Trip to Muzaffarabad city in Kashmir region revealed information about the area, which is usually not published due to lack of awareness and miscommunication. The city suffered heavy damage during 2005 earthquake and since then the construction practices have changed. The report details the construction practices of before and after 2005 earthquake and also mentions the currents flaws in the construction industry including the negligence of authorities. Following the visit to Muzaffarabad another close by city of Mirpur was surveyed. The city survived the 2005 earthquake with minimal damage but with the revised seismic zoning of the country it now faces a high risk for future earthquakes and the constructions in the area are mostly unreinforced brick masonry.

Materials used for masonry construction in Pakistan were tested for their compressive strength, modulus of elasticity, tensile strength, bond's shear and tensile strengths. The strength of materials and their modulus of elasticity showed lower values as compared to the western standards. This is due to the unregulated and non-standardized manufacturing of masonry units. Mortar, which normally contains unregulated and unspecified amount of water, has lower strength and modulus of elasticity. Mortar bond between masonry units however depends on the strength and modulus of elasticity of masonry units and mortar along with the effects of surface finish. The details of these phenomena are mentioned in Chapter-5. PP-bands used for the subsequent shake table tests were also tested for their tensile strength and elasticity.

To demonstrate the effectiveness of PP-band retrofit, two wallettes were tested concurrently on shake table. One of the wallette was retrofit with PP-bands and showed better integrity and higher shear strength. The post peak strength of the retrofit specimen was 7 times higher than non-retrofit specimen in terms of arias intensity as explained in Chapter-6. Shake table test was further extended to a real-scale masonry structure retrofit with PP-bands to demonstrate its effectiveness. The first test showed weakness in bands due to the selected fixing mechanism. However, the retrofit structure showed greater post peak integrity for masonry. The connection details were revised for the second test on similar structure. The subsequent test showed no weakness in the connection details and the structure sustained PGAs of upto 3g. The modified retrofit technique proved strong to the extent that the bands had to be cut to facilitate the process of collapse and avoid damage to equipment.

Chapter-8 discusses the numerical modelling approach for masonry with regards to macro- and meso-modelling. Numerical models for small-scale lab experiments were analysed to validate the wall behaviour with the modelling approach. Further diagonal shear wall test setup of real masonry was modelled in accordance with ASTM standards using meso-modelling approach to compare with the shear strength obtained through lab tests carried out in accordance with BS-EN standards. PP-band retrofit was tried on diagonal shear walls to assess the difference in peak and post peak strength of retrofit wallette in comparison to non-retrofit wallettes.

9.2 Conclusions

Objective one of this research involved a careful selection of a retrofit method viable for rural communities. With a thorough research on available retrofit methods suggested in the past PP-band retrofit was selected as the most economical solution for strengthening existing and new masonry constructions. PP-band material is cheap, readily available and can be done by unskilled workers; waste PP-bands from packaging can also be reused to reduce carbon footprints. Applicability of PP-bands has been addressed in this thesis to provide guidelines backed by practical demonstration on shake table of a real scale masonry structure retrofitted with PP-bands.

Objective two of this research has been addressed in Chapter-3, where small-scale experiments were conducted to understand the failure patterns of masonry under shear loads. These tests also help to determine the effects of openings and retrofit. Experiments revealed that an opening size of 11% of the wall area in the centre of the wall reduces its strength by 25%. Retrofit increase the base displacement values at crack initiation (U^o) for non-glued wall by 100% and for glued wall by 25%. Higher effects on non-glued samples are due to the absence of cohesion forces between bricks, which are a significant force that determines the initiation of cracks. The base-displacement values at collapse (U^f) for retrofitted wall increased 62% in compariosn to its non-retrofit counterpart, thus suggesting a higher influence of PP-bands on the post peak behaviour of masonry. Difference in the base displacement at the time of crack initiation (U^0) and failure (U^f) for retrofit specimen increased by 120% as compared to the non-retrofit specimen, suggesting a better post peak behaviour and wall integrity.

Objective three and four of this research has been covered in Chapter-4 of this research where construction practices and problems faced in Kashmir are detailed. Site survey of Muzaffarabad revealed the problems of the local construction industry. Interviews with locals revealed the sub-standard practices of pre 2005 earthquake and to what extent these practices have changed since then. Major problem in the region is economic condition of the population that hinders adherence to standards of construction, material and site selection. Interviews with personnel from the local authority complain about the lack of available resources and manpower to regulate the construction in the area to the specifications of the codes. However, authorities claim to provide financial grants to those who provide RCC confinements or some other suitable seismic strengthening measures to their structures.

Due to the poor performance of masonry during 2005 earthquake 80% of new house constructions are RCC frame structures. Because people are now moving towards RCC frame construction they believe future damage under similar earthquake of 2005 would be 70% less. Masonry, which used to dominate the city construction, is now being taken over by RCC frame structures. However, due to non-regulated and non-engineered constructions, RCC structures still lack the proper seismic detailing and planning. People are unaware of the activities or existence of disaster management authority in the region.

Field study of Mirpur revealed similar construction mistakes as those present in Muzaffarabad, which resulted in high death toll during 2005 earthquake. The region should be given immediate attention and existing unreinforced masonry structures should be retrofitted to avoid casualties in future earthquakes. Disaster management cell should be created for the region to devise plans for future earthquakes to provide emergency relief.

Objective five of this research has been covered in Chapter-5 of this research. Material tests for masonry revealed increasing cement content in mortar by 100% increases its compressive strength by 250% and flexural strength by 80%. Initial shear strength of the joints increased by approximately 200%. While a 100% increase in the compressive strength of masonry units increases the tensile strength of the units by 50%. No significant effect was shown on the initial shear strength or bond strength of the masonry joints due to the increased compressive strength of the masonry units. Results further revealed that joint strength of masonry is a combined action of mortar and masonry unit's strength and elastic modulus. Using a very high strength masonry unit with weak mortar or vice versa would not result in high bond strength. Surface finish of the bricks, is the most significant factor affecting masonry shear strength and friction coefficient.

Objective six of the research has been achieved in Chapter-6 of shake table test for assessing PP-band effectiveness. Shake table tests of masonry wallettes revealed an increase in post cracking strength of retrofit specimen in comparison to a non-retrofit specimen by 7.4 times in terms of cumulative arias intensity. Shake table test on real scale masonry structures revealed the significance of PP-band fixing mechanism. For Room-1, holes were punctured in PP-bands for fixing screws while Room-2 adopted a different fixing detail that avoided puncturing holes in the bands. Performance for Room-2 was increased 100% compared to Room-1 in terms of cumulative arias intensity. Performance of the PP-bands had been proven to maintain structural integrity to ground vibrations of up to 2g with no life threatening damage. Retrofit structure survived accelerations of 3g before collapse with PP-band cut in places to facilitate demolition without causing damage to equipment.

Objective seven and eight of the research has been realised in Chapter-8 of this report. The shear strength (τ_0) obtained from numerical modelling of masonry according to ASTM is validated against the initial shear strength (f_{v0}) values obtained through lab tests specified in BS-EN. The differences in shear strengths obtained through numerical simulations and lab tests differ by 6% to 35%. Retrofit and non-retrofit numerical models for masonry reveal similar load-displacement curves up to the point of crack initiation. But, the post peak strength for retrofit models show minimum residual strength of 30% of peak strength and upon further application of load it increases gradually. Whereas, non-retrofit models give a residual strength of 10% after crack initiation and upon further application of load it gradually goes down to zero.

9.3 Future Recommendations

Future recommendations made on the basis of current research are as follows:

Investigation of masonry shear behaviour

Small-scale experiments on masonry shear walls as discussed in Chapter-3 helped provide useful information about masonry failure and the effectiveness of PP-band retrofit. This experimental study should be extended to include further wall configurations for studying the effects of wall aspect ratio and opening sizes. Test setup should be modified to allow for the measurement of peak loads which would help determine wall strength and stiffness.

Survey of high seismicity under developed region

Site study carried out in Chapter-4 should be extended to include communities located at higher altitudes on mountains with poor access and resources. Questionnaires should be prepared for house owners having their house built recently to ask them their choice for construction material and the problems they face during construction.

Material test for masonry

Material testing discussed in Chapter-5 should be extended to include more mortar ratios to help ascertain the findings of current study in regards to the affects of mortar strength on joints. Compressive test setup should be modified to measure lateral elongation to help calculate the tensile modulus of elasticity.

Dynamic tests for PP-band retrofit masonry

PP-band retrofit has shown to work well with redbrick masonry especially with the selected connection detail (Chapter-7). This retrofit should be tested with regards to efficiency of various connection types available in literature. Similar study for PP-band retrofit masonry should be conducting for various types of masonry such as adobe and stone. The scheme for PP-band retrofit needs to be extended to assess its effectiveness in terms of double storey structures. Parametric study should be carried out on to provide design guidelines for PP-band retrofit for incorporation into design codes of masonry for seismic actions.

Numerical modelling of masonry

Meso-modelling of masonry can be very computationally expensive and complicate especially if a dynamic analysis of complete 3D structures is desired in the future. The modelling approach devised in this research should be simplified for application in 3D continuum models by using single wythes with larger brick size to account for 230 mm thickness of the wall. Further research in the macro modelling of masonry is recommended because of its easy modelling and reduced computational times. To achieve the desired crack patterns in macro-model, material damage models need to be studied. But, obtaining an accurate crack prediction model that not only produces desired cracks in masonry but also accurately simulates the post cracking behaviour can be cumbersome in case of continuum models. For such a plastic damage model for masonry needs to be developed for measuring the global response in pre and post cracking stage.

