Role of Timber Structural Elements in the Seismic Response of Masonry Structures in the Himalayan Region

By

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Abstract

A large share of the buildings today is still masonry buildings, both reinforced and unreinforced. Post-earthquake reconnaissance studies have shown a remarkable difference between the performance of reinforced and unreinforced masonry buildings. While the unreinforced masonry buildings tend to suffer severe damage, even collapse, resulting in loss of innumerable and invaluable life; reinforced masonry buildings have often been reported to perform remarkably well under seismic events. Sometimes, this performance has exceeded performance of engineered buildings as well.

Himalayas are a highly seismic region in South Asia with multiple major earthquakes recorded across the past two centuries. The remoteness of the region and abundant availability of local materials along with frequent earthquakes has resulted in the development of a seismic culture of earthquake-resistant, timber-reinforced masonry buildings. Though these buildings have shown superior performance under seismic actions, little scientific research has been done to understand and analyse the reason behind this superior performance.

Across the different regions of Himalayas, timber has been used in different structural configurations to increase the seismic resistance of the masonry structures. These traditional building systems remain popular in the Himalayan region for their cheap and easy availability locally. This additional graduation project is a step towards understanding the behaviour of these masonry structures, and the role of timber in preventing catastrophic failure in the former.

In this study, different building typologies in the Himalayas that use timber as a structural element are identified and described. Failure mechanisms of masonry structures are widely studied and a brief overview is presented. In-plane, out-of-plane, combined in-plane and out-of-plane and local failure mechanisms of unreinforced masonry are discussed in detail. Furthermore, a literature review of post-earthquake reconnaissance surveys is conducted to understand different mechanisms through which masonry structures fail.

A review of state-of-the-art on experimental, analytical and numerical studies conducted on resistance of some of the building typologies of Himalayan region (for example, Bhatar, Dhajji Dewari, Ikra, Kath Kuni) is done in this study to understand work done previously. Finally, an analytical analysis is conducted on single room, one-storeyed Bhatar building to investigate the response of an in-plane wall to a lateral load exerted by earthquake excitation.

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1. Introduction

1.1. Background

Masonry is one of the oldest forms of building techniques used by humans. Earliest evidence of brick masonry is reported to be at least 7500 years ago. Even with the prolific adoption of reinforced concrete and structural and reinforcing steel in 20th and 21st century, stone and brick masonry remain popular mainly due to the local availability in remote regions where transporting industrial materials is difficult. A majority of buildings in the world are still masonry buildings. Many seismically active regions in the world are still remote and majority of the buildings in those regions are still masonry buildings.

Himalayan region in the Indian subcontinent is one such highly seismically active region (Singh et al., 2015). The youngest (and highest) mountain range is still growing (Copley et al., 2010). This high seismicity is a result of frequent and continuing collision and convergence of the Indian tectonic plate with the Eurasian tectonic plate (Khattri, 1987). As a result, the Himalayan region has seen major earthquakes in the 20th and 21st century (see Table 1.1) resulting in substantial loss to human life. Along with providing water to majority of river basins in Northern India, Nepal, Bhutan and Pakistan, with water, Himalayas also act as a natural barrier for the Indian subcontinent. The Himalayas can be divided into 4 major regions as shown in Figure 1.1.

a) Karakoram range - Gilgit-Baltistan, Kashmir, Ladakh
b) Western Himalayas – Jammu & Kashmir, Ladakh, Himachal Pradesh, Uttarakhand
c) Central Himalayas – Nepal
d) Eastern Himalayas – Sikkim, Bhutan, Arunachal Pradesh, Assam

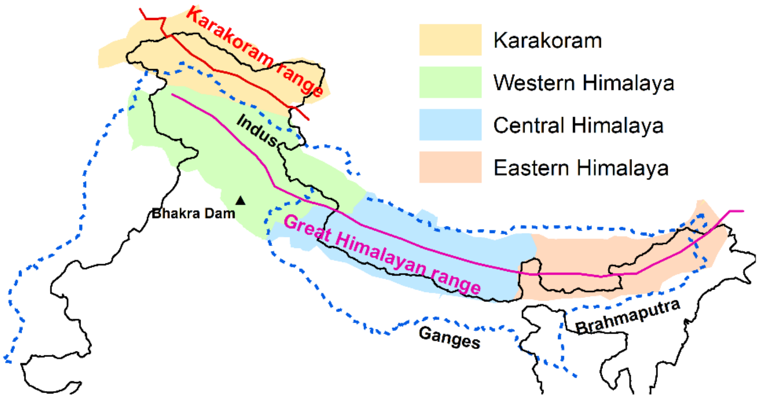


Figure 1.1: Various regions of Himalayas. Map by Kulkarni et al. (2018)

Table 1.1: List of major earthquakes in Himalayan region in the past 150 years

Year	Mw	Location
1897, June 12	8.7	Shillong Plateau, Assam
1905, April 04	8.6	Kangra Valley (Himachal Pradesh)
1918, July 08	7.6	Assam
1934, January 15	8.4	Nepal-Bihar
1950, August 15	8.7	Assam-Tibet
1980, July 29	6.6	Nepal-Uttarakhand
1975, January 19	6.8	Kinnaur (Himachal Pradesh)
1988, August 21	6.6	Bihar-Nepal
1991, October 20	6.8	Uttarakashi (Uttarakhand)

1999, March 29	6.6	Chamoli (Uttarakhand)
2005, October 08	7.6	Muzaffarabad (Kashmir)
2011, September 18	6.9	Sikkim
2015, April 25	7.8	Gorkha (Nepal)
2015, May 12	7.3	Nepal
2015, October 26	7.5	Pakistan, Afghanistan

1.2. Problem Statement

Masonry buildings comprise majority of the building stock in this larger Himalayan region. For instance, masonry buildings form more than 70% of the Nepal's (Gautam & Chaulagain, 2016; Gautam et al., 2018; Parajuli & Kiyono, 2015) and Himachal Pradesh's (*Vulnerability Atlas of India* (2019) as cited in Sharma et al. (2022)) building stocks. A majority of these masonry buildings are unreinforced. Unreinforced masonry (URM) structures have suffered considerable damage in the past earthquakes in the Himalayan region (Ismail & Khattak, 2016; Sharma et al., 2022). The collapse of URM structures has also lead to considerable loss of lives during the past earthquakes (Ali et al., 2013; Javed et al., 2006). Unreinforced masonry buildings suffer failure by development of different types of failure mechanisms. Most of these failure mechanisms can be categorized into in-plane and out-of-plane failures. Akin to other high seismic activity regions like Greece, Italy, Turkey and Portugal (Bostenaru Dan, 2014; Karababa & Guthrie, 2007), the Himalayan region also developed local seismic culture due to the high frequency of earthquakes (Langenbach, 1989). Due to development of seismic culture, particular regions have developed methods of reinforcing the masonry. In current era, with the advent of cement concrete, RC elements are used to reinforce masonry. However, traditionally, timber has been used for many millennia to reinforce masonry, which resists compression, to help the structural system resist tension too (Vasconcelos et al., 2015). Such structures are commonly known as timber-reinforced masonry structures.

Post-earthquake documentation and studies after major earthquakes in the past century have shown timber framed masonry structures to perform better than conventional forms of construction systems throughout different seismic regions of the world (Langenbach, 2007). The same holds true for the Himalayan region. Reconnaissance surveys have shown that timber framed or timber laced building have performed better than other construction techniques after earthquakes in Anantnag (1967) (Gosain & Arya, 1967), Assam (Jain, 2016; Kaushik & Dasgupta, 2013), Kashmir (2005) (Rai & Murty, 2005), Uttarakhand/Himachal Pradesh (Rautela & Joshi, 2009) and the Gorkha, Nepal (2015) (Varum et al., 2018).

Yet, scientific knowledge on the behaviour of these traditionally evolved earthquake resistant systems is scarce. It may be in the form of visual assessments (post-earthquake reconnaissance studies), or vulnerability assessments (based on numerical modelling approaches) or laboratory-based experiments. Improved understanding into how timber imparts greater resilience to the structure in the event of an earthquake will not only inform better maintenance, conservation and preservation of such structures (earthquake-prone habitat structures, heritage structures, etc) but as we approach the age of lowering carbon footprints of buildings, it will also open the avenue of making safer buildings using materials with low carbon footprint.

1.3. Research objective, scope and outline

The main objective of this research is to understand the role of timber structural elements in the seismic response of masonry buildings in the Himalayan region. Specifically, the objectives can be broken down into:

1. To examine different ways timber is used in the different types of masonry buildings in the Himalayan region and their salient features.

2. To understand the different ways an unreinforced or reinforced masonry building can fail under earthquake excitation.

3. To explore various failure mechanisms through which masonry buildings have failed in recent earthquakes in the Himalayan region.

4. To conduct a preliminary, analytical study into the role of horizontal timber bands in increasing the seismic resistance of a stone masonry building through a Bhatar case study building.

This can be formulated into main research question:

How can the role of timber structural elements in the seismic response of masonry buildings in the Himalayan region be assessed ?

In order to answer the main research question, following sub questions are formulated:

SRQ 1. What are the main building typologies present in the Himalayan region which use timber as a structural element?
SRQ 2. What are the different ways in which masonry buildings fail in the event of an earthquake?
SRQ 3. How can the failure mechanisms be categorized and which failure mechanisms can be averted by use of timber?
SRQ 4. Which of these failure mechanisms are commonly seen in collapsed or damaged masonry buildings after earthquakes in Himalayan region?
SRQ 5. How does timber affect the seismic response of a masonry building?
SRQ 6. How can the role of timber be studied – analytically, experimentally or numerically?
SRQ 7. How can the role of timber be quantified?