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Appendix-A

Lab Pictures for Solid Wall Analysis

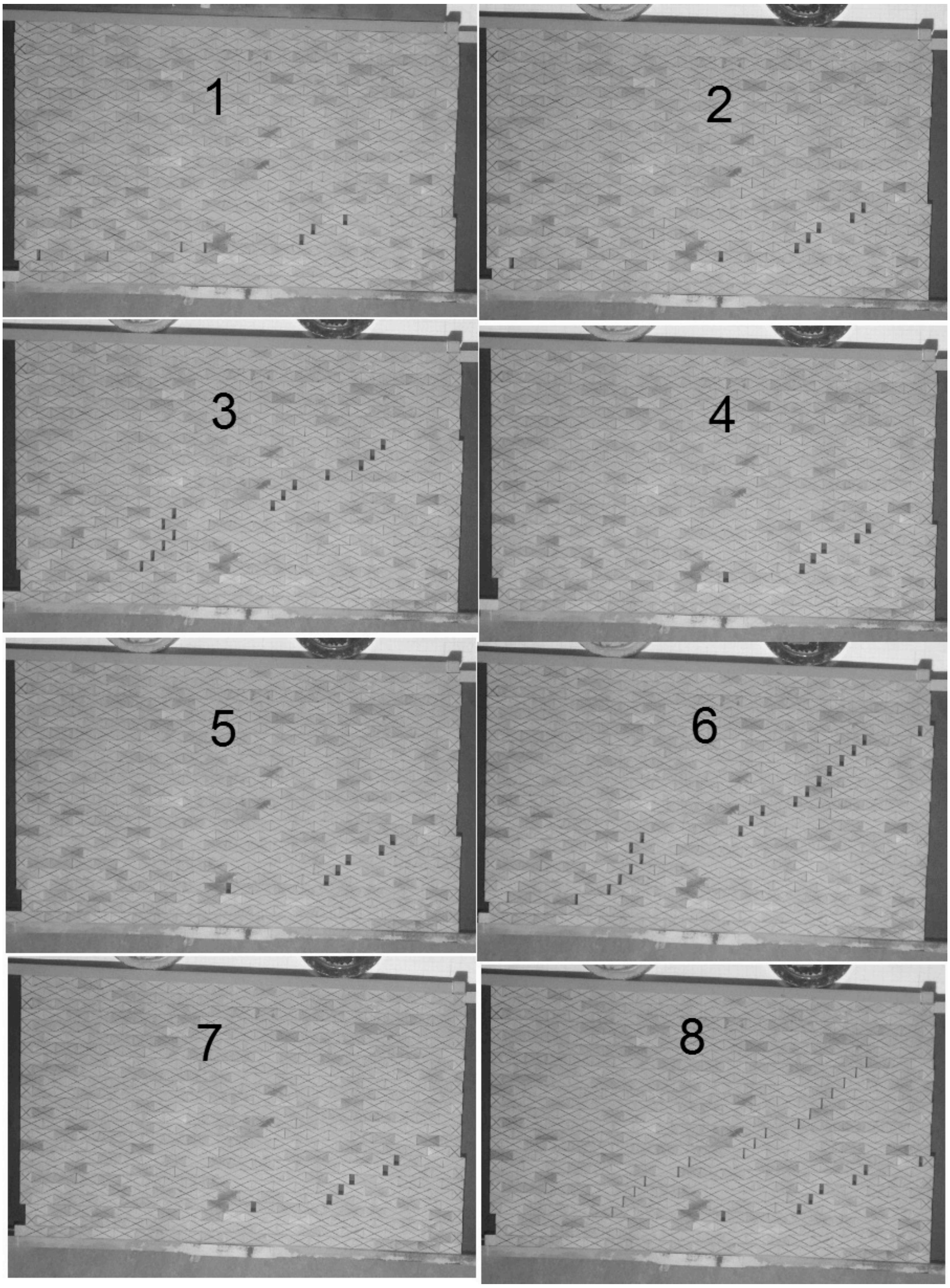

Figure A.1: Non-glued solid wall specimens at 7 mm base displacement

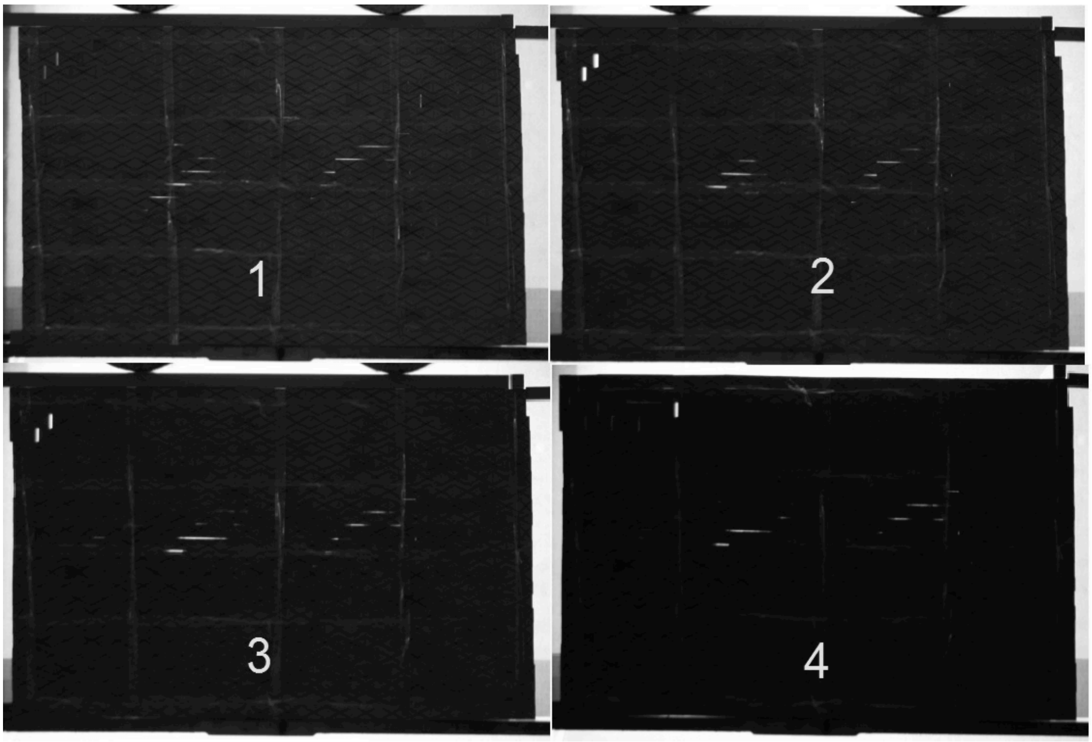

Figure A.2: Retrofitted Non-glued solid wall specimens at 7 mm base displacement

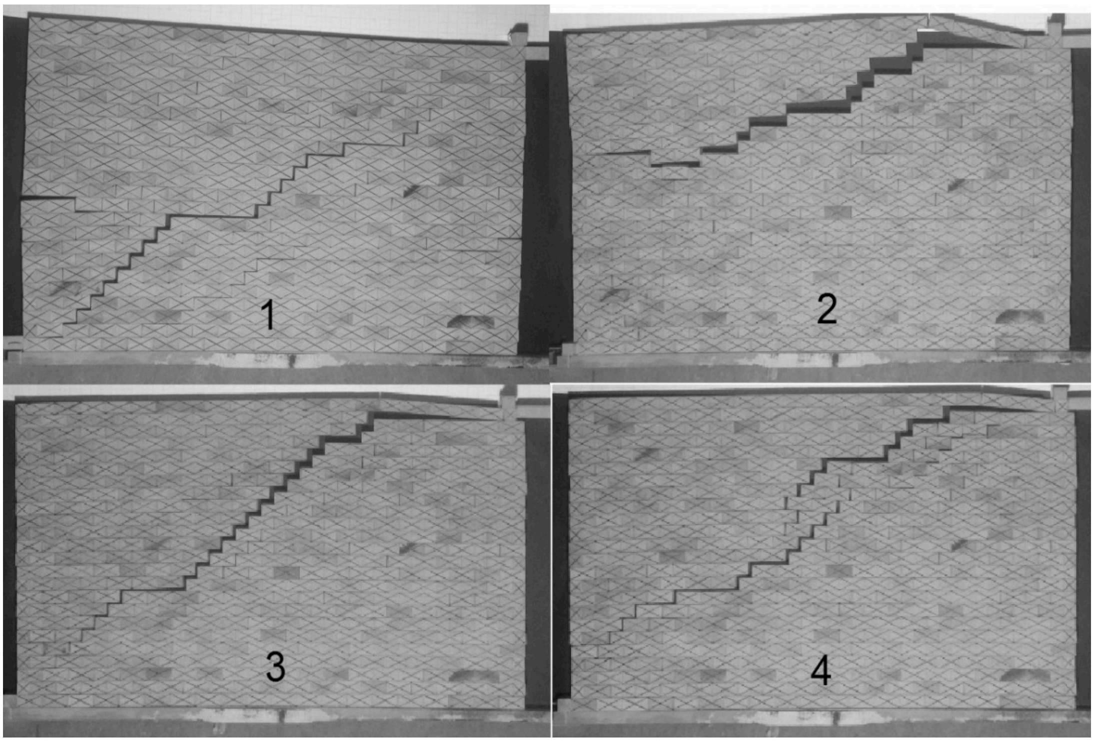

Figure A.3: Glued solid wall specimens at 7 mm base displacement

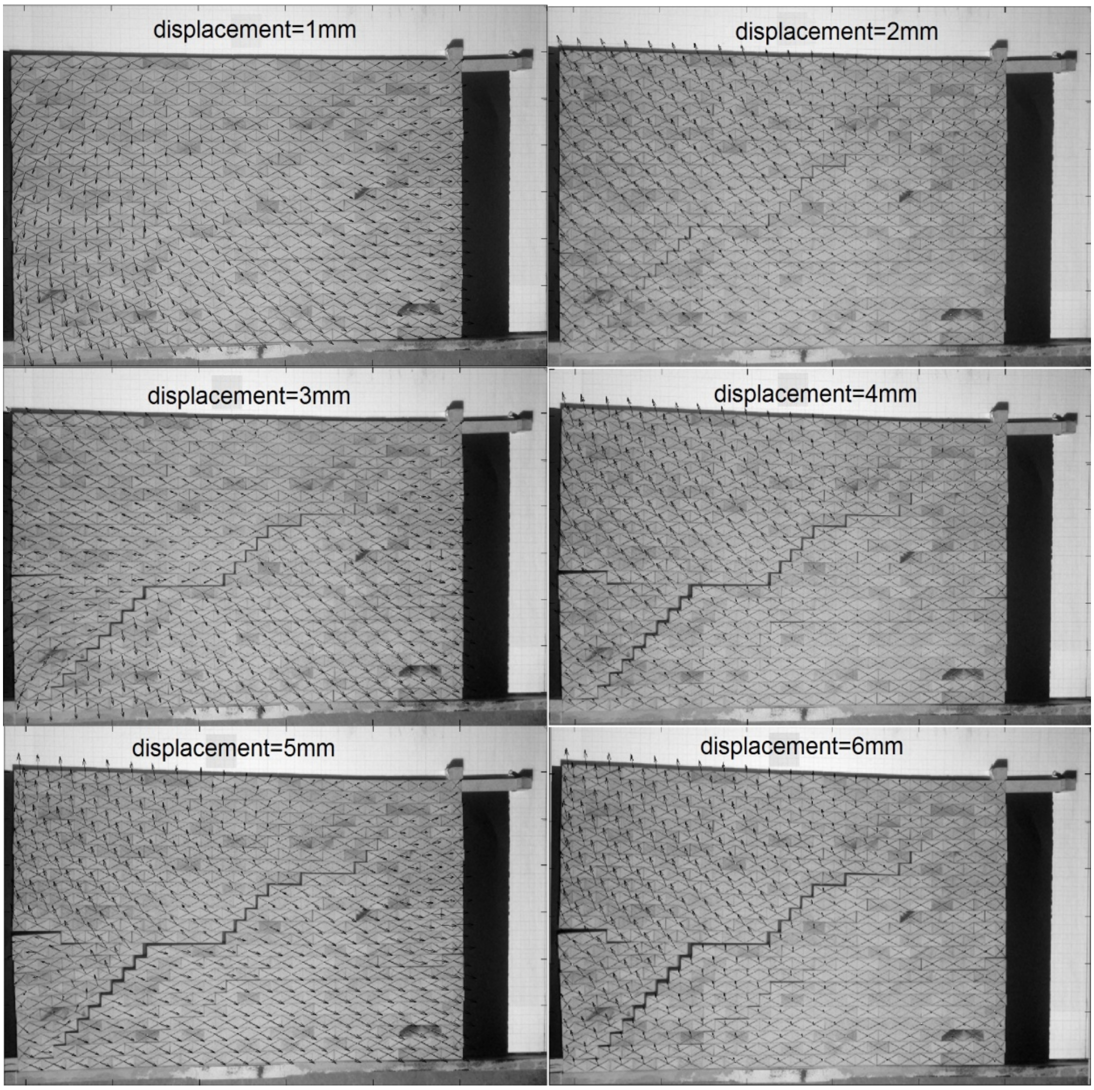

Figure A.4: Progressive displacement vector for Specimen-1 of glued solid wall

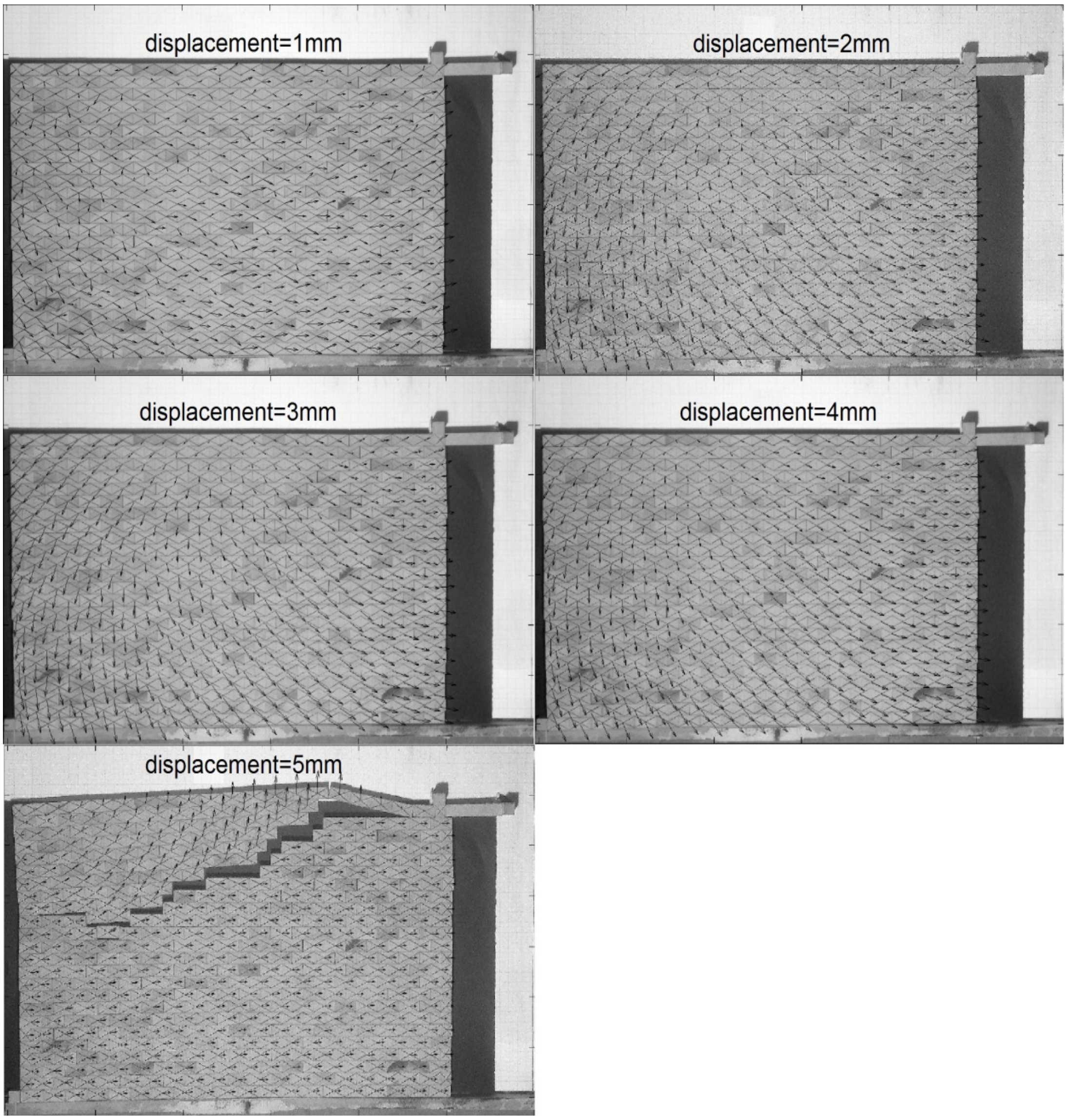

Figure A.5: Progressive displacement vector for Specimen-2 of glued solid wall

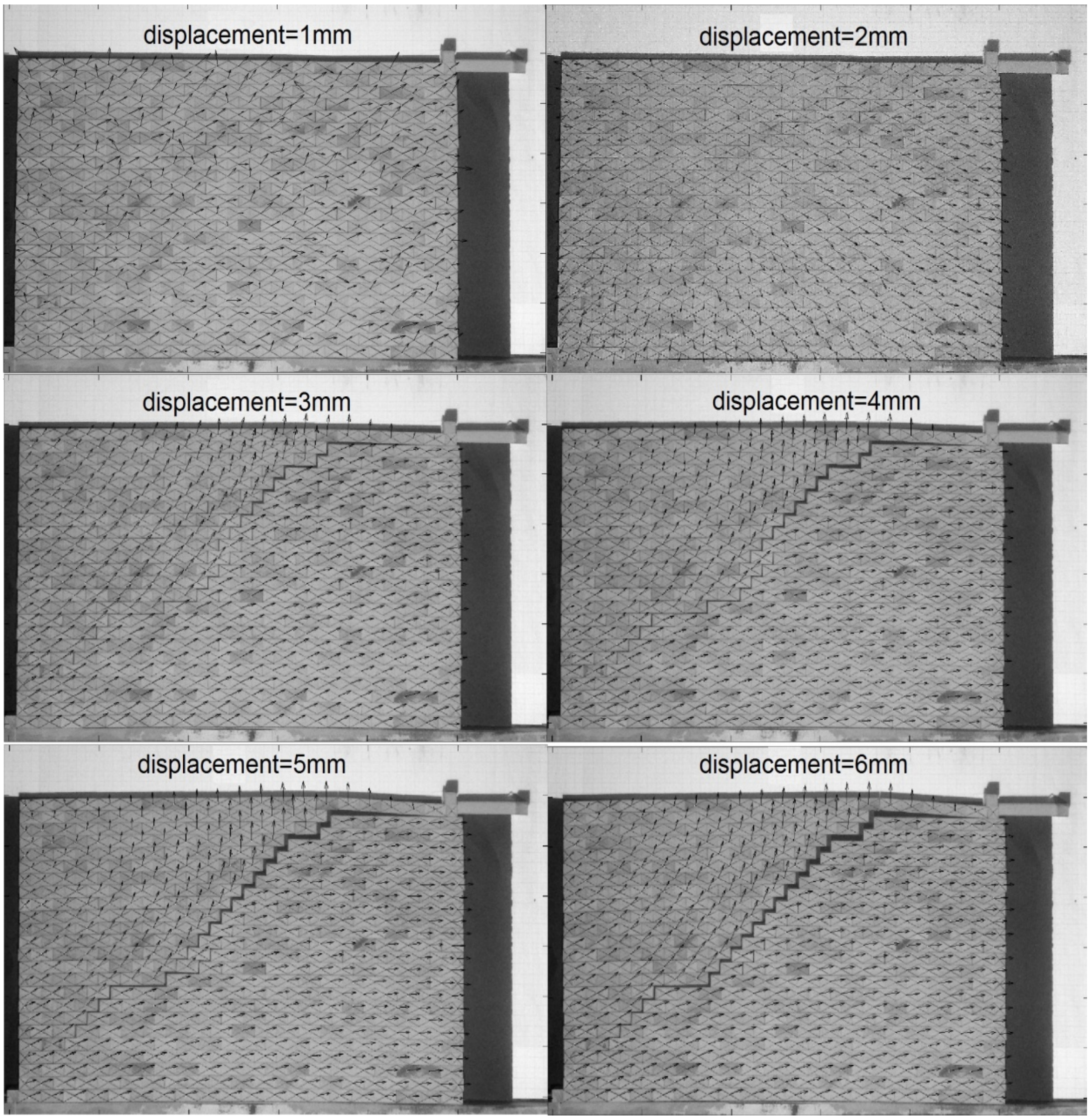

Figure A.6: Progressive displacement vector for Specimen-3 of glued solid wall

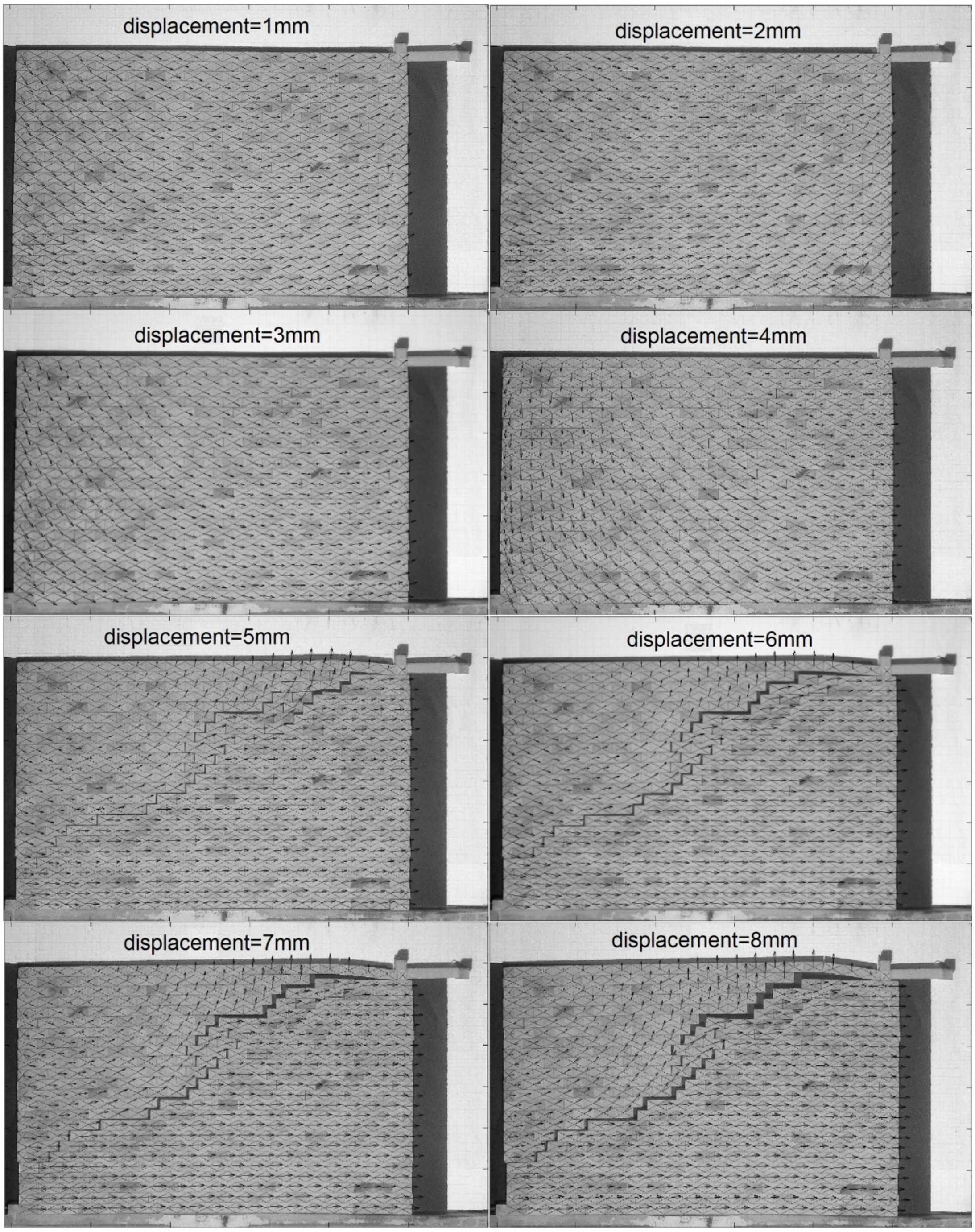

Figure A.7: Progressive displacement vector for Specimen-4 of glued solid wall

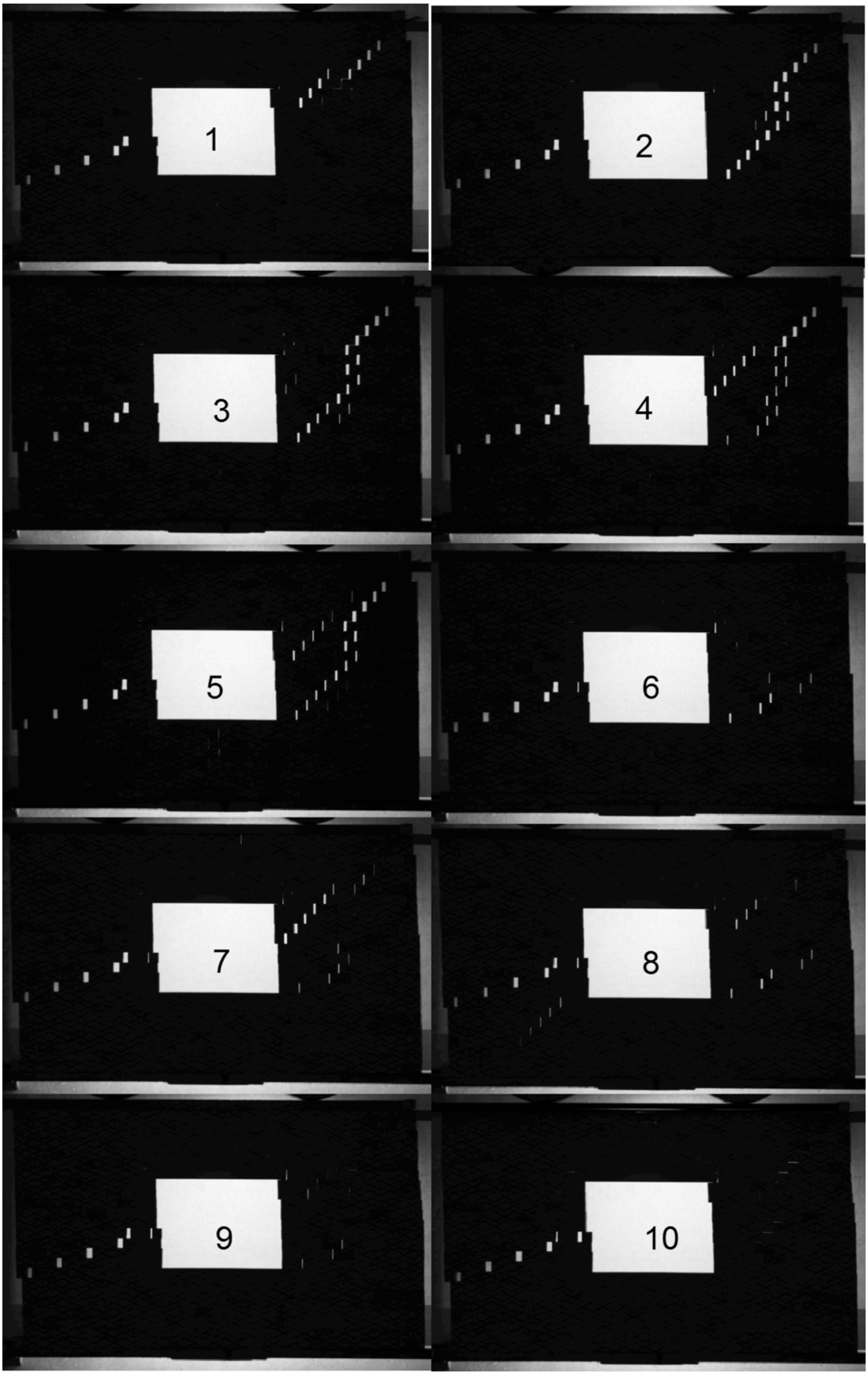

Figure A.8: Non-Glued wall-with-opening specimens at 8 mm base displacement

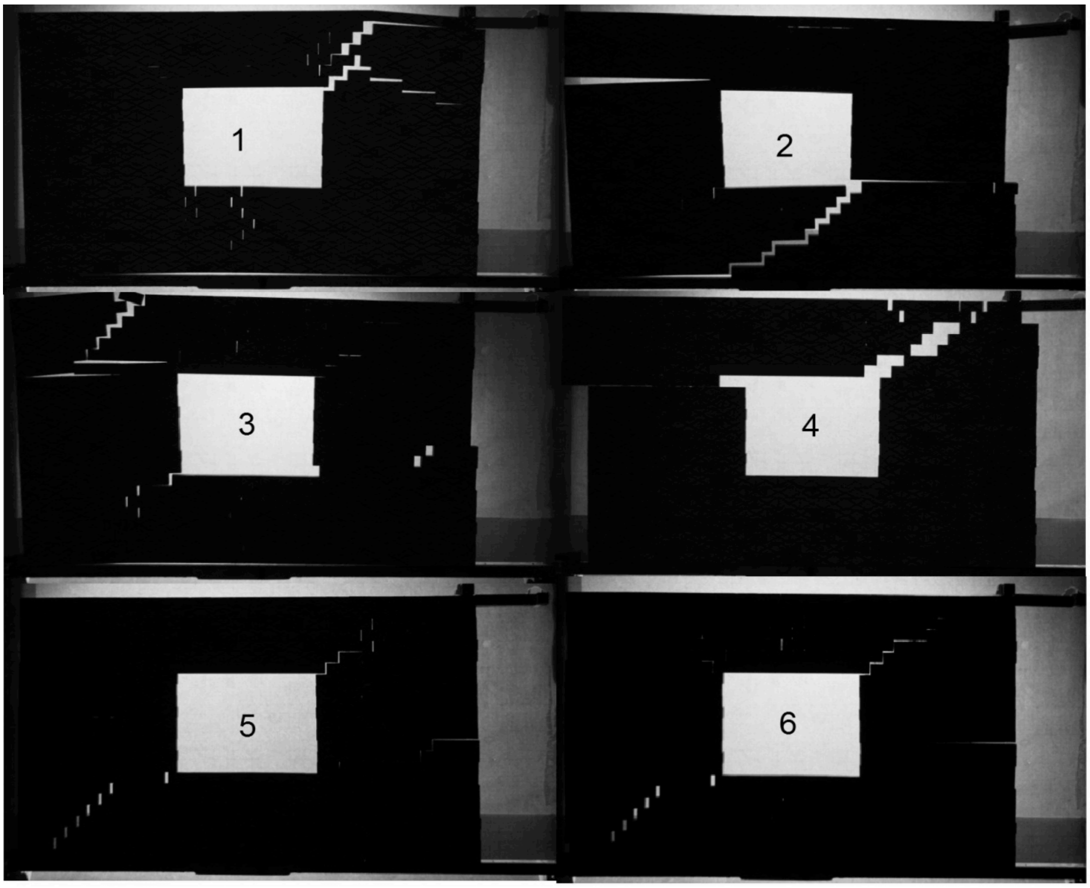

Figure A.9: Glued wall-with-opening specimens at 6 mm base displacement

Figure A.10: Retrofitted glued wall-with-opening specimens at 6 mm base displacement

Appendix-B

Matlab Code for Plotting Displacement Vectors

```
x = data(:,3);   %'x'-coordinates of brick mid-points from excel sheet named 'data'
y = data(:,4);   %'y'-coordinates of brick mid-points from excel sheet named 'data'
zx = data(:,5); %'Δx'-coordinates of brick mid-points from excel sheet named 'data'
zy = data(:,6); %'Δy'-coordinates of brick mid-points from excel sheet named 'data'
rangeY=floor(min(y)):2:ceil(max(y));
rangeX=floor(min(x)):2:ceil(max(x));
[X,Y]=meshgrid(rangeX,rangeY);
rangeZ=floor(min(zy)):.5:ceil(max(zy));
 B = imread ('021.jpg');
A = rgb2gray(B);
imshow (A)
imagesc([-604 0],[-369.3 0],flipud(A));
hold on;
hold on;
quiver(x,y,zx,zy,'color',[0 0 0]);
set(gca,'ydir','normal');
```

Appendix-C

Interview-1

Ref No.: Architect-001

Qualification: Bachelors in Architecture

1) How long have you been involved in the local construction industry?

Ans) 23 years (Government Official)

2) How many single family dwelling houses have you constructed/designed after 2005?

Ans) Altogether around 1000, after 2005 probably few hundred (directly involved or provided technical assistance)

3) How many of the houses you build were masonry type construction?

Ans) Most houses are block masonry with reinforced frame. After 2005 stone masonry is not used. Brick masonry is more dominant in southern regions such as Mirpur.

4) Do you provide all the structural drawings (with reinforcement details and material specifications)?

Ans) For structural design a structural design wing is present called CDO (Central Design Office), where as for small projects we design ourselves. For confined masonry (which is normally no more than two storey, in rural areas mostly single storey with pitched roofs due to high snowfall) we design ourselves. For structures with more than two storeys concrete frame is used and for that we seek assistance of our structure wing.

5) Which seismic strengthening/retrofitting measures you usually adopted and why?

Ans) Strengthening through pillar jacketing. If the cost of retrofitting is more than 30% of the structures costs then we do not recommend retrofitting. For cracks in wall, the wall is constructed again by jacking up the roof. Providing plinth beams and intermediate beams.

6) Do you carry out site investigation?

Ans) Mandatory for structures with 3 or more storeys. For smaller structures as a master plan of the region with zoning and micro zoning exist so we roughly know the bearing capacity.

7) Who normally procures the material?

Ans) Owner. Specification for material is given by us.

8) Do you carry out material testing during construction?

Ans) None

9) Which material and construction type you normally prefer for house construction and why?

Ans) Masonry for single or double storey structures with smaller room size because in far flung areas people can make their own cement blocks and hence lesser steel would have to be transported. For multi storey structures with larger spans RCC frame construction is preferred.

10) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) No one was mentally prepared and no seismic provisions used to be incorporated in the construction. Due to the prevailing contractual system there was a lack of proper monitoring.

11) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) After 2005 the building codes totally changed for the region.

12) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) 80% less damage. The 20% damage would be due to the departments not properly following the building code, no monitoring or site inspection, violation of bye laws and poor implementation, and plus not everyone can afford to get their house designed.

13) Is there any disaster management plan to cope up with future calamities?

Ans) A department is created at Director General level. They have food storage and general helping tools. People themselves are quite aware on how to act in future emergency. Govt. trained people, set up alarms and carried out drills. As a lot of rural area exist so everyone might not have got the training but still a general awareness exist.

Interview-2

Ref No.: Architect-002

Qualification: Bachelors in Architecture

1) How long have you been involved in the local construction industry?

Ans) 27 years (Government Official)

2) How many single family dwelling houses have you constructed/designed after 2005?

Ans) 50

3) How many of the houses you build were masonry type construction?

Ans) Majority were frame structures integrated with masonry.

4) How many houses were seismically designed and which code was followed?

Ans) The shape of the plan is now kept simpler. Due to mountainous terrain flat land is not available so most of the times we have to cut the land to make a terrace. In that case we have to go in more detail. As most of our clients were well-off therefore most of the structural aspects were well taken care off with regards to earthquakes resistance.

5) Do you carry out material testing during construction?

Ans) None

6) Which material and construction type you normally prefer for house construction and why?

Ans) Load bearing masonry structures mostly failed. Masonry is not that bad for earthquake it just needs to be constructed with proper care according to specifications and with proper bracings.

7) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) In small houses finances are the main factor. Earlier Muzaffarabad lied in zone 2. In government sector Wind loads and seismic loads were taken one at a time and the structure was designed for either one of them. In private sector structures were designed only for vertical loads. With new zoning Muzaffarabad now lies in zone 4.

Geologist around 15 years ago before earthquake suspected its occurrence and advised government to make safer structures but no attention was paid. As a result 70% of government buildings almost collapsed and the ones that survived suffered damage. Load bearing masonry structures mostly failed.

Stone masonry was most problematic. The gap between the two leaves/wythes of stone wall was filled with fine rubble instead of proper grout material. As a result the wall behaved as two separate leaves and thus failed. Stone masonry with proper grouting, opening lintels and jams survived the earthquake.

Private structures that collapsed had another reason of incremental construction i.e. floors were added after some time without any consideration of soil load bearing or footing capacity.

Also, the subsequent floors were projected outwards creating complex load path and structural unevenness. In many situations discontinuous columns were provided i.e. columns erected from first floor and not from the ground with proper footing.

8) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Done extensive courses of UNDP for building retrofit and strengthening. Personally have been careful even before 2005 because had a bit of know how about the geology of the region and the expectance of earthquake. 95% of personally designed structures survived 2005 earthquake.

British standards for block masonry suggest steel bars after every 4 to 5 layers of brick Buildings constructed that way with proper work quality can survive earthquakes up to a magnitude of 8. For bigger room sizes and openings we use confined masonry.

In 2005 buildings with magnificent and strong outlook failed whereas those that were not elegantly built and seemed to be of low quality survived the earthquake. On studying their details it was observed that owner, contractor or engineer/architect was wise enough to implement the standards.

Things have improved from 2005 but still not quite hopeful due to the abundance of unguided and unsupervised construction.

9) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Recent construction might survive 7 – 7.5 but beyond that things might get ugly. Government buildings might survive upto 8. In private sectors 70% of the houses might survive. In 95% of the structures some form of confinement is present. Whether the owner is educated or not they are using cement, reinforcement and concrete in their walls.

10) Is there any disaster management plan to cope up with future calamities?

Ans) Public awareness is periodic. Those who experienced 2005 earthquake might act in a more organized way. In 2005 no one was expecting an earthquake and no one was prepared whether public or government. Therefore no one had clue how to act. Everyone looked for help from the outside world. Now SDMA have been established and students at schools are trained for future such scenarios.

Interview-3

Ref No.: Architect-003

Qualification: Bachelors in Architecture

1) How long have you been involved in the local construction industry?

Ans) 10 years (Incharge Architect MDA)

2) How many of houses that you worked on were masonry type construction?

Ans) Pre 2005 100% masonry for single or double storey and frame structures for multi storey or commercial buildings. Post 2005 we have encouraged people to provide frames in houses as well. As a result 60% are now using RCC frames with block masonry. This shift was not on cost basis but due to earthquake where it was observed that frame structures survived and masonry failed especially stone masonry totally collapsed and brick masonry was better also brick in conjunction with frame behaved lot better.

3) For houses mention in previous question were designed by engineer?

Ans) Affording architect or engineer is difficult for most people. We try to guide them and provide typical designs. In commercial structures we force them to get a proper design by an engineer. Authorized architect's designs for single or double storey are accepted and approved even the reinforcement detailing. But for structure with more than 2 storeys we force to hire a structural engineer. There are no proper structural engineers in the region i.e. engineers who did masters in structural engineering. There are no private design companies in the region just 2-3 known professionals practicing privately but they too are not proper structural engineers.

4) Which seismic strengthening/retrofitting measures you usually adopted and why?

Ans) For retrofitting we look at slab condition and if it is manageable then we remove walls and provide beam and columns to restore it. Shear walls may be provided where needed. There are surviving structures which should be checked for safety as prescribed by the bye laws but due to shortage of staff this hasn't been achieved.

5) Do you carry out site investigation?

Ans) None

6) Do you carry out material testing during construction?

Ans) None

7) Which material and construction type you normally prefer for house construction and why?

Ans) Mix of masonry and frame is the new proposed method

8) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) Poor construction, old structures ageing more than 40 years were more affected. In terms of material stone masonry was most problematic it collapsed 100%. Lack of technical knowledge.

9) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Construction got better but only a little due to financial constraints. In city, all houses built after 2005 had their drawings approved by MDA. Local architecture used to have more wood but that is no longer happening. Concrete frame is in vogue.

10) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Chances are the new construction would perform better

11) Is there any disaster management plan to cope up with future calamities?

Ans) Government claims to have such plan and likewise SDMA has been established but personally I think if disaster of 2005 hits again the backup plan will go down the drain. Government has installed equipment that will blow sirens 30sec before the ground starts shaking, probably managed by SDMA. There had been 1 or 2 drills earlier.

Interview-4

Ref No.: Architect-004

Qualification: Bachelors in Architecture

1) How long have you been involved in the local construction industry?

Ans) 14 years

2) Which seismic strengthening/retrofitting measures you usually adopted and why?

Ans) Split and bandage for load bearing. Vertical bars provided.

3) Do you carry out site investigation?

Ans) None. But in our training we used to guide people how to estimate the BC.

4) Do you carry out material testing during construction?

Ans) None

5) Role of authorities in facilitating seismic resistant design?

Ans) HRCs (housing reconstruction centres) formed in April 2006. They were established at district level Muzaffarabad, Hattian, Patika, Rawalakot, Bagh, Tilkot, Haveli, Abbotabad. In start ERRA gave 3 techniques. It was a 10point checklist. It was focused only on reinforced load bearing structures with stone, block and brick.

These are demand driven applications and we cannot force people to do it. This region has no bricks and they were very costly because they had to be imported. So the locally made cement blocks were used. People had a predicament for stone masonry so they altogether avoided it. Therefore more focus on block masonry.

In reinforced masonry technique max room size allowed was 15 feet. At every corner 5/8 inches vertical bar were to be provided from the foundation. At plinth level 4 inch RCC band with 2 bars of ½ inches diameter was provided.

At every corner of window and opening a vertical bar from plinth beam had to be provided. Later it came that these vertical bars should be provided along the wall at every 4 feet distance. At sill level again RCC band of 3inches was to be provided. At room corners stitches made of steel and concrete of L or T shape were to be provided at 1.5 feet spacing. Complete RCC band again at lintel level and roof level. Similar recommendation for block, brick and stone. This technique was 100% implemented in rural areas but not in the city because in city the structures were no longer using load bearing construction. Structures should resist magnitude of 8 to allow safe evacuation time for the occupants.

Local timber frame structures known as dhajji served as a breakthrough. Prior to 2005 earthquake there were 2500 dhajji structures and they resisted the earthquake. After the program finished we had more than 125000 dhajji structures and a 10 point poster was made for this. During construction if a person reached 7 foot height and did not employ the specifications we asked him to apply suitable retrofit so that he can get paid by the government. For this we guided him.

Interview-5

Ref No.: Authority-001 Post: Assistant Director Works

Department: University Works Department (Mirpur)

Experience: 4-5 years on the Job.

1) How many single family dwelling houses in the city are masonry type constructions?

Ans) 40%.

2) How many of these houses are designed by an engineer?

Ans) 5%

3) Do engineers consider seismic design as well?

Ans) None at private level. And only 2-3% government buildings are designed considering earthquake. Region is classed as Zone 3.

4) If the structure is not designed by an engineer then who is in-charge of the structural detailing?

Ans) Contractor.

5) Is site investigation carried out prior to the construction and how do you regulate it?

Ans) None for private works.

6) Is material testing carried out?.

Ans) None at private level. Plus no labs in Mirpur, samples go to Taxila.

7) Who purchases material?

Ans) In 60% houses owner and in 40% contractor.

8) Is there any disaster management scheme for any such future earthquakes as 2005?

Ans) None.

Interview-6

Ref No.: Authority-002 Post: Chief Engineer

Department: Central Design Office, Muzaffarabad (CDO)

1) How many single family dwelling houses in and around Muzaffarabad are masonry type constructions?

Ans) 70% in rural with stone being dominant. 20% in city.

2) How many of these houses are designed by an engineer?

Ans) 10% in urban and none in rural. In city Municipal Corporation and MDA is responsible for housing design but no such body in rural areas.

3) Do you know of any seismic strengthening/retrofitting measures adopted in the region?

Ans) Providing reinforcement between masonry layers or using hollow block with vertical reinforcement i.e. reinforced masonry.

4) Is site investigation carried out prior to the construction and how do you regulate it?

Ans) None for single family dwellings. Only 5% of commercial buildings or multi storey structures have site investigation monitored by municipal corporation and MDA.

5) Is Material testing carried out?

Ans) None. There are no labs in the region except for the one at CDO. But, the regulations exist and the design is only approved by MDA when the material meets the specifications. Nespak conducted a study on the nearby quarries and listed their properties. They also prepared a guideline as to where these materials should be utilized. The purpose was to enable common man to use the locally available material which meets and the specifications and cost cheaper as well.

6) Reasons for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) Earlier there was no awareness for seismic resistant design. Pre 2005 most of the masonry houses weren't constructed according to code provisions or specifications e.g. stone size, erection, laying, mortar not meeting code standards. Those constructions in which code was followed 80% of those survived. Bricks had regular size as compared to stone masonry therefore they performed better. Now most people presume that masonry is inherently weak for earthquakes so they have shifted to concrete frames. Also the poor construction and material quality led to the large scale destruction.

7) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) 80% of the people moved from masonry to concrete frames on the assumption that it is safer. In rural areas 50% of the people do construction very carefully with proper work quality and they try to keep their structures light. There is no know how of the seismic detailing for RCC structures only the perception that RCC is safer over concrete. ERRA tried to train the local people for single storey houses in which they added corner reinforcements for masonry i.e. they tried to educate people for reinforced masonry.

8) How do you facilitate/regulate the implementation?

Ans) A seismic hazard map for Muzaffarabad is prepared which divides the city in different seismic zones. Also, a thrust line passing through the city is marked and 100 metres on either side of the thrust boundary is declared Redzone i.e. highly seismic and dangerous zone, and any construction of permanent nature is prohibited in that region. Only light weight or steel structure is recommended. We provide designs according to seismic provision of the code even for people who approach us for help with their private construction.

9) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Damage would be 70-75% less.

10) Is there any disaster management scheme for any such future events as 2005?

Ans) Government has established State Disaster Management Authority (SDMA) after the earthquake. They have different action plans for different types of emergencies e.g. flood, earthquake, land slide, etc.

Interview-7

Ref No.: Authority-003 Post: Sub Engineer

Department: University Works Department , Mirpur

Experience: 3.5 years on the Job.

1) How many single family dwelling houses in and around Muzaffarabad are masonry type constructions?

Ans) Maximum are frame structures in the city but in the outskirts and rural areas masonry is dominant.

2) How many of these houses are designed by an engineer?

Ans) Less than 5%

3) Do engineers consider seismic design as well?

Ans) After 2005 earthquake government buildings were constructed with earthquake consideration. But is case of private houses, as only a handful are structurally designed by an engineer therefore no such seismic resistant design is present. Only architectural drawings are available and the structural detailing is done by local masons or foreman.

4) If the structure is not designed by an engineer then who is in-charge of the structural detailing?

Ans) Local masons and Contractor.

5) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) The city is classed as Redzone i.e. high seismicity region, but people are not aware of it. In case of the city construction, it is quite safe but some structure might still fail because although they might be designed by an engineer but the people working on ground and even engineers themselves are not aware and trained for seismic design. Also, whatever is given in the drawings by the engineer is not exactly followed on site whether private or public sector.

Damage would be lesser as compared to Muzaffarabad because there had more stone masonry and that too with poor construction, whereas Mirpur has more brick masonry. Brick masonry behaves better as compared to stone masonry because in case of brick masonry the whole wall would not collapse only pieces would fall.

In case of Muzaffarabad damage would be 80% less. The remaining 20% damage would be due to poor site selection. Although the thrust boundary is marked and construction prohibited, the local authorities failed to implement this and people who had their lands in the Redzone have constructed there because they had no other option or land.

6) Is there any disaster management scheme for any such future events as 2005?

Ans) No such plan exists for now. If the close by Mangla dam breaks in any future earthquakes, Mirpur would not be affected because it is situated on the upstream but downstream cities such as Dina, Jhelum would be in great danger. In case of Muzaffarabad there might be something in the papers but on ground there is nothing.

Interview-8

Ref No.: Contractor-001 Company Size: C-6

Qualification: Matriculation

1) How long have you been involved in the local construction industry?

Ans) 7 Years in Kashmir and 32 years altogether.

2) How many single family dwelling houses have you constructed after 2005 earthquake?

Ans) 34

3) How many of these houses were masonry type construction?

Ans) 13 Houses were brick masonry. Rest were concrete frame.

4) How many of these houses were designed by an engineer?

Ans) As those 13 brick masonry houses were government projects so they all were engineer designed. For private works drawings were provided by owner and there was no engineer involvement.

5) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Yes if engineer is hired he provides the structural details.

6) Do engineers consider seismic design as well?

Ans) Yes.

7) Do you know of any seismic strengthening/retrofitting measures adopted in the region?

Ans) There were 6” beams provided throughout after every 3 feet.

8) How often is site investigation carried out prior to the construction?

Ans) Always

9) Who normally procures the material?

Ans) Contractor purchases but specifications given by the Govt. Department. Material is usually brought from Taxila or Islamabad, nothing is available locally.

10) How often material testing is carried out? (Engineer & Non-Engineered)

Testing is done for government projects

11) Where do you normally buy materials from or what factors you consider for material procurement?

Ans) Taxila or Islamabad, nothing is available locally.

12) Which material and construction type you normally prefer for house construction and why?

Ans) Concrete frame is preferred for its durability, strength and earthquake resistance. Whereas masonry for ease of construction.

13) In your view what was the reason for 2005 earthquake’s large scale destruction and what measures are being taken against that?

Ans) The region is mountainous and the earthquake itself was too severe.

14) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Work is durable and of higher quality. Plus there were no intermediate beams/horizontal RCC bands earlier. People have understood the implications of weak construction. NGOs and Govt. have provided training time to time.

15) Role of authorities in facilitating seismic resistant design?

Ans) Monitors and regulates work to maintain standards. Guidelines are given time to time.

16) Is there any disaster management plan to cope up with future calamities?

Ans) There was no such plan available when 2005 earthquake hit, but now the govt. has some plan.

Interview-9

Ref No.: Contractor-002 Company Size: C-5

Qualification: Intermediate in Faculty of Arts

1) How long have you been involved in the local construction industry?

Ans) 20 Years in Kashmir.

2) How many single family dwelling houses have you constructed after 2005 earthquake?

Ans) Own House after the previous one fell in 2005 which was stone masonry.

3) What type of construction you used for the house?

Ans) Concrete frame with block masonry.

4) Was it designed by an engineer?

Ans) Designed at Central Design Office.

5) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Yes.

6) Do engineers consider seismic design as well?

Ans) Seismic design was considered.

7) How often is site investigation carried out prior to the construction?

Ans) Most people cannot afford site investigation but those who can carry out site investigation.

8) Who normally procures the material?

Ans) For Govt. Projects it is purchased by contractor and approved by PWD. For private works either owner or contractor.

9) How often material testing is carried out? (Engineer & Non-Engineered)

Ans) For private works at most steel might be tested because that is more problematic.

10) Where do you normally buy materials from or what factors you consider for material procurement?

Ans) Crush from Margalla, Sand from Lawrencepur. It gets cheaper if you export materials from these places in bulk and plus they are of better quality than that available locally.

11) Which material and construction type you normally prefer for house construction and why?

Ans) Concrete frame is preferred for earthquake resistance. Those who cannot afford go for shelter type construction especially in rural areas in which they use concrete pillars in combination with wood panelling for walls and sheeting material for roofs.

12) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) Buildings not properly designed either Govt. or private. No one expected an earthquake. 90% of houses that fell were stone masonry. Brick masonry was satisfactory. Mostly frame structures survived.

13) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Framed Structure designed by Central Design Office. Most people are aware of the danger but some are still practicing the old construction habits. For those who have financial constraints would build 2 rooms instead of 4 or 6 but would not compromise on material or construction quality. They are less willing to take chances.

14) Role of authorities in facilitating seismic resistant design?

Ans) Nothing is done for under privileged and everyone is on their own. Initially they were involved and showed interest but now there is no monitoring of the constructions in the city and even worse in case of surrounding villages.

15) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Damage would be less as compared to the previous one.

16) Is there any disaster management plan to cope up with future calamities?

Ans) Nothing.

Interview-10

Ref No.: Contractor-003 Company Size: C-3

Qualification: N/A

1) How long have you been involved in the local construction industry?

Ans) Since 1978. (35 years)

2) How many single family dwelling houses have you constructed after 2005 earthquake?

Ans) Constructed 102 Schools for govt. after 2005 all frame structures. Design and consultancy by NESPAK (National Engineering Services Pakistan)

3) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Yes.

4) Do engineers consider seismic design as well?

Ans) Seismic design was considered.

5)Do you know of any seismic strengthening/retrofitting measures adopted in the region?

Ans) Strengthening columns by adding reinforcement and increasing its size. For roofs reinforcement is laid again and concreted. For walls showing cracks they would be constructed again.

6) Which of these methods you prefer and which you don't? Explain.

Ans) Retrofitting is deemed expensive so people prefer to go with new construction.

7) How often is site investigation carried out prior to the construction?

Ans) People either have no awareness about it or else they are not willing to spend on it. On govt. projects soil investigation is done at some places and their bearing capacities are given. Entire region is hilly therefore slopes can't be avoided and plain land is quite expensive. In rural areas people base their site selection on their experience.

8) Who normally procures the material?

Ans) Contractor.

9) How often material testing is carried out? (Engineer & Non-Engineered)

Ans) In government works material testing is carried out. Steel is tested at UET taxila.

10) Which material and construction type you normally prefer for house construction and why?

Ans) For single storey houses masonry is fine but for 2 or more storeys frame is necessary.

11) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) People were not expecting an earthquake and that too of such intensity. Rural area altogether lack engineers.

12) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) MDA should provide design services for local people constructing their house. People are much aware now and are paying more attention to the seismic considerations in their new construction.

13) Role of authorities in facilitating seismic resistant design?

Ans) Authorities are not as keen and active as they should be.

14) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Damage would be minor.

15) Is there any disaster management plan to cope up with future calamities?

Ans) No Plan available. State Disaster Management Authority(SDMA) was established in the region but there seem to be no visible activity. People would eventually be on their own plan and their past knowledge of previous earthquake.

Interview-11

Ref No.: Contractor-004 Company Size: N/A

Professional Qualification: Primary Schooling

1) How long have you been involved in the local construction industry?

Ans) 10.5 years local, 23 years altogether

2) How many single family dwelling houses have you constructed after 2005 earthquake?

Ans) 20-25

3) How many of these houses were masonry type construction?

Ans) None. All concrete frames with block masonry

4) How many of these houses were designed by an engineer?

Ans) All

5) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Yes. But keeping in view the financial constraints of owner alterations are made by the contractor on his own judgement. Contractor endeavours to follow the reinforcement or material specifications given in the drawings but in case where they are missing he uses his own judgement. Inspects sites once or twice or even more depending on how much the owner can afford.

6) Do engineers consider seismic design as well?

Ans) Given by engineer but not everyone can afford

How often is site investigation carried out prior to the construction?

Depends if the owner can afford it or not. Otherwise the contractor uses their personal judgement to decide the condition of soil in order to set the depth of footing. For loose soil the footing goes deeper.

7) Who normally procures the material?

Ans) Contractor

8) How often material testing is carried out? (Engineer & Non-Engineered)

Ans) No testing in private housing works.

9) Where do you normally buy materials from or what factors you consider for material procurement?

Ans) Locally available material. Only 10 % can afford good quality material the remaining 90% have no choice but to take the risk.

10) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Instead of discontinuous lintels over openings continuous band is provided over all walls. Continuous Plinth beams over all walls have been introduced after earthquake and also beams at roof level. Column stirrups used to be at 1' or 9" spacing now they are kept at 4"-6" spacing. Concrete ratio is ensured at 1:2:4 Increased number of columns (no consideration of architectural aspects with regards to seismic vulnerability)

11) Role of authorities in facilitating seismic resistant design?

Ans) Local authorities such as MDA should send their engineers to visit local house construction sites to ensure that approved drawings and specified guidelines are being followed. This way a common man would not be burdened with the expense of arranging a private engineer for site inspection.

12) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Old buildings that have been refurbished are more vulnerable to earthquake damage.

13) Is there any disaster management plan to cope up with future calamities?

Ans) People have no resources to cope with such disasters. No such plan or awareness created by Govt.

Interview-12

Ref No.: Contractor-005 Company Size: N/A

Professional Qualification: N/A

1) How long have you been involved in the local construction industry?

Ans) 15 years

2) How many single family dwelling houses have you constructed after 2005 earthquake?

Ans) 400-500

3) How many of these houses were masonry type construction?

Ans) None. All concrete frames with block masonry. Mostly government buildings use brick masonry(90%) and in private works mostly block masonry is preferred as brick are too costly.

4) How many of these houses were designed by an engineer?

Ans) No idea who the engineer is but the drawings are approved by MDA and NOC is issued to the owner.

5) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Reinforcement details are given in the drawings and followed likewise.

6) How often is site investigation carried out prior to the construction?