1.4. Methodology

The assessment consists of a literature review of :

a) Different typologies of buildings present in the Himalayan region.

b) Different failure modes of unreinforced masonry buildings.

c) Post-earthquake studies, reconnaissance surveys and visual assessment reports on the failure of masonry structures in Himalayan region.

d) Analytical and Experimental studies conducted on the behaviour of selected building typologies under lateral loads.

Additionally, an analytical consideration was done to assess the change in seismic resistance of a stone masonry building when horizontal timber bands are introduced to the same building. For this case study, a Bhatar building is chosen as it is one of the most commonly found building in both rural and urban areas of the Himalayas.

To achieve the aforementioned objectives of this study, the report is divided into chapters, synopsis of which is presented below:

- Chapter 2 explores and reports the selected building typologies commonly seen in the Himalayan region. Since the focus of this study is role of timber structural elements; only those typologies are presented that use timber as a structural element.

- Chapter 3 presents a summary of various failure modes commonly observed in masonry buildings, both reinforced and unreinforced, after earthquakes majorly, but not limited to the Himalayan region.

- Chapter 4 presents a comprehensive review of failure mechanisms observed in post-earthquake reconnaissance studies conducted after recent earthquakes in the Himalayan region.

- Chapter 5 briefly summarizes distinct characteristic of timber-framed or timber-laced masonry and their possible behaviour mechanisms under influence of lateral loads.

- Chapter 6 provides a literature review of the state-of-the-art of various analytical, experimental and numerical modelling studies conducted, to analyse the contribution of timber towards the whole structure's resistance to lateral loads, for some of the typologies mentioned above that use timber as a structural element.

- Chapter 7 presents an analytical consideration for the contribution of timber elements in the seismic response of a masonry structure in the Himalayan region taken as a case study.

- Chapter 8 concludes the study by summarising its main outcomes and providing recommendations for and possibilities into possible future research.

2. Building Typologies

Different regions of Himalayas have different types of structures that have developed over many centuries. Building typologies of any region are influenced primarily by the material availability in that region. Himalayas is no different in this aspect. This is evident from the Assam-type house made of bamboo in the Eastern Himalayan region with abundant bamboo availability; to the brick-dominant architecture of Central Himalayan, low-altitude Kathmandu valley due to the rich alluvial soil deposits; to the countless stone masonry villages in high-altitude Western Himalayas due to rocky soil. Lately, with proliferation of steel and cement, reinforced concrete buildings have gained popularity and trust of the local residents of this region. However, most of the RC buildings are not engineered and are designed by local masons or small contractors.

The scope of this literature review has been kept limited to the typologies that exhibit a structural use of wood. Therefore, buildings made out of rammed earth, brick/stone masonry without timber use, reinforced concrete are not studied. Evidently, due to Himalayan region being covered with both deciduous and evergreen forests, timber is used copiously in construction of buildings. Wood finds range of applications in buildings in Himalayan region ranging from structural (beams, columns, joists, planks) to non-structural (doors, windows).

2.1. Taq (Bhatar)

This vernacular construction technique is heavily used in the Gilgit-Baltistan (Pakistan) and Kashmir (India, Pakistan). A form of timber-laced, it is based on dry stone masonry with horizontal timber bands/beams at sill, lintel and roof/floor level to increase confinement. The compressive resistance is provided by the random rubble stone masonry whereas the timber beams provide tensile and bending resistance. The usage of timber also allows for better connected corner joints providing resistance to out-of-plane wall movements. This helps the Bhatar technique to resist wall cracking (Carabbio et al., 2018).

Foundation: Foundation consists of shallow strip foundation made out of stone.
Walls: 8cm-10cm horizontal wooden beams are connected with cross pieces to make ladder-type (see Figure 2.4) timber frames which are present at ground, sill, lintel and roof/floor level. The masonry piers and walls between these timber bands are either stone or brick laid in mud mortar (see Figure 2.4 and Figure 2.1). Certain regions also have a practice of dry stone masonry (Figure 2.2 and Figure 2.3).
Floor/roof: At roof level, timber floor joists span opposite walls and rest on timber roof beams (bands) at roof level. The joists are topped by planks, twigs and compacted earth making the roof particularly heavy.


Figure 2.1: Taq building with horizontal timber bands and brick masonry (Jain, 2016)

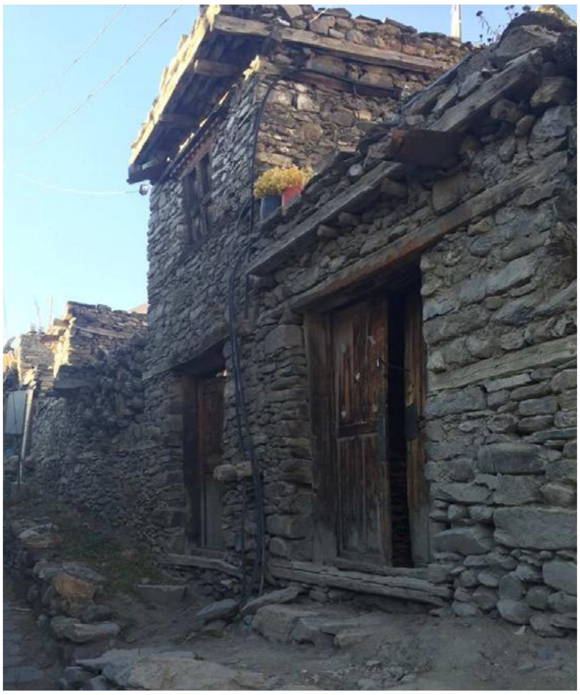


Figure 2.2: Typical Bhatar wall with dressed stone masonry (photo credit: Tom Whitty)


Figure 2.3: Taq wall with dry stone masonry (photo credit: Tom Schacher)

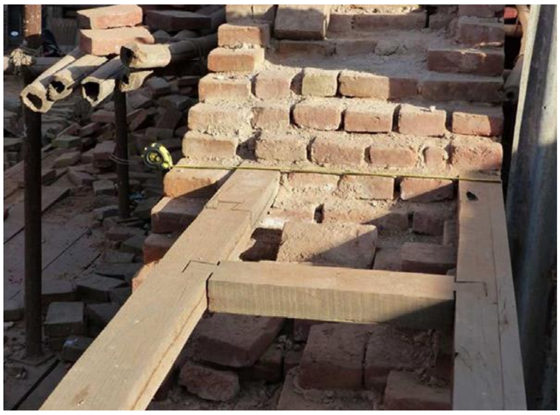


Figure 2.4: Timber ladder (band) with brick masonry (photo credit: Martijn Schildkamp)

2.2. Dhajji Dewari

Dhajji Dewari is a form of construction technique widely used in India and Pakistan administered Northern Himalayan state of Kashmir. In Persian, *Dhajji* means “patchwork quilt” and Dewari “wall”. It might indicate to it’s appearance which resembles the Persian patchwork quilt which is done using waste cloth strips. This technique is similar to “half-timbered” construction technique in ____, while also being referred as “brick-nogged timber-frame construction” in Indian building codes.

Foundation: Similar to Taq/Bhatar, the shallow foundations are made from stone masonry.
Walls: The vertical and lateral load-carrying system is a timber-frame with masonry infill. Masonry infill material varies from stone masonry to brick masonry depending on the availability of the material in the region. The mortar is traditionally mud mortar, however there is a recent prevalence of cement mortar as well. The masonry patches between the horizontal timber bands are further divided into smaller masonry patches by vertical and diagonal timber bracing elements (Figure 2.5). The finished wall may or may not be plastered by mud mortar.
Floor/roof: Traditionally, clay layer is spread over the wooden planks for the floor, that in turn rest over wooden beams spanning between load-bearing walls (Hicyilmaz & Stephenson, 2011).

Connections: Mortise and tenon joints and nails both are used to connect the timber elements. In Kashmir, locking peg is also used sparingly for connections. Of late, metal straps is gaining more widespread use for connections due to easy availability of metal and dying knowledge of sophisticated joinery and dearth of skilled craftsmen.

Dhajji-dewari is often used in the upper floors of a building in conjunction with another timber laced system – Taq/Bhatar, on the lower floor as can be seen in Figure 2.6. This is done to take advantage of this system's thinner and thus lightweight walls. There is evidence for the relatively better performance of this construction system compared to other traditional and conventional construction systems, including RC frame structures, after the 1967 and 2005 earthquakes. The pinned connections allow controlled movement leading to larger dissipation capacity for the system (Gani et al., 2021). While modern Dhajji-Dewari constructions in cities have uniform timber cross-sections and symmetrical bracings (Figure 2.7), in villages, often non-uniform bracings with random lengths can be seen to utilise available lengths of wood (Figure 2.8)

Figure 2.5: Three storeyed Dhajji Dewari house
(Jain, 2016)

Figure 2.6: Bhatar and Dhajji Dewari used in same structure
(Jain, 2016)

Figure 2.7: A modern Dhajji Dewari house
(Sharma, 2022)

Figure 2.8: Random patterns in timber bracing, possibly to use available sizes of wood
(Hicyilmaz et al., 2009)

2.3. Thathara house

Found in the Northern state of Himachal Pradesh, this construction style is named after the local term for wooden planks.

Foundation: Shallow rubble stone masonry is done in strip footings layout for foundation.