Ans) No site investigation

7) Who normally procures the material?

Ans) Owner

8) How often material testing is carried out? (Engineer & Non-Engineered)

Ans) No testing in private housing works.

9) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) Laps were not provided with sufficient length.

Material quality was low. Beam joint were not proper Less material provided in terms of reinforcement and concrete ratio. Most damage was suffered by stone masonry because they were designed for gravity loads only and it was after the occurrence of 2005 earthquake that lateral loads were recognised. The space between the two leaves of stone wall were not properly grouted and as a result of lateral shaking these walls easily disintegrated and collapsed. For basements and especially the walls against hill slopes fully reinforced concrete wall is provided by people who can afford.

10) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Reinforcement has increased. Consideration for lateral actions. Strict adherence to concreting standards with a ratio of 1:2:4. Stirrups shape and slight inward bend at the ends. Main reinforcement used to be #4 now #5 or #6 is used. Rings used to be #2 now #3 is used. Lap lengths have increased.

11) Role of authorities in facilitating seismic resistant design?

Ans) Local authorities such as MDA should send their engineers to visit local house construction sites to ensure that approved drawings and specified guidelines are being followed. This way a common man would not be burdened with the expense of arranging a private engineer for site inspection.

12) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Damage would be less compared to 2005

13) Is there any disaster management plan to cope up with future calamities?

Ans) There were workshops on first aid techniques but after about a year the NGOs left and since then nothing is been done. No disaster management wing in the area by government.

Interview-13

Ref No.: Contractor-006 Company Size: N/A

Professional Qualification: N/A

1) How long have you been involved in the local construction industry?

Ans) 3 years

2) How many of the houses you build were masonry type construction?

Ans) None. All concrete frames with block masonry.

3) How many of these houses were designed by an engineer?

Ans) Some had engineers but most were designed and detailed by contractor himself and he also made sure the drawings got approved from authorities.

4) How often is site investigation carried out prior to the construction?

Ans) Contractors know the region well and base their decisions on their knowledge and experience. Flat land is scarce therefore most of the houses are built on hill slopes with cut and fill.

5) How often material testing is carried out? (Engineer & Non-Engineered)

Ans) No testing in private housing works.

6) Which material and construction type you normally prefer for house construction and why?

Ans) Concrete performed satisfactory in 2005 earthquake; even if the structure collapsed it gave enough time for the people to evacuate the building. Stone masonry on the other hand was very problematic. Barely any stone masonry structure survived and they also didn't give enough time for evacuation.

7) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Post 2005 construction is more durable with increased amount of steel reinforcement. For 3-storey the footing is laid at a depth of 7.5 feet. Plan is kept simple and square.

8) Role of authorities in facilitating seismic resistant design?

Local authorities such as MDA should send their engineers to visit local house construction sites to ensure that approved drawings and specified guidelines are being followed. This way a common man would not be burdened with the expense of arranging a private engineer for site inspection.

9) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Future disaster would be less in comparison to 2005 earthquake because of the local masses are aware of the possible earthquake threats and are taking measures to ensure good quality construction.

10) Is there any disaster management plan to cope up with future calamities?

Ans) People are aware and mentally prepared. There happens to be a local authority for disaster management in the region but they hardly have any activity.

Interview-14

Ref No.: Contractor-007 Company Size: N/A

Professional Qualification: N/A

1) How long have you been involved in the local construction industry?

Ans) 11 years

2) How many of the houses you build were masonry type construction?

Ans) None. All concrete frames with block masonry.

3) How many of these houses were designed by an engineer?

Ans) In urban area drawings come from an engineer and construction is carried according to them. But in rural areas there is no engineer. Construction is based on financial constraints, but the quality of the work is tried to be maintained. If the budget is limited then construction is carried out in parts rather than going for the complete structure with cheaper quality of work and material.

4) How often is site investigation carried out prior to the construction?

Ans) None.

5) Who normally procures the material?

Ans) Owner

6) How often material testing is carried out? (Engineer & Non-Engineered)

Ans) No testing in private housing works. Only concrete ratio is maintained and checked on scale.

7) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Now with the introduction of plinth beams all through the structure, it behaves as a box and the in case of earthquake the whole structure would move altogether rather than in pieces (i.e. no differential movement). Earlier there used to be no reinforcement in the footing and the column reinforcements were vertically rested on the footing pad. Now they provide footing reinforcement and the column reinforcement is spread on to it to create lap joint. Site with good load bearing and compacted soil is selected and preferably flat too. Plan is kept simple and people are spending more on strength and durability rather than decor and outlook.

8) Is there any disaster management plan to cope up with future calamities?

Ans) No plan given by local authority.

Interview-15

Ref No.: Contractor-008 Company Size: Class-A

Professional Qualification: N/A

1) How long have you been involved in the local construction industry?

Ans) 11 years

2) How many single family dwelling houses have you constructed after 2005 earthquake?

Ans) 400-500

3) How many of these houses were masonry type construction?

Ans) In private sector most were concrete frames with block masonry. Brick masonry is most used in Government buildings. Some of the brick masonry structures had columns, but most didn't. They have more chances of survival with columns.

4) How many of these houses were designed by an engineer?

Ans) Before 2005 there hardly used to be any engineer involvement, but now the owner needs to get their drawing approved from the local authority and acquire NOC.

5) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Yes if engineer is hired. In backward and poorer community layout and reinforcement is decided by the contractor.

6) How often is site investigation carried out prior to the construction?

Ans) None. Only in govt works

7) Who normally procures the material?

Ans) Owner

8) How often material testing is carried out? (Engineer & Non-Engineered)

Ans) No testing in private housing works. Only in govt works.

9) Which material and construction type you normally prefer for house construction and why?

Ans) Brick masonry with steel ties. Cracking was less for brick masonry with confinements during 2005 earthquake. Also masonry is easier to construct. Stone masonry was most problematic. Reason not identified.

10) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Earlier there used to be no reinforcement in footing/basement. Footing depths used to be 1.5-3 feet. Now they are at 6-7 feet depth. Column reinforcement is almost twice now. Stirrup spacing used to be 1.25-1.5 feet. Stirrups on laps are kept now to 4" spacing. 1:2:4 concrete for private works. 1:1.5:3 concrete for government works. People now just don't hire any contractor, they go for reputed ones. Earlier hill slopes were cut to a straighter angle and construction was carried adjacent to the cut face without providing any gap between the structure and the hill slope. Now not only a gap of 2-3 feet is provided but the reinforced concrete is provided to stabilize the cut hill face.

11) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Damage would be less because people would construct smaller structure if they are short on finances but wouldn't compromise on material.

12) Is there any disaster management plan to cope up with future calamities?

Ans) Apparently no backup plan to counter similar disaster of 2005. The only idea is to run to an open field or plain land. No awareness or plan provided by govt.

Interview-16

Ref No.: Contractor-009 Company Size: C-3

Professional Qualification: Bachelors in Architecture

1) How long have you been involved in the local construction industry?

Ans) 10 years. (Mirpur)

2) How many single family dwelling houses have you constructed after 2005 earthquake?

Ans) More than 100 private houses built.

3) How many of these houses were masonry type construction?

Ans) 3% confined brick masonry. 97% unconfined brick masonry.

4) How many of these houses were designed by an engineer?

Ans) 3-7% are designed by an engineer but in case of commercial buildings 90% are designed by engineer. In private houses they usually don't have engineer involvement and they only follow a set pattern for layout.

5) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Yes if engineer is hired.

6) If the structure is not designed by an engineer then who is in-charge of the structural detailing?

Ans) Contractor is responsible for the structural design i.e. member sizes and reinforcement detailing.

7) Do you know of any seismic strengthening/retrofitting measures adopted in the region?

Ans) Providing columns and continuous lintels and plinth RCC bands. Filling cracks with grout.

8) How often is site investigation carried out prior to the construction?

Ans) None. Only in govt works

9) Who normally procures the material?

Ans) Mostly contractor.

10) How often material testing is carried out? (Engineer & Non-Engineered)

Ans) No testing in private housing works. Only in govt works.

11) Where do you normally buy materials from or what factors you consider for material procurement?

Ans) Priority is given to cost but main aim is to have a balance of price and quality for the type of work at hand. For example Mangla crush has average strength and is cheaper where as Margalla crush is stronger as well as more expensive.

12) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) There was no awareness. Buildings were not constructed with seismic consideration. Even if we had the threat of an earthquake and were we in a position to spend on it?

13) Identify any changes in the construction practices/attitude after the 20S05 earthquake?

Ans) 90% change in construction attitudes. Most people endeavour to build stronger houses but those who cannot afford are still taking the risk of cheap and unreliable construction. Most people try to provide confinement or atleast partial confinement based on their finances. 50% are providing plinth beams. No other awareness with regards to simpler building plans or architecture. Only method deemed safer against earthquake is to provide beams and columns.

14) Role of authorities in facilitating seismic resistant design?

Ans) Site investigation, material testing and soil testing is very expensive for an individual who is building his house, therefore govt should facilitate this. For instance the govt department should have a log of soil properties in the region or a bearing capacity map should be prepared which gives the variation of soil bearing capacity and soil properties in the region. Also specified structure types should be made compulsory for different soil type.

Secondly their officials must visit construction sites and ensure that the building codes are being followed (supervision & implementation). Implementation has improved to some extent due to public awareness, whereas local authorities make no checks on implementation due to the lack of staff. Plus the local architects or engineers are not paid much on small house projects so they are least bothered to inspect the site. Local authorities have the laws and regulations but are short on funds for implementation.

15) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Damage would be around 50% less.

16) Is there any disaster management plan to cope up with future calamities?

Ans) General awareness should be created with media, seminars, social gatherings. Government has no social setup to create awareness. A booklet was published and distributed in schools. Emergency exits with regards to occupancy and structure type to facilitate safe and quick exit are not considered for private constructions, but provided in case of government buildings.

Interview-17

Ref No.: Contractor-010 Company Size: C-5

Professional Qualification: N/A

1) How long have you been involved in the local construction industry?

Ans) 10 years. (Mirpur)

2) How many single family dwelling houses have you constructed after 2005 earthquake?

Ans) 2.(own house)

3) How many of these houses were masonry type construction?

Ans) Both were brick masonry with concrete frame. Plinth beams and columns provided.

4) How many of these houses were designed by an engineer?

Ans) Both

5) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Yes.

6) Do engineers consider seismic design as well?

Ans) No consideration for seismic resistant design

7) How often material testing is carried out? (Engineer & Non-Engineered)

Ans) Only steel is tested. Mostly non graded steel is used.

8) Which material and construction type you normally prefer for house construction and why?

Ans) Concrete frame because if a wall fails the whole structure won't collapse where as in load bearing construction the whole building might collapse with the failure of load bearing wall.

9) Identify any changes in the construction practices/attitude after the 20S05 earthquake?

Ans) None

10) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Damage would be around 50% less because more concrete frame structures are constructed after 2005.

11) Is there any disaster management plan to cope up with future calamities?

Ans) None.

Interview-18

Ref No.: Labour-001 Post: Labour-Helper

1) How long have you been involved in the local construction industry?

Ans) 30 years(Muzaffarabad)

2) How many single family dwelling houses have you worked on after 2005 earthquake?

Ans) 200

3) How many of these houses were masonry type construction?

Ans) All concrete frames

4) How many of these houses were designed by an engineer?

Ans) After 2005 earthquake all built according to engineered drawings

5) How often material testing is carried out?

Ans) Yes

6) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) Old structures, Poor maintenance, Bad work quality, No plinth beam provided

7) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Better material and better concrete ratio with better reinforcement.

8) Role of authorities in facilitating seismic resistant design?

Ans) No interest from the authority but the hired engineers sometimes visits.

9) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) 100% better

10) Is there any disaster management scheme for any such future events as 2005?

Ans) People with experience have the knowledge now and would act in an organized manner, but nothing from the government side.

Interview-19

Ref No.: Labour-002 Post: Formwork Fixer

1) How long have you been involved in the local construction industry?

Ans) 4-5 years(Muzaffarabad)

2) How many single family dwelling houses have you worked on after 2005 earthquake?

Ans) 200-300

3) How many of these houses were masonry type construction?

Ans) None

4) How many of these houses were designed by an engineer?

Ans) All

5) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Drawings were provided and every detail given

6) Which seismic strengthening/retrofitting measures are usually adopted?

Ans) No idea we just follow the drawings

7) Who normally procures the material?

Ans) Owner

8) How often material testing is carried out?

Ans) Contractor carries out material testing

9) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Own house fell in Hattian Bala and now reconstructed using dhajji with stone and mud wall filling plus sheets attached on to it.

10) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Work is better but can't say for sure

11) Is there any disaster management scheme for any such future events as 2005?

Ans) None

Interview-20

Ref No.: Labour-003 Post: Mason

1) How long have you been involved in the local construction industry?

Ans) 15-16 years(Muzaffarabad)

2) How many of these houses you constructed were masonry type construction?

Ans) 20-25 in Muzaffarabad city were brick masonry. For 9 inch walls no columns were provided, but for 4.5 inch walls columns were provided.

3) How many of these houses were designed by an engineer?

Ans) All

4) Which seismic strengthening/retrofitting measures are usually adopted?

Ans) No idea we just follow the drawings

5) How often is site investigation carried out prior to the construction?

Ans) 3 ft deep footing for hard soil and 4 ft or more for soft soils. Engineer makes the decision.

6) Who normally procures the material?

Ans) Owner or sometimes contractor

7) Where materials are usually purchased from and what factors are considered?

Ans) Depends on the work. For small works the nearest available material is used but for bigger works better material with better rates is preferred.

8) Which material and construction type are normally preferred for house construction and why?

Ans) Concrete frame for its strength and masonry for its ease of construction. In comparison to bricks block is more preferred as lesser number of blocks are required due to their bigger size in comparison to brick and is cheaper as well.

9) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) Brick masonry structures collapsed more in comparison to concrete frame structures; the only reason for brick masonry failed was because the bricks were not properly soaked before being laid and even after laying proper curing was not carried out. In stone masonry every layer should have atleast 2-3 bigger sized stones that connect the two leaves together. The fill between the two wythes should be proper. Saw tooth joints were used earlier as well.

10) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Shape of the stirrups rings changed. Now bricks are left overnight in a water tank for them to properly absorb all the water before being laid. Length of the overlap increased from 1 ft to 1.5-2 ft. 6 reinforcement bars in columns instead of 4.

11) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Work is better so damage would be less.

12) Is there any disaster management scheme for any such future events as 2005?

Ans) Government is prepared and people are also prepared for future disasters. Major development is that now people know they have to move out of the buildings into the open.

Interview-21

Ref No.: Labour-004 Post: Mason

1) How long have you been involved in the local construction industry?

Ans) 9 years (Hattian)

2) How many single family dwelling houses have you worked on after 2005 earthquake?

Ans) 500

3) How many of these houses were masonry type construction?

Ans) 1%

4) How many of these houses were designed by an engineer?

Ans) All

5) In your view what was the reason for 2005 earthquake's large scale destruction and what measures are being taken against that?

Ans) No columns in masonry especially stone masonry as a result stone masonry structures collapsed most.

6) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) 60% less. People living on mountains at higher altitudes still do not follow column practice.

7) Is there any disaster management scheme for any such future events as 2005?

Ans) Might have to live in tents again but no idea where they'll come from. No government activity noted in this regard.

Interview-22

Ref No.: Labour-005 Post: Labour-Helper

1) How long have you been involved in the local construction industry?

Ans) 1 years (Mirpur) used to work in Karachi earlier.

2) How many of these houses were masonry type construction?

Ans) Around 60% of houses in the region have brick masonry.

3) How many of these houses were designed by an engineer?

Ans) Structures designed by engineer

4) Do the Engineers provide all structural drawings (with reinforcement details and material specifications)?

Ans) Engineer provides all drawings to contractor.

5) Identify any changes in the construction practices/attitude after the 2005 earthquake?

Ans) Immense difference in construction practice between Mirpur and Karachi city. Material is different. 3 wheelbarrows of sand for 1 bag of cement for mortar for block work (Karachi). 2 wheelbarrows of sand in 1 bag of cement for brick work (Mirpur). Where there is a danger concrete wall is used instead of masonry.

Interview-23

Ref No.: Labour-006 Post: Electrician

1) How long have you been involved in the local construction industry?

Ans) 3 months (Mirpur) used to work in Iran earlier.

2) In your opinion if a similar calamity occurs as 2005 earthquake how bad would be the damage?

Ans) Government buildings are strong. Beams are strong to the extent that a nail when driven into the beam would get bent. Concrete ratio is of higher quality.

3) Is there any disaster management scheme for any such future events as 2005?

Ans) No disaster management and no awareness with regards to construction or disaster management

Appendix-D

Individual Segment Elongation for PP-band Tensile test

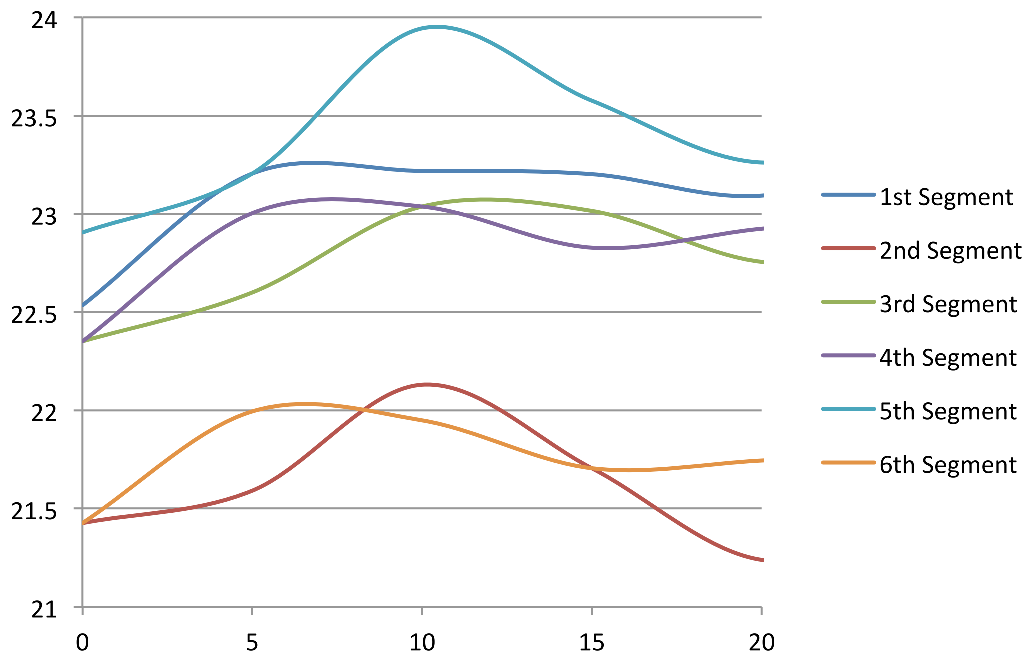

Figure D.1: Study of individual segment elongation for PP-band tensile test-150mm-1

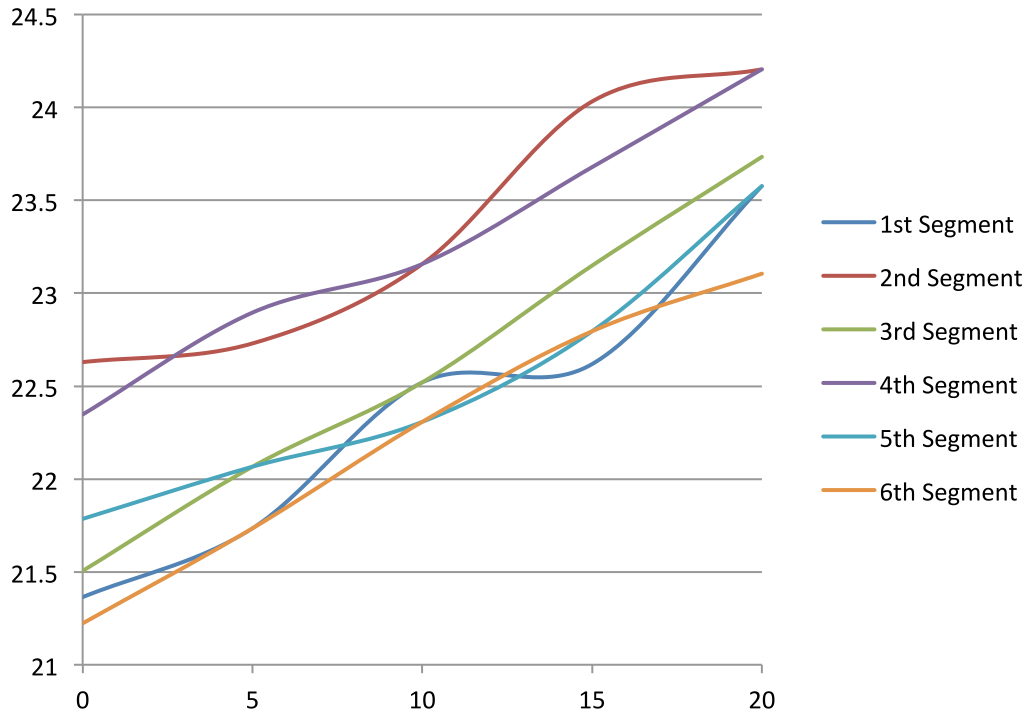

Figure D.2: Study of individual segment elongation for PP-band tensile test-150mm-2

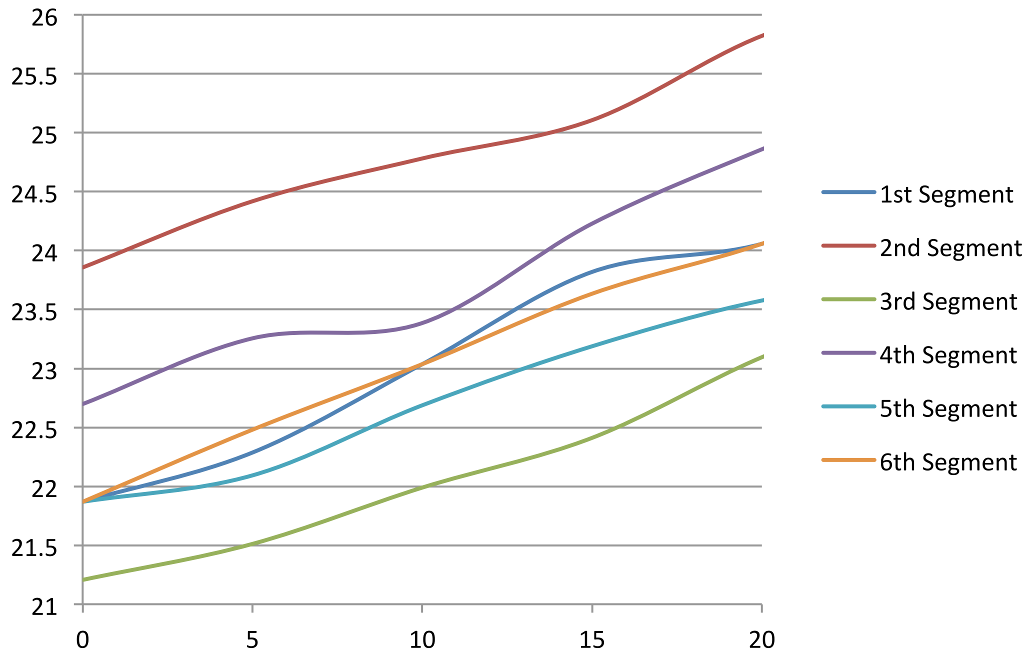

Figure D.3: Study of individual segment elongation for PP-band tensile test-150mm-3

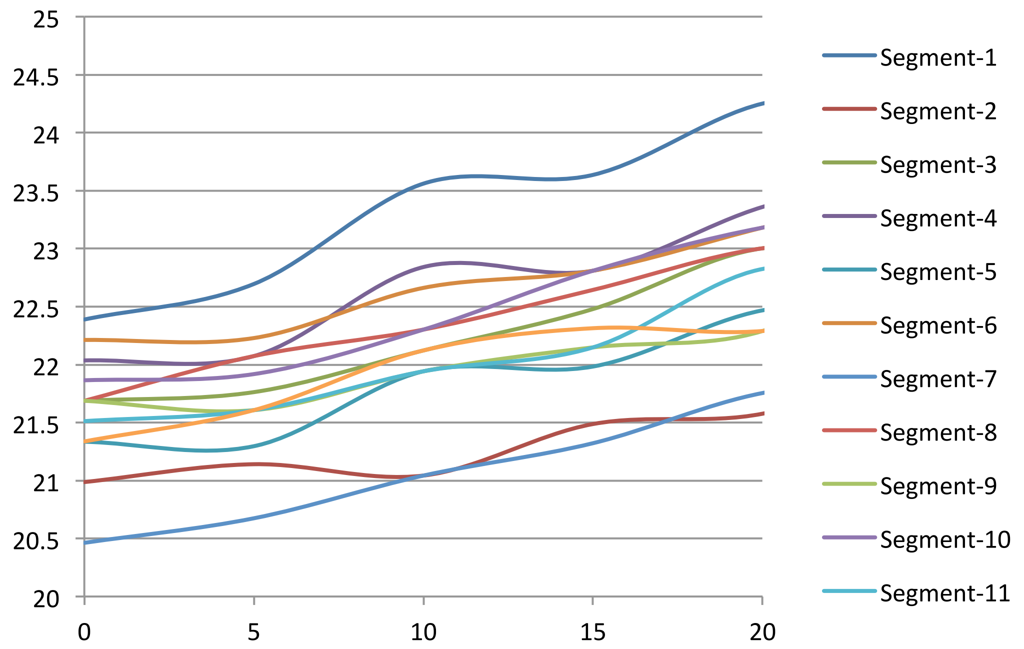

Figure D.4: Study of individual segment elongation for PP-band tensile test-300mm-1

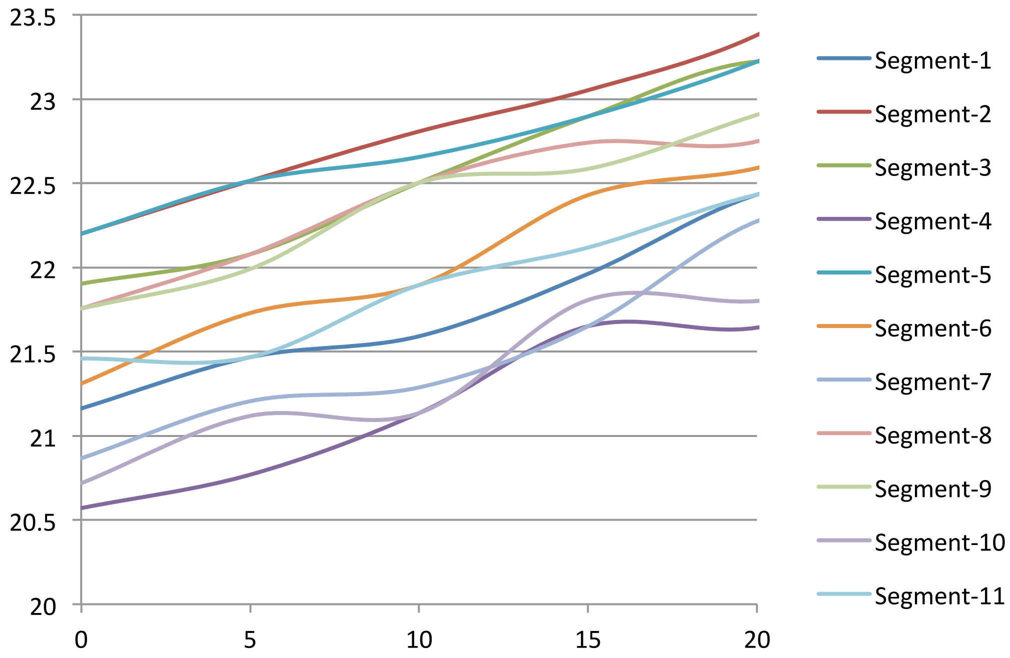

Figure D.5: Study of individual segment elongation for PP-band tensile test-300mm-2

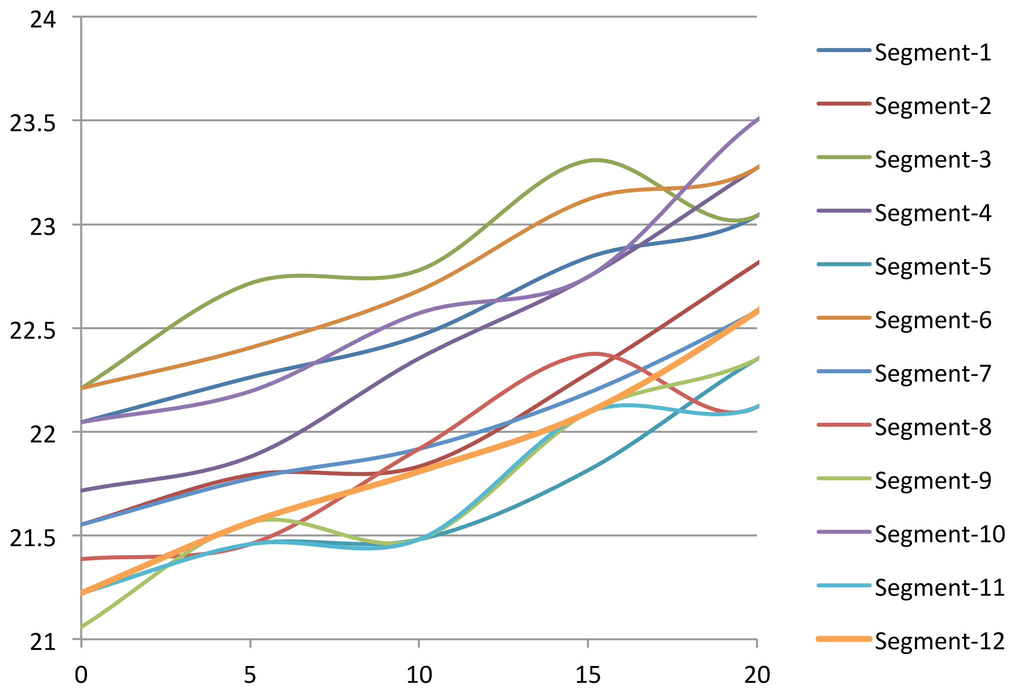

Figure D.6: Study of individual segment elongation for PP-band tensile test-300mm-3

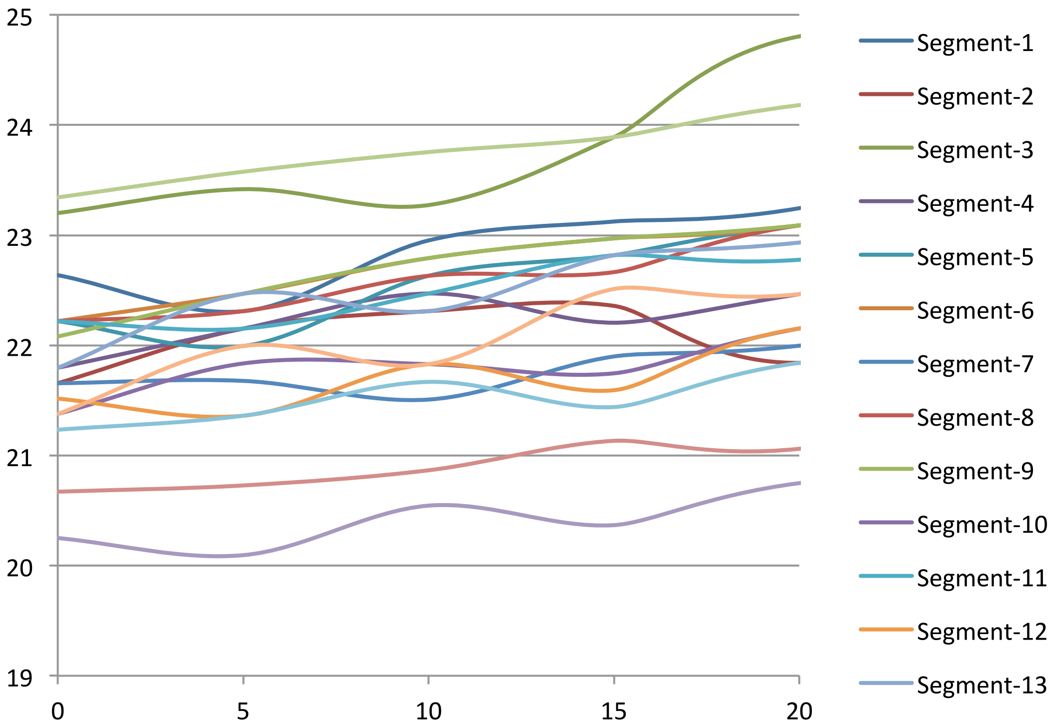

Figure D.7: Study of individual segment elongation for PP-band tensile test-450mm-1