Walls: Wooden planks are used to construct load-bearing columns, known as "thola", at the corners (Figure 2.9). Typically around 500mm x 500mm, these columns are often infilled with either dry-stone masonry (Figure 2.11) or stone masonry with mud mortar, either after the column has reached roof level or simultaneously while installing the wooden planks for columns. At sill, lintel and roof levels, horizontal timber bands are provisioned occasionally.

Floors/ Roof: The beams and floors are also made using timber elements. Timber joists/beams support wooden planks upon which traditionally stone slates formed the roofing however in current times, corrugated iron sheet roofing has replaced the stone slate roofing.

Connections: The columns and beams have no moment bearing connections. Diagonal timber bracings or horizontal wooden runners after every few courses of stone masonry is also seen as common practice (Rahul et al., 2013). Perpendicular planks are connected vertically by timber dowels (Figure 2.10). These dowels provide restraint to displacement of the planks in plane or out of plane direction.

Figure 2.9: Thola columns in Thathara house(a) Thola in a modern construction. (b) An older thola column with rudimentary, unfinished planks and unsophisticated joinery (Rahul et al., 2013)

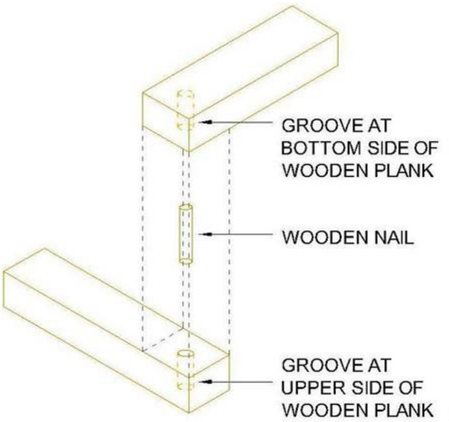


Figure 2.10: Connection of perpendicular wooden planks by wooden dowel (Rahul et al., 2013)

Figure 2.11: Thola with stone masonry infill within (Rahul et al., 2013)

2.4. Kath Kuni/Koti Banal

Literally meaning “wooden corner”, *Kath-kuni* is an indigenous building tradition evolved over the past millennia in the Western Himalayan state of Himachal Pradesh. The same construction technique is known as *Koti Banal* (named after a village), in the neighbouring state of Uttarakhand. This traditional knowledge system evolved through the seismic culture developed over the past 900-1000 years due to high seismicity in Western Himalayas. It is also dependent on the local abundance of Deodar cedar (*Cedrus deodara*) wood and stone.

Foundation: Foundation usually consists of dry stone masonry in foundation trenches dug up to 600mm to 900mm beneath the ground level. An additional raised platform of dry-stacked stones is constructed as a base platform for the structure.

Walls: Kath-Kuni walls consist of double horizontal timber cross-sections connected to the corresponding beams in the transverse direction through timber dowels. The vertical space between the beams is packed with dry stone masonry (van der Zanden, 2018). Thus, the structure has alternate layers of timber beams and dressed stones (Figure 2.12).

Floors: Wooden beams typically span the walls and are clamped on top of the wall, upon which timber planks are nailed. This makes the floors act as flexible diaphragms.

Roof: At the roof level, additional wooden beams span the centre of the walls, dividing the structure into four parts. These beams act as joists for the timber purlins on to slate stone plates are pinned. The purlins span from one gable wall to another.

Connections: While the parallel timber beams at each level are held together in transverse direction by dovetail connections, known as *maanwi*; the vertical connection between timber beams in subsequent layers is done by dowels, locally known as *kadils (van der Zanden, 2018)* as shown in Figure 2.13. The sufficiency of these connectors between different timber members ensure the effective transfer of shear force from the top to the foundation of the building.

Regular and symmetric plans, small openings, shear walls and distribution of wooden beams across the height of the structure are distinct features that increase the resistance of this typology to lateral loads (Rautela et al., 2008) (Rautela & Joshi, 2009).

(a) In Dharali village (Rautela & Joshi, 2008)

(b) Koti Banal house (Rautela & Joshi, 2008)

Figure 2.12: Kath-Kuni/Koti Banal house

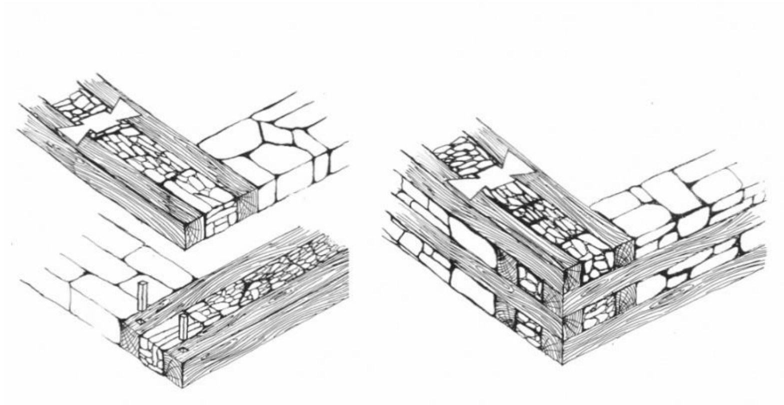


Figure 2.13: Connections showing *maanwi* and *kadil* (Shah & Thakkar, 2018)

2.5. Dry stone

Dry stone masonry walls are generally found in the hilly terrains of Himalayan region as stone is abundantly available in such regions. Uttarakhand, Kashmir, Himachal Pradesh, Sikkim states in India and Nepal are some of the regions where this typology is prevalent in rural areas. The widespread use is mostly linked to the cheap stone as well as stone masons available locally.

Foundation: Shallow rubble dry-stone masonry is laid in foundation.

Wall: Traditional load carrying system is 500mm thick stone walls with dry masonry (Figure 2.15). In recent times, for the purpose of longer spans, additional columns are provided between the masonry walls (Figure 2.16). However, these RC elements are non-engineered and are designed by local masons/contractors using their conventional wisdom and rules of thumb.
Floor: Floor consists of timber joists supporting timber planks (Figure 2.14). Thick layer of mud is laid on top of the planks to finish the floor. Reinforced Concrete is frequently used in recent horizontal or vertical expansions in older buildings.
Roof: The traditional roofing method is timber-framed sloping roofs with either wooden shingles or GI sheets for roofing, without any cross-bracing. A more recent alteration is the 115-150 mm thick in-situ cast RC slabs on the stone walls (Figure 2.15).
Connections: The wooden beams are often merely placed on top of the stone masonry walls without robust connections. For roof, the rafters are nailed onto the roof beam without proper anchorage.

This roof with no bracing or anchorages is sensitive to damage occurring from horizontal loads during earthquakes. The non-engineered RC slabs, though offer marginally better resistance to out-of-plane behaviour of the system as it provides a lateral restraint to the masonry walls, however the very weak dry stone masonry results in high susceptibility to very low in-plane and out-of-plane resistance (Sood et al., 2013). The columns are non-engineered as well, where the local masons and builders depend on their experience, wisdom and rule of thumbs for the cross section size and reinforcement ratios and sizes. The confidence of the local masons on reinforced concrete's vertical load carrying capacity results in very slender cross-sections with insufficient reinforcement and lack of ductile detailing which have been observed to collapse during earthquakes with crumbling of the wall while the slab survives.


Figure 2.14: Dry stone masonry walls with wooden floor (Sood et al., 2013)


Figure 2.15: Dry stone masonry wall with stone slate sloping roof (Sood et al., 2013)


Figure 2.16: RC columns with stone masonry walls and RC slab floor (Sood et al., 2013)


Figure 2.17: Wooden beam at lintel/floor level (Sood et al., 2013)

2.6. Newari house

The traditional Newari house represents the Newa architecture of Newari community in Nepal. Most common in the urban conglomerations of Kathmandu valley, a typical Newari house is 5-7 metres deep and 4-8 metres wide the structure has a 2.20-2.50 meters floor height.

Foundation: Shallow rubble stone masonry is laid in strip footings for foundation.

Wall: The rich alluvial soil of the Kathmandu valley is suitable for burnt bricks and therefore, there's a rich tradition of brick architecture in the region. Multiple variants of brick masonry walls exist. Masonry is generally done with mud mortar. In addition to the usual burnt brick masonry with mud mortar, two major variants are – a) the inside leaf and outside leaf of the wall are apart and the cavity between these two leaves is filled with broken brickbats of burnt brick. This technique of making walls results in delamination of the leaves in the event of an earthquake as the mud mortar separates from the bricks after a few years. b) trapezoidal bricks, locally known as "*dachi aapa*", taper in width on one end giving an appearance of no mortar on the outer surface while having enough clearance for mud mortar med towards the inner surface.

Floor/Roof: Wooden joists are covered by wooden planks similarly to other typologies in the Himalayan region. The planks are then layered with soil to complete the floor. The traditional roof is sloping with country-made terracotta tiles.

The unreinforced masonry walls act as both the vertical load-carrying system and the lateral load-carrying system. The openings have elaborate timber work with double frames – one flush with the external wall face while the other is flush with the internal wall face. The two opening frames are connected by transverse timber elements. The traditional floors are usually timber planks and beams however, more recently, reinforced concrete slabs have been either cast during the vertical extension of buildings or replacing the timber floors altogether. Instances of timber and concrete floor in the same building are also present. For many structures, expansion entails constructing RCC framed structures on top of the original masonry ground or first floors (D'Ayala & Bajracharya, 2003). Newari architecture was initially developed according to sound seismic principles like smaller openings, horizontal timber lacing, symmetrical plans and well connected adjacent buildings with similar roof level as shown in Figure 2.19. However with rapid urbanization and densification of layouts in urban areas in the 20th century, the good construction practices gave way for unsafe ones like large openings, asymmetrical plans and reduced usage of timber as seen in Figure 2.18. The proliferation of concrete has also led to practices such as RC columns being started on top of masonry walls for vertical expansion or timber floors being substituted by RC floors. While the assessment of a building's seismic response is valuable, because of it's presence within a system of houses, the evaluation of this entire system would be more valuable.

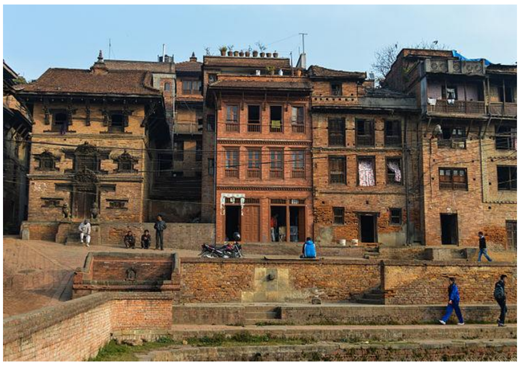


Figure 2.18: Traditional Newari housespicture by Francisco Anzola under Creative Commons Attribution 2.0 Generic

Figure 2.19: 300 year old Newari house in Patan Durbar Square of Kathmanduby Gautam (2018)

2.7. Assam-type house

Assam-type house, also known as Ikra house, evolved as the local seismic culture in the North-eastern Himalayan state of Assam, due to the high incidence of earthquakes in the region (Chand et al., 2017).

Foundation: Unlike most housing typologies existing in the region, in traditional Assam-type house, the main vertical wooden posts are neither inserted into the foundation, nor connected to additional vertical wooden member from the foundation. Instead, they are simply clamped onto the foundation element. Traditionally, there was a lack of a proper foundation. However, recent Ikra houses have a modest concrete stub onto which the main vertical member is secured through steel clamps.

Wall: Horizontal and vertical timber elements form the mainframe of the structure which is then infilled by masonry below the sill level and by woven Ikra (a local river reed) and/or bamboo mesh panels above the sill level as can be seen in Figure 2.21. The panels are later plastered by mud, lime or cement mortar. There are no diagonal timber bracings.

Roof: Timber trusses are covered with corrugated galvanised iron (CGI) roofing sheets to make the roof. However, traditionally, thatch roof or stacks Ikra (river reed) were also used to form the roof (Kaushik & Babu, 2012).

Connections: The timber elements are connected through different types of joints (mortise and tenon, groove and wedge, groove and tooth) using nuts, bolts and nails (Chand et al., 2020a).

Figure 2.20: Typical Ikra house in Gangtok, Sikkim (Alpa Sheth)

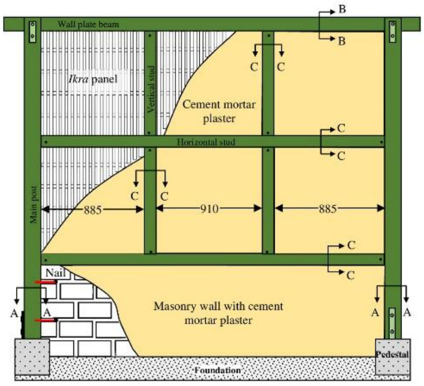


Figure 2.21: Details of a typical Assam type house (Chand et al., 2020a)

Figure 2.22: All Saints church rebuilt after 1897 earthquake in Shillong (Dahunsi, 2008)

2.8. Summary table of all typologies

Building Typology	Regions	Building materials	Foundation	Walls/Framing	Floor	Roof	Connections	Seismic feature
Taq (bhatar)	Kashmir	Stone, Brick, Wood, Soil	Shallow Strip Drystone foundation: Mostly made of stone masonry, manually excavated to a depth of between 0.30 and 0.75 meters, depending on the soil conditions	Timber embedded into masonry walls as horizontal runners placed at the floor level and at the top of windows, tying together all the elements of the building preventing the spreading of cracks in masonry.	A diaphragm that sits on top of the timber joists separating the ceiling and floor levels. Made of timber planks/boards covered with a thick layer of rammed mud.	Flat roof is heavy and similar to intermediate floors whereas sloping roofs are pitched with wooden joists and purlins, covered with either wooden planks (traditionally) or CGI sheets (recent constructions).	Horizontal bands simply placed on top of the load-bearing masonry walls without any connections to the adjacent masonry elements. Connections between the masonry piers are made using small timber elements because the wall width is substantial.	The horizontal wooden bands reduce the height of the stone/brick masonry panels limiting the cracking to smaller areas. The This prevents loss of compressive stress in the masonry wall due to smaller heights. Rocking failure is avoided and in-plane failure becomes dominant.
Dhajji Dewari	Kashmir, Himachal Pradesh, Uttarakhand	Stone, Brick, Wood, Soil	Shallow Strip Drystone foundation: Normally, without any drainage measures around the timber frame base. Recent constructions have solid masonry or lightly reinforced concrete may be used for the shallow foundations. Traditionally, bolting to the foundation was not widely used (lack of positive anchorage), but with the availability of long bolts, more common usage of bolts to provide positive coupling between the timber frame and the strip foundations.	Timber frame with infill walls - wooden frames hold masonry in small sections. Along with vertical elements, presence of cross members dividing the masonry into smaller panels. Frequent use of lean mud mortar. Commonly used in upper story walls, particularly for gable walls.	Wooden columns connected by primary beams and secondary beams span between them. Floor typically made of wooden boards nailed to secondary beams covered with a mud screed for leveling. In areas with scarce availability of timber, scrap pieces of wood used instead. Plywood, often low-quality fiberboard commonly used as ceiling in recent constructions. Although not rigid, floors believed to be stiff enough to distribute lateral loads to the walls.	Wooden trusses covered in corrugated iron, either zinc-galvanized or painted. Trusses usually configured to form a gable or hipped roof. Historically, rough-cut purlins used to span between the roof trusses, on which shingles placed as the weather surface, but more recently metal, asbestos, cement or plastic corrugated sheets commonly used as roofing materials.	Traditional dhajji buildings do not use dowels or pegs to connect wooden elements, with the exception of the Kashmiri joint which uses a locking peg. Use of nails and metal strapping to strengthen connections is becoming more common, but it has poor performance. It is often used as substitute for proper carpentry as it is a fast method and as labor costs are high.	Ring beams provide horizontal bracing to connect the walls and distribute lateral forces evenly preventing out-of-plane failure. Roof trusses aligned with wall posts and hipped roof (with stiffness in both orthogonal directions) used instead of gable wall. Floor beams with sufficient overlap and locking with main beams handle reversible loads. Well connected floor boards form strong floor diaphragm. Proper connection of orthogonal walls and good bracing ensure smaller masonry panels providing the masonry many lines to arch against and preventing crack propagation in the infill.
Thathara Style	Himachal Pradesh, Uttarakhand	Stone, Wood, Soil	Shallow Dry stone masonry: trench with a depth of 1 to 1.5 meters and width of 500mm, depending on the soil type. The trench is then filled with layers of stones without any mortar, reaching a height of 500mm above the ground level. The foundation width is around 500mm. In hilly regions, rock can be found at a relatively shallow depth in certain situations where the foundation depth is terminated.	Wooden planks, also known as tholas, used to create vertical load-bearing members, or columns. Unique combination of timber and stone. Serve as both the vertical frame element and the horizontal frame elements	Timber planks or beams. Thick, rammed soil floors have been replaced with ballast and concrete or plaster finishing. In floors, 20mm thick wooden planks are covered with 25mm thick mud plaster.	Timber planks or beams support natural stone slates, wood plank, plywood, or manufactured wood panels that are placed on joists and supported by beams or walls. Stone slates are commonly used as roof covering and both gable and hipped roof styles are used. Gable roofs with an incline of approximately 17 degrees are used.	Roof beam (or wall plates) placed over *Tholas* and walls without positive connection leading to no moment bearing connection. Rafters secured in place with iron nail connections. Wooden planks or slates connected to the purlins with iron nails. Connection between the wooden supports (*Thatharas*) made by inserting wooden pins into small holes or by using mortise and tenon joints between two planks in alternate course.	In order to protect the walls from outward pressure and to provide small window openings, horizontal wooden members are used. In some instances, additional wooden bracing is also employed. Increasing the thickness of wooden columns, known as 'Tholas', increases the structure's resistance to lateral movement. Older constructions often feature a light wooden plank covering on floors and A-shaped bracing for roofs.
Koti Banal/Kath Kuni	Uttarakhand, Himachal Pradesh, Jammu & Kashmir	Wood, Stones, Slates	Shallow Strip Drystone foundation : In situations where rock is present at the surface, a platform made of dry stone masonry is constructed directly on the ground without any additional foundation.	A hybrid timber-reinforced stone masonry system with interconnected timber logs forming a strong cribbage for reinforcement purposes. The stones mostly assembled without any grout or mortar, allowing for a certain level of flexibility and allowing lateral deflections of the building without causing damage. Outer walls parallel to the floor beams frequently supported by vertical shear keys over several storeys.	Wooden planks laid on joists supported by beams or walls. Floors able to flex like a diaphragm because no cross or inclined planks present. Beams are connected to the walls with pins, providing support for the walls perpendicular to the beams in an out-of-plane motion.	Beams hold up the joists that support the floorboards on each level. For the roof, a wooden frame serves as a flexible diaphragm, and it is covered with large slate tiles.	Pairs of wooden beams are connected by wooden shear pins/tenons, acting like a wooden frame that is reinforced by well-dressed flat stones between the logs, increasing the bearing and lateral capacity of the construction.	Walls that run parallel to the floor beams are supported in an out-of-plane manner by using a large timber log that is longer than the building's dimensions and has holes at both ends. A vertical member, known as a shear key, is inserted into the hole and has a length equal to multiple stories, providing support to the walls in the out-of-plane direction. The use of horizontal wooden logs in the vertical walls is similar to the use of seismic bends (ring beams) in contemporary masonry buildings.
Dry stone construction	Himachal Pradesh, Uttarakhand, Jammu & Kashmir	Stone, Wood, Soil	Shallow Strip Dry Stone masonry: Manually packed stones of various shapes and sizes without any mortar. Depth variesfrom 900mm for loose soil to 200mm for hard strata. Width is typically the same as the width of the wall above, around 500mm. In some cases, dry stone retaining walls are used to stabilize the backfill or create a flat surface to support the building.	Unreinforced stone masonry walls with no use of mortar. No horizontal or vertical timber bands/elements. The stone masonry walls are the primary system for resisting vertical loads. The floors and roof transfer the weight load to 500mm thick walls, made of undressed river stones and without the use of mortar. In some instances of more recent constructions, RC columns are added to support longer span beams.	Traditionally, wooden planks and beams were overlaid with thick layer of rammed soil to make the floor. Of late, most new buildings have 115 to 150mm thick cast in-situ RC slabs, cast manually on site without the use of any compaction equipment. The RC slabs usually rest on the full thickness of the walls, but no extra reinforcement is added along the walls.	Traditionally, gable or hipped sloping roofs, covered with slate stones available locally in abundance. Beams, rafters, and purlins made from durable hardwoods. Recent constructions feature sloping roofs with timber rafters covered with either GI sheeting or wood shingles. Depending on the local availability, stone slates are also used as roof cladding. Flat RC roofs in recent structures expected to improve the seismic behavior of the building due to its in-plane rigidity and good bearing on the walls.	Due to the dry stone construction of the walls, anchorage of the roofs to the walls is not commonly seen. Sloping roofs often lack cross bracing and ties, making them vulnerable to damage during earthquakes.	Longer stones that connect the two layers of the walls are placed at regular intervals to prevent the walls from splitting during shaking.
Newari House	Nepal	Brick, Soil, Wood	Strip Stone masonry: Depth upto 900-1200 mm, stone or brick solid masonry without reinforcement. Width typically 400-500 mm. Use of mud mortar prevelant.	Unreinforced brick masonry with mud mortar and vertical wooden posts. Ordinary sun-dried bricks or "dachi aapa" bricks, with a trapezoidal cross section commonly used. Timber frame consists of columns with a capital on top supporting a double beam. Columns have square cross section and are pinned to the ground, with a distance of 100 to 150 mm between them. Adjacent timber frames are typically connected only at the level of the beam.	Timber planks and beams used traditionally, however in situ reinforced cement concrete solid slabs more popular in recent constructions often replacing or used in conjuction with old timber floors.	Timber beams placed at regular intervals, typically 150 to 200 mm apart, spanning width of the wall, support either wooden planks or bamboo chirpat, covered with a layer of compressed mud.	The floor structures are not stiff and rigid, instead have closely spaced joists that are connected to the walls with wooden pegs. This allows for the even distribution of lateral loads.	Redistribution of vertical loads homogenously by wooden pegs anchoring the wall to floor joists or teh wall plates under the floor joists Both leaves of masonry walls tied together by timber bands along the walls and returns between the perpendicular walls.
Ikra	Assam, Meghalaya, Manipur	Bamboo, Ikra, Wood, Brick, Soil, Sand	Strip foundation with manually excavated depth upto 500-900 mm. Solid brick masonry with mud mortar used traditionally however recent constructions have use of cement mortar. No reinforcement used in foundation.	A timber frame with no diagonal braces filled in with Ikra (local river reed) or bamboo mesh. The frame made up of both horizontal and vertical elements.	Traditionally wooden plank flooring used, while rural areas tend to have mud plaster flooring. Cement flooring on top of a base layer of sand or brick soling popular in recent times.	Pitched Corrugated Galvanized iron (CGI) sheeting over timber trusses is the most widely used roofing system. Traditionally, it was built out of natural materials like grass or leaves.	The vertical wooden posts and pillars are connected to roof truss and horizontal wooden beams at different levels, such as the floor, sill, lintel, and eaves, using nails, steel U-clamps, and steel bolts.	The wooden frames of houses are connected to the lightweight walls and roof through flexible connections providing good resistance to earthquakes. The Ikra wall system and the light roof contribute to the overall lightness of the structure. Because of low mass, good performance is seen during earthquakes.