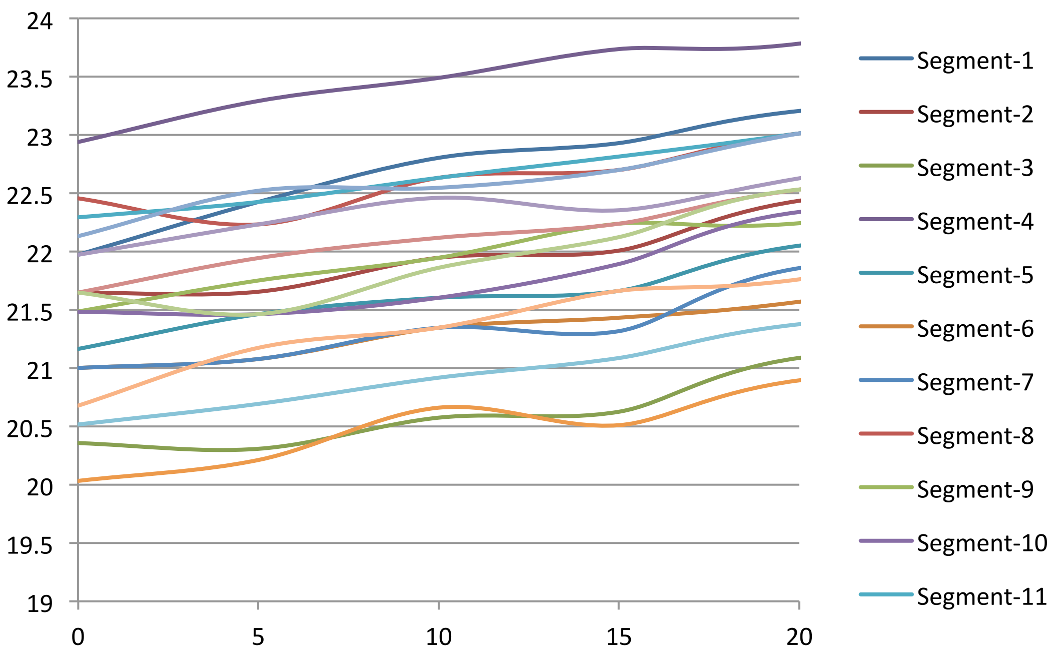

Figure D.8: Study of individual segment elongation for PP-band tensile test-450mm-2

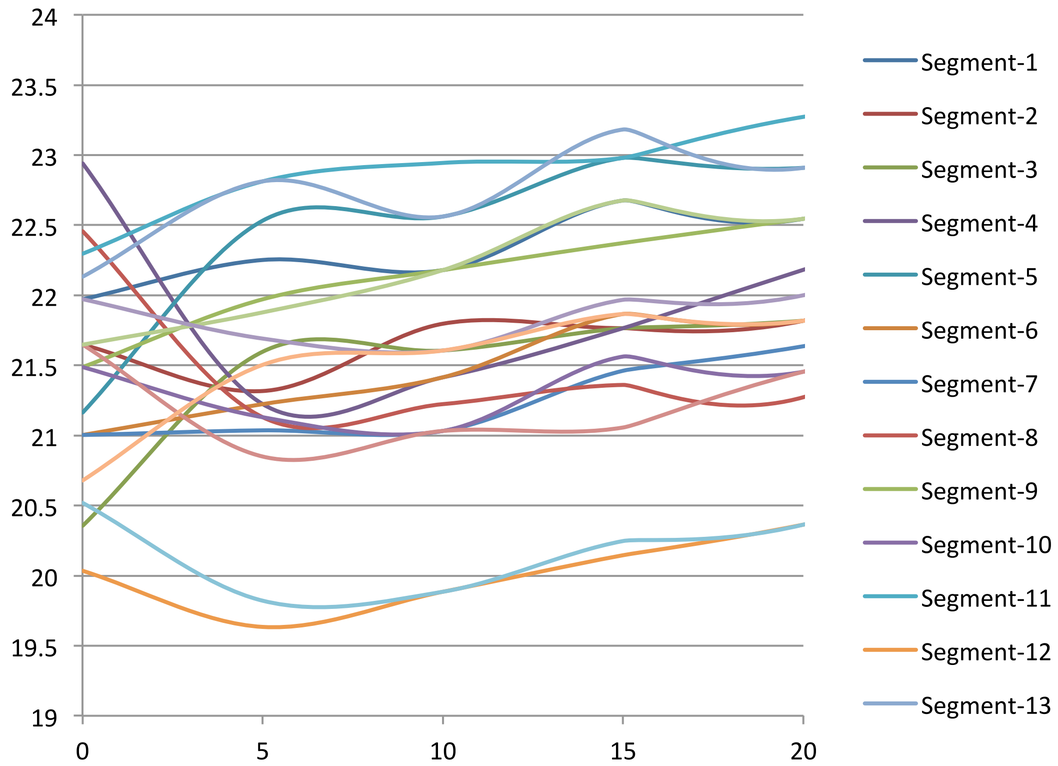

Figure D.9: Study of individual segment elongation for PP-band tensile test-450mm-3

Appendix-E

Parametric Study for Maximum Nominal Stress

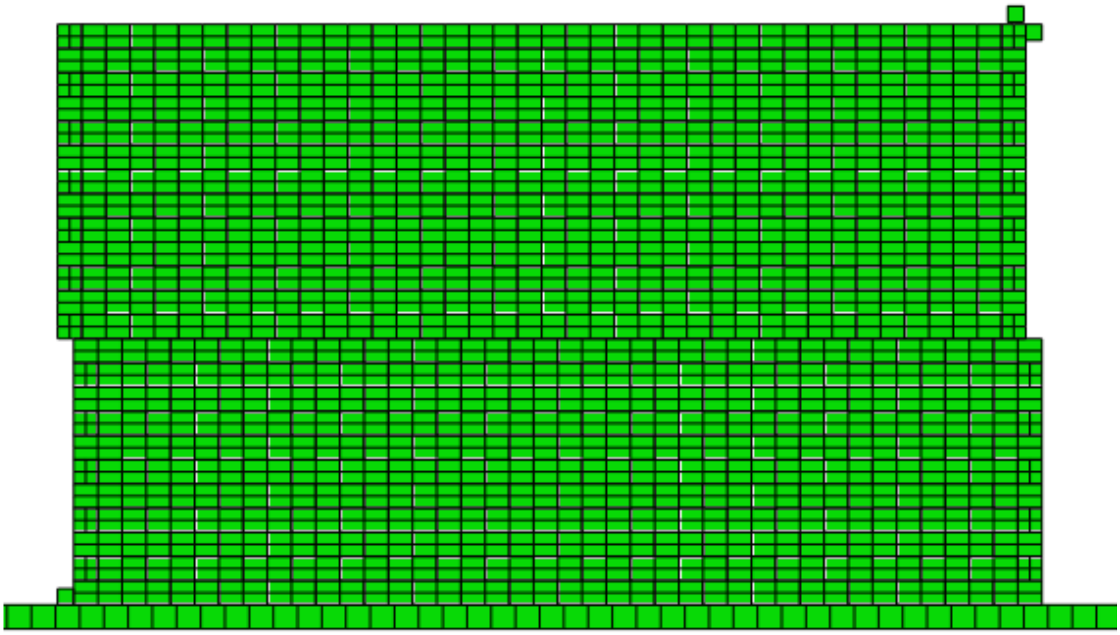

Figure E.1: Wall deform geometry for shear stress value at 1e5 N/m²

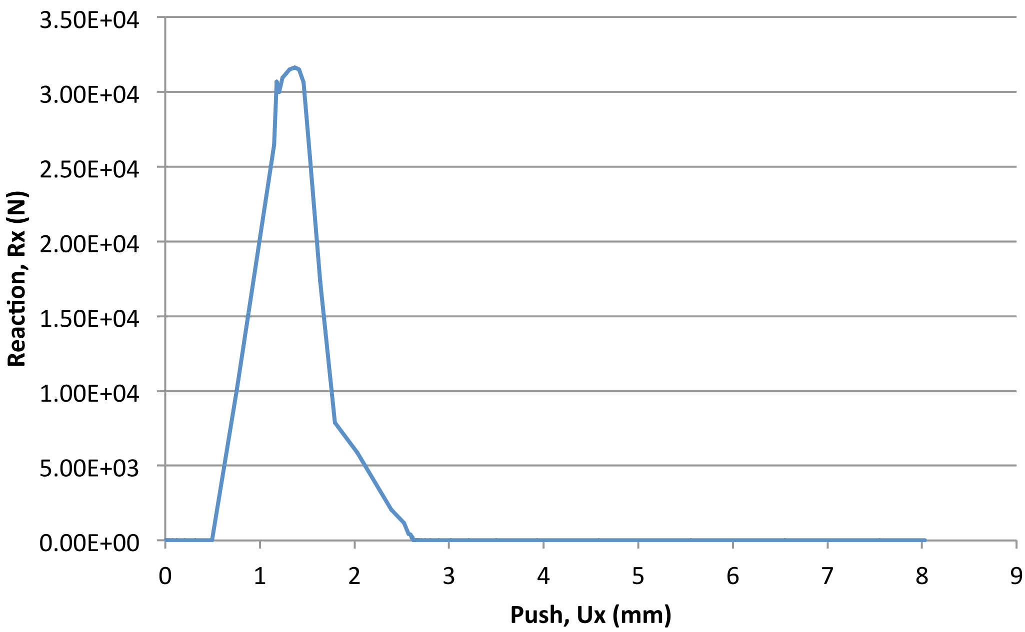

Figure E.2: Lateral reaction vs. base displacement for shear stress value at 1e5 N/m²

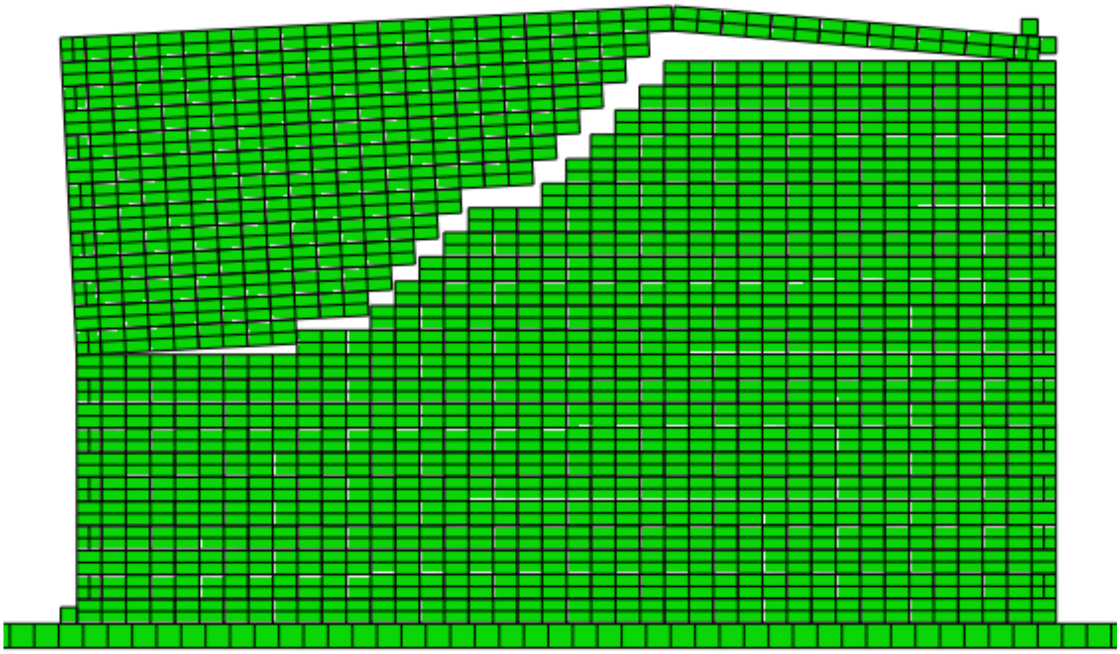

Figure E.3: Wall deform geometry for Shear stress value at 1e6 N/m^2

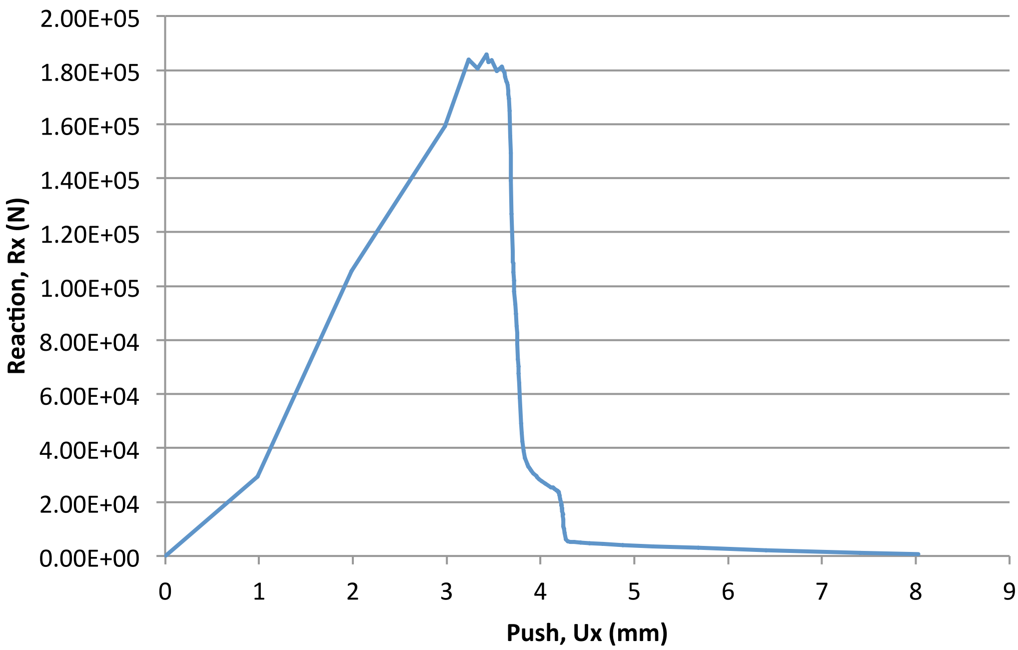

Figure E.4: Lateral reaction vs. base displacement for shear stress value at 1e6 N/m^2

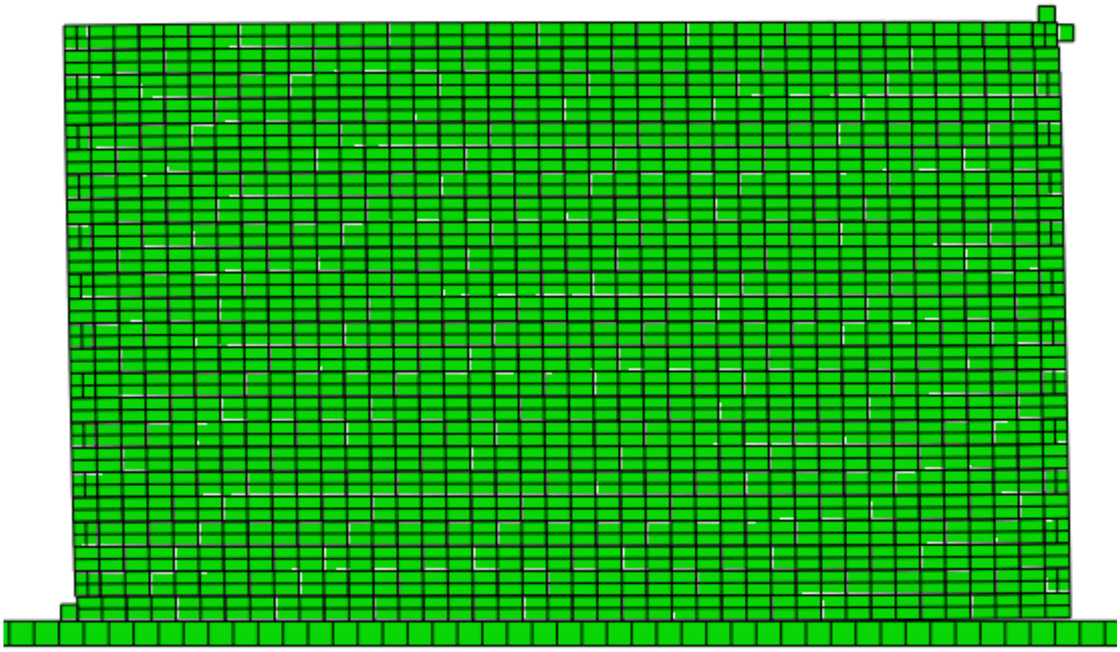

Figure E.5: Wall deform geometry for Shear stress value at 1e7 N/m²

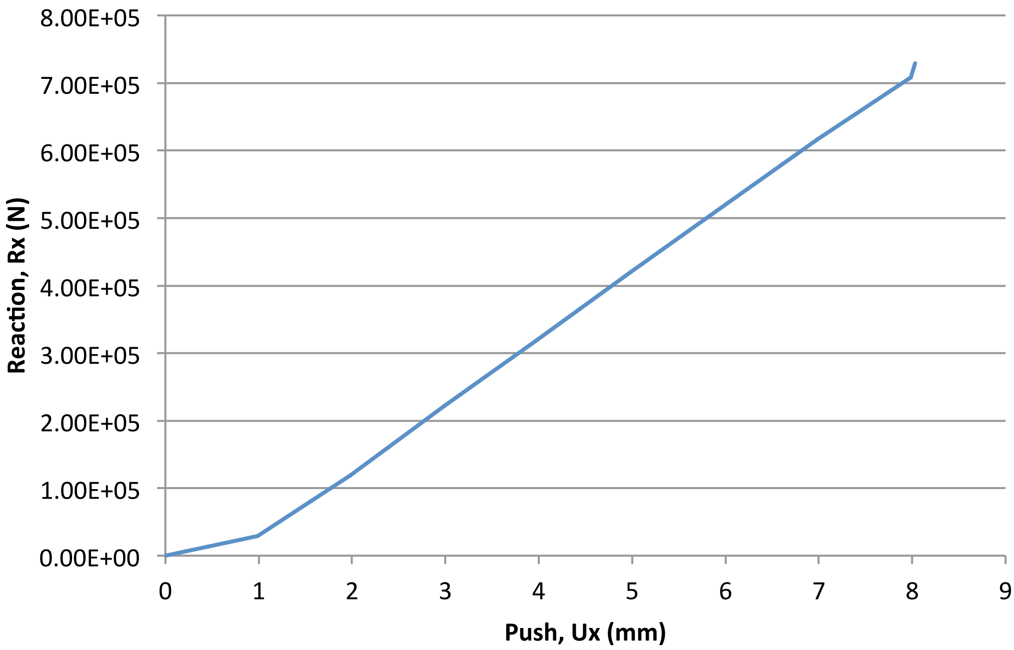

Figure E.6: Lateral reaction vs. base displacement for shear stress value at 1e7 N/m²