3. Types of failure mechanisms

This chapter provides a general overview of the commonly occurring failure mechanisms in masonry structure.

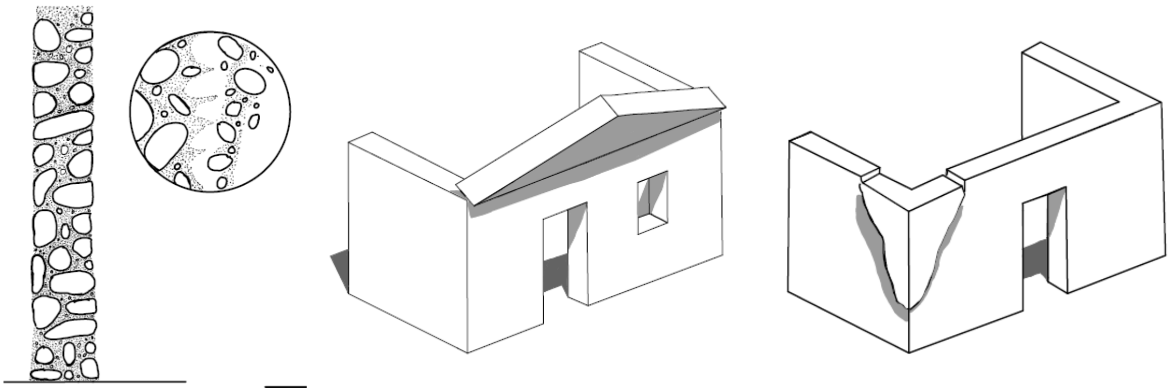
3.1. Local mechanisms

The non-homogeneity and non-monoletheism of masonry leads many local failure mechanisms developing in such structures in the event of an earthquake. The substantial self-weight and insignificant tensile strength of masonry further exaggerates these mechanisms. A selection of local failure mechanisms are discussed below.

Disintegration of masonry: Masonry disintegration can occur due to a multitude of reasons, ranging from crumbling of masonry due to inability of a cross section to resist lateral loads, to bad workmanship (absence of bond stones), to usage of inferior building materials, to separation of the leaves due to lack of cohesion. The lack of cohesion can also be a result of bad construction practices like using weak mortar or round stones (Figure 3.1).

Overturning of gable end walls: Amplified earthquake excitations at the gable height combined with improper connection with the roof makes the gable vulnerable to overturning. This is further aggravated by the fact that the rafters of a sloping roof rest on the orthogonal wall and hence the gable endures no overburden weight that is placed on the transversal wall (Figure 3.1).

Top corner damage: Unlike at floor level, at roof level, there is a lack of horizontal floor members like joists or beams connecting the in-plane and out-of-plane walls which in turn prevents the formation of diaphragm action. On one hand, this, along with the lateral thrust from the roof and openings present near the corner result in rocking and sliding in the in-plane wall. On the other hand, the out-of-plane wall undergoes flexural failure. As a result, both in-plane and out-of-plane walls undergo diagonal cracking resulting in wedge type diagonal cracks (Vlachakis et al., 2020) (Figure 3.1).



(a) (b) (c)

Figure 3.1: Local failure mechanisms

3.2. In-plane failure mechanisms

In-plane failure mechanisms utilize maximum capacity of the wall while dissipating significant energy compared to out-of-plane failure mechanisms. Additionally, in-plane failures are also less brittle in nature, making them the desired failure mode. The in-plane behaviour can be activated by preventing the structure from failing through out-of-plane or local failure mechanisms (Vlachakis et al., 2020).

During seismic activity, the typical behaviour of a wall with openings can be described by dividing the wall into – a) piers, b) spandrels, c) joints. In-plane failure can occur in load-bearing masonry walls through two basic behaviours – flexural behaviour and/or shear behaviour. Both, shear and flexural behaviour has associated failure modes. Main parameters affecting the capacity of masonry walls loaded in-plane are (a) mechanical properties of the material, (b) boundary conditions, (c) extent of vertical load on the wall, (d) slenderness of the wall (Celano et al., 2021).

Flexural behaviour: The two flexural failure modes – rocking and toe crushing, are dependent on the ratio of vertical load relative to the compressive strength of the masonry wall.

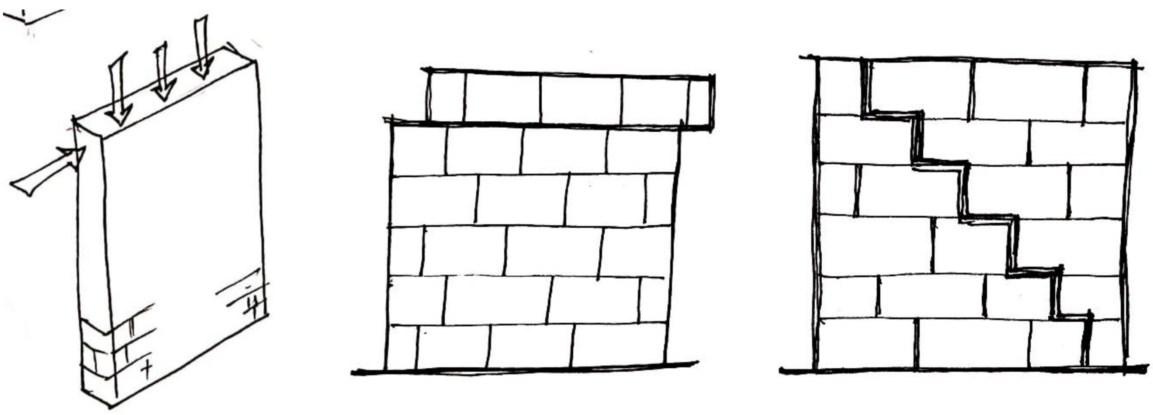
Rocking: In case this ratio is low, the lateral load results in tensile flexural cracking at the corners. The masonry wall in this case starts acting as rigid body and rotates around the compressed corner (Proença et al., 2018). According to Moon et al. (2006), flexural (or rocking) damage appears as tensile cracks at the top or bottom of slender piers with large aspect ratios and low overburden stresses. This occurs due to rigid body rocking of the masonry piers about the compression toe. This rotation as a rigid body leaves the pier with no lateral load-resisting capacity (Bruneau, 1994) (Figure 3.2).

Toe-crushing: Under several cycles, the compression stress in the toe region exceeds its compression strength leading to toe-crushing. The toe-crushing can also occur if the ratio of vertical load to compressive strength is high enough (Figure 3.2).

Shear behaviour: Lateral load produces two primary failure modes – sliding shear failure and diagonal shear failure.

Sliding shear: When the flexural cracking occurs at the corners under tension, the resisting cross section reduces. This reduction results in sliding shear failure (Proença et al., 2018). The cracks in this failure mode could propagate in two ways – (a) over a horizontal bed joint plane also called as bed joint sliding, and (b) stepped diagonal cracks (Figure 3.2).

Diagonal shear: Diagonal shear failure appears in the form of diagonal X-cracks in low aspect, squat masonry piers that are heavily loaded (Naseer et al., 2010). These cracks usually develop when the tensile strength of the masonry is exceeded, starting at the centre of the wall and propagating towards the corners. The diagonal cracking in rubble masonry develops in the form of almost straight cracks (Figure 3.2).



(a) (b) (c)

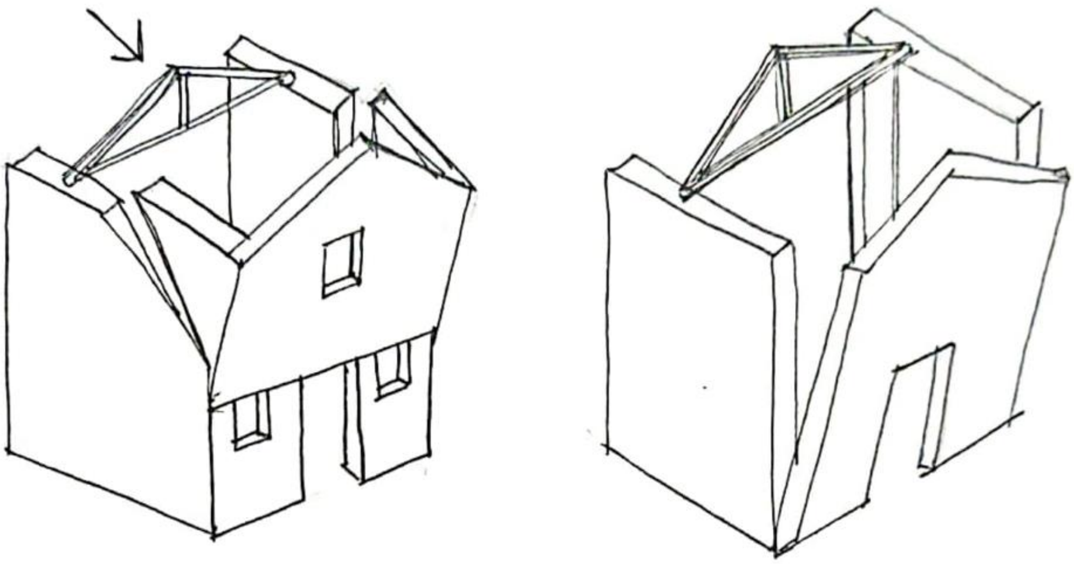
Figure 3.2: (a)-(c) In-plane failure mechanisms

3.3. Out-of-plane failure mechanisms

An **out-of-plane** masonry wall undergoes flexure (bending) when subjected to a horizontal load. Out-of-plane bending is a result of inability of the structure to behave in a "box-like manner". The horizontal diaphragms do not sufficiently connect the structure and the structure is unable to resist the inertial force developed in the walls perpendicular to the seismic action (Vlachakis et al., 2020). Slenderness also affects out-of-plane behaviour as the top part of the wall acts as cantilever leading to toppling over or collapse of the specific part.

The location and extend of supported edges largely influences the internal stresses at the interface of the constituent materials, and in turn, crack patterns and the failure modes (Vaculik, 2012). While long walls without sufficient transversal support, in essence one-way spanning walls, undergo one-way bending; walls with sufficient transversal support suffer two-way (biaxial) bending .

Transverse walls at more frequent distance reduce the unconstrained length of the out-of-plane wall increasing the resistance to lateral loads. However, mere presence of transverse walls is not enough for sufficient resistance to seismic actions. Adequate connections are also required to activate the in-plane wall into carrying the inertia forces of the out-of-plane wall (Vlachakis et al., 2020). Most unreinforced masonry buildings do not have the abovementioned adequate connection. However, timber acts as an excellent connector, in timber-reinforced masonry buildings, for the transversal walls to connect to the in-plane walls.



(a) (b)

Figure 3.3: Out-of-plane failure mechanisms

4. Post-earthquake reconnaissance studies: Failure mechanisms observed in unreinforced masonry structures in Himalayan region

This chapter presents the specific failure modes occurring in masonry buildings in the Himalayan region.

4.1. In-plane failures

In-plane failure mechanisms can be commonly seen in unreinforced masonry buildings after earthquakes. A common in-plane failure mode is diagonal cracks starting at the corner of openings and propagating towards the corners of the walls as seen in Figure 4.1. Masonry walls also depict diagonal shear failures in the piers between the windows (see Figure 4.2) This type of failure is desirable as it is ductile and it was seen that such failure modes did not usually result in complete collapse of the wall (Naseer et al., 2010). Apart from diagonal shear failure, flexural cracks appear at the top and bottom of the openings as seen in Figure 4.4 as the piers start rotating in a rigid body motion due to cyclic alternating bending stresses. Providing multiple ventilators just beneath the roof has been seen to cause in-plane damage, even collapse of roof as it decreases the wall volume and hence the capacity of the wall to resist shear resistance considerably. This can be seen in Figure 4.3. Spandrel failure (Figure 4.6) in unreinforced masonry is dictated by the geometrical and material properties but most importantly the configuration of constraints on the four sides of the in-plane wall.

(a) (a) (b)

Figure 4.1: In-plane shear damage in walls with openingsin (a) in brick masonry building (Shakya & Kawan, 2016), (b) and (c) Stone masonry buildings (Adhikari & D'Ayala, 2020)

(a) (b)

Figure 4.2: (a)-(b) - Diagonal shear failures of masonry wall piers(Naseer et al., 2010)

(a)

(b)

Figure 4.3: (a)-(b) - Damage of masonry piers adjacent to the roof due to multiple ventilators/openings(Naseer et al., 2010)

(a)

(b)

Figure 4.4: (a)-(b) Flexural failure of masonry wall piers (Naseer et al., 2010)

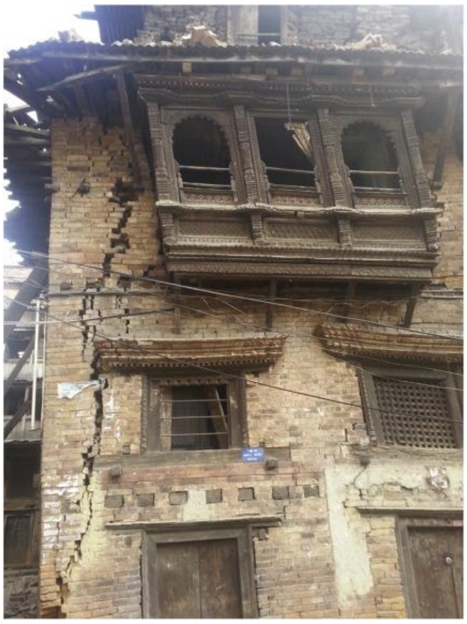


Figure 4.5: Tension failure of building (Shakya & Kawan, 2016)

Figure 4.6: Spandrel failure (Shakya & Kawan, 2016)

An overview of the various in-plane failure mechanisms developed in masonry buildings derived from the literature review of selected post-earthquake field reconnaissance surveys is presented in the Table 1.