Appendix-F

Python Code for Numerical Models of Masonry

##################################DATA_INPUT########################

Interaction Property

- *'Friction'*

fric_form = PENALTY #"formulation"

direction = ISOTROPIC #"directionality"

stress_limit = None #"shear stress limit"

fric_coeff = 0.1 #"table" friction coefficient

frac = 0.005 #"fraction" maximum elastic slip

- *'Normal'*

method = PENALTY #"constraintEnforcementMethod"

separate = ON #"allowSeparation"

- *'Cohesive'*

Knn = 1e10 #"table" stiffness-normal (newton/metre)

Kss = 1e10 #"table" stiffness-shear (newton/metre)

Ktt = 1e10 #"table" stiffness-tangential (newton/metre)

coh_eligibility = INITIAL_NODES #"eligibility" slave-nodes initially in contact

- *'Damage'*

crit = QUAD_TRACTION #"criterion"

init_normal = 1e6 #"iniTable" damage initiating stress(newton/sq.metre)

init_shear = 1e6 #"iniTable" damage initiating stress(newton/sq.metre)

init_tangential = 1e6 #"iniTable" damage initiating stress(newton/sq.metre)

evol_value = 0.0002 #"evolTable" (metre)

stab_viscosity = 0.002 #viscosityCoeff" stabilization vicosity coeff

- *Interaction Choice*

inter_brick =
['Cohesive+Friction+75%Damage','Cohesive+Friction+Damage','Cohesive+Friction+125%Damage']

sliding_val_1 = SMALL #"sliding"

wall_to_support = 'Normal'

sliding_val_2 = FINITE #"sliding"

########################MODELLING_CODE#######################

Interaction Property

- *'Normal'*

mdb.models['Model-1'].ContactProperty('Normal')

mdb.models['Model-1'].interactionProperties['Normal'].NormalBehavior(pressureOverclosure=HARD, constraintEnforcementMethod=method, allowSeparation=separate, stiffnessBehavior=LINEAR, contactStiffness=DEFAULT, contactStiffnessScaleFactor=1.0, clearanceAtZeroContactPressure=0.0)

- *'Cohesive+Friction+Damage'*

mdb.models['Model-1'].ContactProperty('Cohesive+Friction+Damage') mdb.models['Model-1'].interactionProperties['Cohesive+Friction+Damage'].CohesiveBehavior(eligibility=coh_eligibility, defaultPenalties=OFF, table=((Knn, Kss, Ktt),))

mdb.models['Model-1'].interactionProperties['Cohesive+Friction+Damage'].Damage(criterion=QUAD_TRACTION, initTable=((init_normal, init_shear, init_tangential),), useEvolution=ON, evolTable=((evol_value,),), useStabilization=ON, viscosityCoef=stab_viscosity)

mdb.models['Model-1'].interactionProperties['Cohesive+Friction+Damage'].TangentialBehavior(dependencies=0, formulation=fric_form, directionality=direction, pressureDependency=OFF, slipRateDependency=OFF, temperatureDependency=OFF, table=((fric_coeff,),), shearStressLimit=stress_limit, maximumElasticSlip=FRACTION, fraction=0.005, elasticSlipStiffness=None)

- *'Cohesive+Friction+75%Damage'*

mdb.models['Model-1'].ContactProperty('Cohesive+Friction+75%Damage')

mdb.models['Model-1'].interactionProperties['Cohesive+Friction+75%Damage'].CohesiveBehavior(eligibility=coh_eligibility, defaultPenalties=OFF, table=((Knn, Kss, Ktt),))

mdb.models['Model-1'].interactionProperties['Cohesive+Friction+75%Damage'].Damage(criterion=QUAD_TRACTION, initTable=((.75*init_normal, 0.75*init_shear, 0.75*init_tangential),), useEvolution=ON, evolTable=((0.75*evol_value,),), useStabilization=ON, viscosityCoef=stab_viscosity)

mdb.models['Model-1'].interactionProperties['Cohesive+Friction+75%Damage'].TangentialBehavior(dependencies=0, formulation=fric_form, directionality=direction, pressureDependency=OFF, slipRateDependency=OFF, temperatureDependency=OFF, table=((fric_coeff,),), shearStressLimit=stress_limit, maximumElasticSlip=FRACTION, fraction=0.005, elasticSlipStiffness=None)

- *'Cohesive+Friction+125%Damage'*

mdb.models['Model-1'].ContactProperty('Cohesive+Friction+125%Damage')

mdb.models['Model-1'].interactionProperties['Cohesive+Friction+125%Damage'].CohesiveBehavior(eligibility=coh_eligibility, defaultPenalties=OFF, table=((Knn, Kss, Ktt),))

mdb.models['Model-1'].interactionProperties['Cohesive+Friction+125%Damage'].Damage(criterion=QUAD_TRACTION, initTable=((1.25*init_normal, 1.25*init_shear, 1.25*init_tangential),), useEvolution=ON, evolTable=((1.25*evol_value,),), useStabilization=ON, viscosityCoef=stab_viscosity)

mdb.models['Model-1'].interactionProperties['Cohesive+Friction+125%Damage'].TangentialBehavior(dependencies=0, formulation=fric_form, directionality=direction, pressureDependency=OFF, slipRateDependency=OFF, temperatureDependency=OFF, table=((fric_coeff,),), shearStressLimit=stress_limit, maximumElasticSlip=FRACTION, fraction=0.005, elasticSlipStiffness=None)

Interactions Assignment

- *Interaction between Base and Wall*

for i in range(1,x+1):

mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, createStepName='Initial', datumAxis=None, initialClearance=OMIT, interactionProperty=wall_to_support, master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Base'].edges.getSequenceFromMask(mask=('[44]',),)), name='Int-Base'+str(i), slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-'+str(i)+'-

1'].edges.getSequenceFromMask(mask=('[113]',),)), sliding=sliding_val_2, thickness=ON)

- *Interaction between Right restraint and Wall*

```
if y%2 == 0:
    mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, createStepName='Initial', datumAxis=None, initialClearance=OMIT, interactionProperty=wall_to_support, master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Right-Support'].edges.getSequenceFromMask(mask=('[24 ]', ), )), name='Int-Right-Support', slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Half-Brick-lin-2-'+str(y/2)].edges.getSequenceFromMask(mask=('[137 ]', ), )), sliding=sliding_val_2, thickness=ON)
else:
    mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, createStepName='Initial', datumAxis=None, initialClearance=OMIT, interactionProperty=wall_to_support, master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Right-Support'].edges.getSequenceFromMask(mask=('[24 ]', ), )), name='Int-Right-Support', slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-'+str(x)+'-'+str(y)].edges.getSequenceFromMask(mask=('[138 ]', ), )), sliding=sliding_val_2, thickness=ON)
```

- *Interaction between Top restraint and Wall*

```
if y%2 == 0:
    mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, createStepName='Initial', datumAxis=None, initialClearance=OMIT, interactionProperty=wall_to_support,
```

master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Top-Support'].edges.getSequenceFromMask(mask=('[44]',),)), name='Int-Top-Support', slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Half-Brick-lin-2-'+str(y/2)].edges.getSequenceFromMask(mask=('[113]',),)), sliding=sliding_val_2, thickness=ON)

else:

mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, createStepName='Initial', datumAxis=None, initialClearance=OMIT, interactionProperty=wall_to_support, master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Top-Support'].edges.getSequenceFromMask(mask=('[44]',),)), name='Int-Top-Support', slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-'+str(x)+'-'+str(y)].edges.getSequenceFromMask(mask=('[24]',),)), sliding=sliding_val_2, thickness=ON)

- *Interaction between Push brick and Wall*

mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, createStepName='Initial', datumAxis=None, initialClearance=OMIT, interactionProperty=wall_to_support, master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Push-Brick'].edges.getSequenceFromMask(mask=('[44]',),)), name='Int-Push-Brick', slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-1-1'].edges.getSequenceFromMask(mask=('[137]',),)), sliding=sliding_val_2, thickness=ON)

- *Interaction between Base and Push brick*

mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, createStepName='Initial', datumAxis=None, initialClearance=OMIT, interactionProperty=wall_to_support, master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Base'].edges.getSequenceFromMask(mask=('[44]',),)), name='Int-Base-PushBrick', slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Push-Brick'].edges.getSequenceFromMask(mask=('[44]',),)), sliding=sliding_val_2, thickness=ON)

- *Interaction between Bricks in Wall*

```
for j in range(1,y+1,2):
    for i in range(1,x):
        mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, contactControls='Control', createStepName='Step-1', datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick), master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-'+str(i)+'-'+str(j)].edges.getSequenceFromMask(mask=('[138 ]', ), )), name='Int-'+str(i)+'-'+str(j), slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-'+str(i+1)+'-'+str(j)].edges.getSequenceFromMask(mask=('[137 ]', ), )), sliding=sliding_val_1, thickness=ON)
for j in range(2,y+1,2):
    for i in range(2,x):
        mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, contactControls='Control', createStepName='Step-1', datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick), master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-'+str(i)+'-'+str(j)].edges.getSequenceFromMask(mask=('[138 ]', ), )), name='Int-'+str(i)+'-'+str(j), slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-'+str(i+1)+'-'+str(j)].edges.getSequenceFromMask(mask=('[137 ]', ), )), sliding=sliding_val_1, thickness=ON)
for j in range(2,y+1,2):
```

```
    mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE,
clearanceRegion=None, contactControls='Control', createStepName='Step-1',
datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick),
master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Half-
Brick-lin-1-'+str(j/2)].edges.getSequenceFromMask(mask=('[137 ]', ), )), name='Int-
1-'+str(j), slave=Region(side1Edges=mdb.models['Model-
1'].rootAssembly.instances['Full-Brick-lin-2-
'+str(j)].edges.getSequenceFromMask(mask=('[137 ]', ), )), sliding=sliding_val_1,
thickness=ON)

for j in range(2,y+1,2):

    mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE,
clearanceRegion=None, contactControls='Control', createStepName='Step-1',
datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick),
master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-
Brick-lin-'+str(x)+'-'+str(j)].edges.getSequenceFromMask(mask=('[138 ]', ), )),
name='Int-'+str(x)+'-'+str(j), slave=Region(side1Edges=mdb.models['Model-
1'].rootAssembly.instances['Half-Brick-lin-2-
'+str(j/2)].edges.getSequenceFromMask(mask=('[138 ]', ), )), sliding=sliding_val_1,
thickness=ON)

for j in range(1,y,2):

    for i in range(1,x):

        mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE,
clearanceRegion=None, contactControls='Control', createStepName='Step-1',
datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick),
master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-
Brick-lin-'+str(i)+'-'+str(j)].edges.getSequenceFromMask(mask=('[24 ]', ), )),
name='Int-H-'+str(i*2)+'-'+str(j), slave=Region(side1Edges=mdb.models['Model-
1'].rootAssembly.instances['Full-Brick-lin-'+str(i+1)+'-
'+str(j+1)].edges.getSequenceFromMask(mask=('[111 ]', ), )), sliding=sliding_val_1,
thickness=ON)
```

```
for j in range(1,y,2):

    for i in range(2,x+1):

        mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE,
clearanceRegion=None, contactControls='Control', createStepName='Step-1',
datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick),
master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-
Brick-lin-'+str(i)+'-'+str(j)].edges.getSequenceFromMask(mask=('[139 ]', ), )),
name='Int-H-'+str((i*2)-1)+'-'+str(j), slave=Region(side1Edges=mdb.models['Model-
1'].rootAssembly.instances['Full-Brick-lin-'+str(i)+'-
'+str(j+1)].edges.getSequenceFromMask(mask=('[44 ]', ), )), sliding=sliding_val_1,
thickness=ON)

for j in range(2,y,2):

    for i in range(2,x+1):

        mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE,
clearanceRegion=None, contactControls='Control', createStepName='Step-1',
datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick),
master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-
Brick-lin-'+str(i)+'-'+str(j)].edges.getSequenceFromMask(mask=('[139 ]', ), )),
name='Int-H-'+str((i*2)-2)+'-'+str(j), slave=Region(side1Edges=mdb.models['Model-
1'].rootAssembly.instances['Full-Brick-lin-'+str(i-1)+'-
'+str(j+1)].edges.getSequenceFromMask(mask=('[44 ]', ), )), sliding=sliding_val_1,
thickness=ON)

for j in range(2,y,2):

    for i in range(2,x+1):

        mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE,
clearanceRegion=None, contactControls='Control', createStepName='Step-1',
datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick),
master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-
```

```
Brick-lin-'+str(i)+'-'+str(j)].edges.getSequenceFromMask(mask=('[24 ]', ), )),
name='Int-H-'+str((i*2)-1)+'-'+str(j), slave=Region(side1Edges=mdb.models['Model-
1'].rootAssembly.instances['Full-Brick-lin-'+str(i)+'-
'+str(j+1)].edges.getSequenceFromMask(mask=('[111 ]', ), )), sliding=sliding_val_1,
thickness=ON)

for j in range(1,y):

    if j%2==0:

        mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE,
clearanceRegion=None, contactControls='Control', createStepName='Step-1',
datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick),
master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Half-
Brick-lin-1-'+str(j/2)].edges.getSequenceFromMask(mask=('[113 ]', ), )), name='Int-
H-1-'+str(j), slave=Region(side1Edges=mdb.models['Model-
1'].rootAssembly.instances['Full-Brick-lin-1-
'+str(j+1)].edges.getSequenceFromMask(mask=('[111 ]', ), )), sliding=sliding_val_1,
thickness=ON)

else:

        mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE,
clearanceRegion=None, contactControls='Control', createStepName='Step-1',
datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick),
master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-
Brick-lin-1-'+str(j)].edges.getSequenceFromMask(mask=('[139 ]', ), )), name='Int-H-
1-'+str(j), slave=Region(side1Edges=mdb.models['Model-
1'].rootAssembly.instances['Half-Brick-lin-1-
'+str((j+1)/2)].edges.getSequenceFromMask(mask=('[63 ]', ), )),
sliding=sliding_val_1, thickness=ON)

for j in range(1,y):

    if j%2==0:
```

```
mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, contactControls='Control', createStepName='Step-1', datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick), master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Half-Brick-lin-2-'+str(j/2)].edges.getSequenceFromMask(mask=('[113 ]', ), )), name='Int-H-'+str(x*2)+'-'+str(j), slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-'+str(x)+'-'+str(j+1)].edges.getSequenceFromMask(mask=('[44 ]', ), )), sliding=sliding_val_1, thickness=ON)

else:

    mdb.models['Model-1'].SurfaceToSurfaceContactStd(adjustMethod=NONE, clearanceRegion=None, contactControls='Control', createStepName='Step-1', datumAxis=None, initialClearance=OMIT, interactionProperty=choice(inter_brick), master=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Full-Brick-lin-'+str(x)+'-'+str(j)].edges.getSequenceFromMask(mask=('[24 ]', ), )), name='Int-H-'+str(x*2)+'-'+str(j), slave=Region(side1Edges=mdb.models['Model-1'].rootAssembly.instances['Half-Brick-lin-2-'+str((j+1)/2)].edges.getSequenceFromMask(mask=('[63 ]', ), )), sliding=sliding_val_1, thickness=ON)
```