Table 4.1: In-plane failure mechanisms

	Failure mode	Types of damage	Causes
1.	**Flexural failure**		
A)	Rocking failure	Flexural cracks at	
		(a) top and bottom of piers	(a) slender geometry, (b) low overburden stresses
		(b) in portions of walls between openings (piers)	High aspect ratio of pier
B)	Toe-crushing failure	Compression of piers and crushing of point of compression (toe)	(a) large overburden stresses in piers with high axial stresses, (b) weak piers, strong spandrels (c) repeated cycles of large drift levels (d) compression strength exceeded, (e) rigid and heavy RC floor and roof
2.	**Shear failure**		
A)	Diagonal shear failure	Diagonal X-cracks in piers and spandrels (some straight cracks in rubble masonry)	(a) heavily loaded masonry wall piers, (b) low aspect ratios, (c) lack of reinforcing components (d) principal tensile stress reaches tensile capacity at the centre of the member (e) strong mortar with comparatively weak bricks
B)	Sliding shear failure		
I)	Horizontal shear sliding (Bed-joint sliding mechanism)	Horizontal crack over mortar bed joints	(a) frictional capacity of the member exceeded (ductile behaviour) (b) high aspect ratio of pier, cracked pier moves as rigid body
Ii)	Diagonal shear sliding (Diagonal step joint sliding mechanism)	X stepwise cracks over the member	
3.	**Other in-plane failure mechanisms**		
A)	Torsional failure due to In-plane shear damage	Severe near-collapse shear failure due to compression of external corner piers	Large mass of rigid RC floor and roof diaphragms on clay brick piers
B)	Cracking around openings	Vertical/diagonal cracks at openings:	Discontinuity in wall with many openings
		(a) corner of openings	(a) Stress concentration at corners of windows and doors (b) Absence of sill and lintel bands
		(b) between two openings one above the other	A) Flexible floor diaphragm b) Absent of sill and lintel band

C)	Tension failure	Vertical cracks at the centre, ends or corners of the walls	A) High and narrow walls b) Openings too close to corners c) Deficient bond at corners (wall to wall connection)
D)	Diagonal tension with joint sliding	Cracks in squat walls	Good quality masonry
F)	Torsion and warping Failure	Excessive cracking due to shear in all walls especially near corners	A) Asymmetry in plan and elevation b) Imbalance in the sizes and positions of openings in the walls
G)	Spandrel failure	Flexural and diagonal cracking, bed joint sliding	(a) spandrel geometries, (b) relative material properties, (c) boundary conditions

4.2. Out-of-plane failure mechanisms

Gable overturning was found to be one of the most common types of failure mechanisms to develop in multiple post-earthquake reconnaissance surveys. No vertical load on top of the gable wall, no ties and braces tying the gable to the roof structure and insufficient connection to the perpendicular in-plane walls are some of the factors contributing to the disproportionate contribution of gable wall overturning to failures. In such a circumstance, the gable essentially behaves akin to a parapet wall as observed by Gautam et al. (2016) in Figure 4.11. Improper connections to the perpendicular walls also lead to separation of the out-of-plane wall as observed by Dizhur et al. (2016) and Adhikari and D'Ayala (2020) in Figure 4.14. The thrust from a poorly connected roof might also lead to partial or severe damage to out-of-plane wall (Figure 4.8). Another reason for the failure of the out-of-plane wall is the relatively longer length because the transversal internal walls are not tied to the out-of-plane wall (Figure 4.9). An example of inadequate connection to the roof is where the timber joists are simply embedded or rested on the top of the wall while not being provided a positive connection as seen in Figure 4.12. Asymmetrical plans and protrusions also lead to failure (see Figure 4.15).

(a) (b)

Figure 4.7: (a) – (b) Out of plane collapse of façade wall (Dizhur et al., 2016)

(a) (b)

Figure 4.8: (a) – (b) Overturning of façade (Dizhur et al., 2016)

Figure 4.9: Complete collapse of façade (Gautam et al., 2016)

Figure 4.10: Out-of-plane collapse of majority of walls (Gautam et al., 2016)

Figure 4.11: Out of plane collapse and overturning of gable masonry (a) (Javed et al., 2006) (b) (Gautam et al., 2016)

Figure 4.12: OOP collapse of wall due to inadequate connection with roof floor and transversal wall (Rai et al., 2015)

Figure 4.13: Collapse of RC slab on top of masonry walls (Gautam & Chaulagain, 2016)

(a) (b) (c)

Figure 4.14: Separation of orthogonal wall (a) – (b) Brick masonry (Dizhur et al., 2016) (c) Stone masonry- (Adhikari & D'Ayala, 2020)

Figure 4.15: Out of plane damage due to asymmetrical plan or protruded part of the building (Dizhur et al., 2016)

An overview of the various out-of-plane failure mechanisms developed in masonry buildings derived from the literature review of selected post-earthquake field reconnaissance surveys is presented in the Table 2.

Table 4.2: Out-of-plane failure mechanisms

Failure mode	Types of damage	Causes
Vertical/partial overturning	Partial or complete overturning of loadbearing walls	Deficient lateral capacity due to: (a) long span façades, (b) flexible floor diaphragms, (c) weak connections between orthogonal walls, (d) lack of diagonal bracing (e) lack of adequate connection between the walls and floor/roof diaphragms
Overturning with side walls	Separation of orthogonal wall at the connection of façade and returning wall	(a) Lack of connection (corner ties) between orthogonal walls (b) tensile strength of the exceeded (c) high shear stresses at wall intersections due to flange action (more susceptible to cracking)
Gable overturning	Out of plane overturning of heavy masonry gable wall	Overturning at even relatively lateral loads due to: (a) no/little vertical loads, (b) inadequate connection between gable and roof, (c) insufficient lateral restraint leads to parapet wall-type behaviour
Vertical/partial overturning	Cracking and/or collapse at top of the OOP wall	Amplified acceleration across the height due to: (a) slenderness, (b) lower overburden weight (c) inadequately connected with the roof
	Damage/overturning of protrusion	(a) building plan irregularity (b) torsion
	Failure of incrementally built walls	Inadequate connection capacity required to distribute the inertia forces of the OOP façade to the in-plane walls (by quoin-stones, timber-laces or tie rods)
	Vertical cracks at corners across the height of the building	Reduced strength of corner connections due to presence of openings near corners
	Damage of corner supporting inclined roof in both directions	(a) lack of proper connection between walls and floor, (b) lateral thrust by the roof, (c) inertial forces due to IP rotation of rigid diaphragm
OOP vibration	Damage or collapse of short piers (and roof)	Reduced length of wall between multiple ventilators adjacent to floor

4.3. Combined In-plane and Out-of-plane effects

Rai et al. (2015) also observed the out-of-plane failure of a wall already weakened by in-plane shear damage depicted by step-type diagonal cracks. The bidirectional effects of earthquake excitation is more prone to occur in structures with high ratio of openings. It was also found that this failure mechanism is more dangerous as it led to out of plane sliding of the wall, and consequently collapse of the structure (Naseer et al., 2010).
Increase in the overall surface area of openings in load-bearing masonry walls also increases its susceptibility to combined effects of in-plane and out-of-plane effects. Already cracked (through in-plane shear) wall is vulnerable to overturning and sliding from out-of-plane actions (Naseer et al., 2010).

Figure 4.16: Combined in-plane and out-of-plane failure observed in Kashmir after 2005 earthquake (Naseer et al., 2010)

Figure 4.17: Combined in-plane and out-of-plane failure observed at Nikosera in unreinforced masonry after 2015 Gorkha earthquake (Rai et al., 2015)

An overview of the various out failure mechanisms developed in masonry buildings due to combined effects of in-plane and out-of-plane actions, derived from the literature review of selected post-earthquake field reconnaissance surveys, is presented in the Table 3.

Table 4.3: Combined In-plane and Out-of-plane effects

Failure mode	Types of damage	Causes
OOP damage after IP shear damage	Cracking or collapse of an already in-plane damaged wall by out-of-plane overturning	High proportion of openings rendering the wall susceptible to bidirectional effects as in-plane shear cracks already present
IP flexure of the spandrel and the OOP response of the façade	Vertical cracks at the end sections of the spandrels	
IP shear damage and the OOP behaviour of the façade	Diagonal cracks at the lower corners of the openings propagating towards the corners of the structure	
Combination of IP rocking-sliding and OOP flexural failure	Wedge type diagonal cracks	(a) presence of a thrusting roof, (b) openings at orthogonal wall close to the corner
Wedge biaxial failure	Bursting collapse of the corner in a rhombus shape	(a) high biaxial stresses in the corner (b) recess corner of a plan (irregular structure)

	Walls loaded in out-of-plane separate from the perpendicular in-plane walls	Absence of adequate connections with perpendicular walls
	Crack starting from the top end at the diaphragm level at intersection of IP and OOP loaded walls	Excessive movement in diaphragm due to: (a) substantial flexibility of diaphragm, (b) lack of proper shear anchorage with IP walls

4.4. Localized damage

Local failures have been observed in the field occurring mainly due to poor construction practices, inferior quality of building materials and insufficient structural details. As seen from the Figure 4.18, the lack of horizontal tie or lace connecting the orthogonal walls adequately results in corner damage. Poor masonry, most commonly absence of pass-through stones in random rubble masonry (Figure 4.19) and poor quality mud mortar (Figure 4.20) lead to delamination of wythes. This effect is exacerbated by out-of-plane movement. Another area of frequently observed local damage is the thrust from the inclined roof that is not connected to the walls appropriately. This lateral thrust results in either partial local damage or complete collapse of the masonry (Figure 4.21).

(a) (b) (c)

Figure 4.18: Corner damage (a) in Muzaffarabad after 2005 Kashmir earthquake (Naseer et al., 2010), (b) in unreinforced stone masonry building after 2015 Hindukush earthquake (Ismail & Khattak, 2016) (c) in stone masonry building after 2011 Sikkim earthquake (Gautam et al., 2021)

(a) (b) (c)

Figure 4.19: Delamination of external leaf of stone masonry wall due to out-of-plane effect (a) after 2015 Gorkha earthquake (Kuffel et al., 2015), (b) after 2005 Kashimr earthquake (Javed et al., 2006), (c) in Solukhumbu after 2015 Gorkha (Nepal) earthquake (Gautam et al., 2016)

v

Figure 4.20: Masonry delamination due to (a)-(c) - absence of through stones. (d) - segregated mud mortar

Figure 4.21: Failure due to lateral thrust from inclined roof (Naseer et al., 2010)
(a) Collapse of masonry wall, (b) Localised damage

An overview of the various out failure mechanisms developed in masonry buildings due to combined effects of in-plane and out-of-plane actions, derived from the literature review of selected post-earthquake field reconnaissance surveys, is presented in the table 4.

Table 4.4: Localised damage

Failure mode	Types of damage	Causes
Damage at corners	Heavy corner damage	(a) stress concentration at the corners of the openings (b) no cornerstones
	Localized damage due to thrust action of the roof	floors supported by transversal walls spanning only in one direction, not connected adequately by the horizontal systems necessary for diaphragm action
	Delamination of wall leaf/wythe	(a) poorly integrated multi-leaf masonry due to absence of pass-through stones, (b) poor construction practices like mud-mortar with high water quantity results in larger voids and poor binding of masonry units, (c) loss of inter-stone friction due to vibration caused by the earthquake excitation
	Collapse of External Veneer of Masonry Walls	Collapse at even low level of seismic excitation due to: (a) weak mortar (b) no through stones across the double-leaf thickness
Pounding damage	(a) localised heavy in-plane damage of loadbearing piers, (b) cracks at the floor level, (c) sway of buildings	Lack of space (seismic separation gaps) in densely populated areas varying heights leading to lateral forces at the contact points

5. Seismic behaviour of timber-masonry structures

5.1. Distinct characteristics of timber masonry structures

Timber-frame masonry structures have superior performance in the event of an earthquake compared to other forms of buildings in the seismic zones due to following reasons:

a) *Weight*: Lighter weight of the structures leads to lower seismic loads.

b) *Framing*: Usage of timber in the framing system enables the structure to resist tensile stresses

c) *Infill materials:* Depending on abundance of a particular material in a region, the infill material varies from unburnt clay brick, fired brick, stone to bamboo, reed etc.

Two distinct types of timber masonry structures are present in the Himalayan region – timber framed masonry structures and timber-laced masonry structures. While Assam-type Ikra houses and Dhajji Dewari are timber-framed structures, Taq (Bhatar) and Kath Kuni (Koti-Banal) are timber-laced structures. Post-earthquake reconnaissance surveys after multiple earthquakes in the past century have found the performance of these timber-masonry structures to be superior than other conventional structures in the Himalayan region.

Figure 5.1: Stone masonry houses with seismic resistant features of timber elements

5.2. Key features of seismic resistance of timber-masonry structures

The manner in which timber structural elements help masonry structures resist the lateral loads can be categorized as following :

- Connection of orthogonal walls: The timber lacing in timber-laced structures (timber bands at multiple levels – ground, sill, lintel, floor, roof) ensures effective connection between the orthogonal walls. This avoids separation of the transversal walls as well as corner damage.

- Timber cross members connecting timber on two faces of masonry act as the through-stones, reducing the possibility of delamination of the wythe.

- Timber bands at regular heights divides the slender masonry walls with large weights into smaller panels, reducing the possibility of overturning of out-of-plane wall.

- Timber beams/bands at the roof/floor level enables the distribution of the lateral load due to seismic action from the out-of-plane walls to the in-plane walls. The activation of in-plane walls helps in the structure attaining box behaviour further increasing the capacity of the masonry walls to resist the horizontal loads.

- The timber-frame (Dhajji-dewari) connects the openings to the frame, reducing the vulnerability induced by diagonal cracks emanating from the corners. The timber-frame reduces the discontinuity in the walls induced my multiple openings and reduces the stress concentrations at the corners of the openings.

- The timber-framing also divides the masonry into smaller sub-panels. This helps in limiting the crack lengths in masonry.

- The timber bands in timber-laced masonry arrest the vertical cracks in high and narrow walls reducing the impact of tension failure on a building.

- The overturning of the out-of-plane wall is avoided by the timber acting as the flexural member by providing the bending strength needed to resist the lateral loads induced by earthquake excitation.

- Dhajji-dewari also has timber framing in the gable wall. This reduces the vulnerability of gable wall to overturn by providing better connection of the gable wall to the load-bearing walls.

- Since the tensile strength of the masonry is very low, the structural timber members provide added tension strength to the walls increasing its resistance to the horizontal loads.

5.3. Resistance of timber-reinforced structures v/s conventional structures

There are multiple instances of timber-reinforced masonry structures performing considerably better than engineered or non-engineered Reinforced Concrete structures during earthquakes. Some examples are presented through pictures below:

Figure 5.2: A traditional timber-reinforced *hımış* building standing in the backdrop while several multi-storeyed modern RCC buildings suffered pancake collapse in Adapazari after the 1999 earthquake (Langenbach, 1999)

Figure 5.3: A *hımış* building suffered relatively lesser damage whereas a 4 storeyed RCC building next to it suffered devastation with ground floor collapsing completely after the 1999 Duzce earthquake (Doğangün et al., 2006)

Figure 5.4: A *hımış¸* timber reinforced masonry house survived with very little damage next to a collapsed four-story RC building after the 1999 Duzce earthquake (Doğangün et al., 2006)

Figure 5.5: Only minor hairline cracks were noticed in the 150 year old timber-masonry Swaminarayan Temple while more modern RCC structure in the complex (visible in the front) collapsed completely after the 2001 Bhuj earthquake (Photo credit : Randolph Langenbach)

6. Past studies on seismic behaviour of timber framed masonry

6.1. Literature reviews

Gani et al. (2021) conducted a review of post-earthquake behaviour of five different timber framed masonry systems across as many countries. For the Dhajji Dewari traditional construction system, it was found that the main features imparting earthquake resistance to the houses built using this technique were – a) the regularity and symmetry in geometry, b) the combined construction system of Bhatar (Taq) in the lower floors and dhajji-dewari in higher floors which lowers the centre of gravity of the building as a whole, c) small openings reducing the crack propagation.

Moreover, timber has multiple roles in increasing the earthquake resistance of this construction system:

a) The vertical timber posts (if provided) are placed closer together reducing the size of the wall panels. This helps in localizing the cracks to a particular smaller wall panel reducing its spread to other wall panels. This limits the damage and prevents out of plane failure.

b) pliable wooden floors allow movement of the walls allowing for higher absorption of seismic energy.

c) the carpentry connections are pinned, permitting limited movement in the joint improving the energy-dissipating capacity of the system.

6.2. Post-earthquake reconnaissance surveys

In their report on the Anantnag earthquake of February 20, 1967, Gosain and Arya (1967) suggest that the horizonal timber bands contribute significantly to the ductility of the Dhajji Dewari structures. They also observed that more the timber used in a building, lesser the amount of damage it underwent. They also mention that during the 1967 Anantnag earthquake, three-five storeyed building underwent relatively little to no damage. Arya (1970) also classifies diagonally braced timber buildings as highly suitable noting that they have "*minimum weight, high strength to lateral forces and high ductility or deformation capacity which are the most desirable qualities for resisting the applied forces and absorbing the kinetic energy fed into the structure by the ground shaking*". Well-built timber buildings have also been categorized into the low vulnerability category V1 in the Earthquake Vulnerability Assessment (Gupta et al., 2007).

According to Rai and Murty (2005), Dhajji-dewari construction - small masonry panels confined by timber elements, is different from typical brick masonry. This configuration also deems it superior in terms of seismic performance in the 2005 Kashmir earthquake as the "*timber studs…resist progressive destruction of the…wall…and prevent propagation of diagonal shear cracks…and out of plane failure*."

Timber-framed or timber-laced masonry also have another distinct advantage of the high equivalent hysteretic damping ratio from the friction induced in the masonry. While in uncracked modern masonry (brick with Portland cement mortar) and cracked modern masonry, this internal damping is of the magnitude of four and six-seven percent respectively; in Dhajji-dewari or taq wall, this internal damping is in the order of twenty percent. This has been substantiated by experiments comparing the damping of frames with infill of cement mortar masonry against those with mud mortar masonry infills (Dar et al., 2012). Prof. Arya's explanation for this is that "*there are many more planes of cracking in the dhajji dewari compared to the modem masonry*." Subsequently, timber framed masonry can be treated as membranes with joint action instead of frame systems of as masonry (Rai & Murty, 2005).

6.3. Analytical Studies

6.3.1. Full analytical study on Taq (Bhatar) building

Carabbio et al. (2018) conducted a full analytical study on the structural behaviour a typical Bhatar one room building unit. The study entailed a discussion on the material properties of the building materials used in the construction of a Bhatar structure, the geometry of the wooden bands used and static and seismic analysis (and assumptions) for the in-plane and out-of-plane seismic behaviour of a single Bhatar wall. The properties of the commonly used timber species in the Himalayan region, Shorea Robusta, and limestone as stated by the authors are listed in Table 7.1 and Table 7.4 in Paragraph 7.1 and 7.4.2.

The friction coefficient due to the vertical load and the self-weight was calculated using Barton's non-linear model that provides a more realistic estimation of the friction behaviour of the rock interfaces. The authors observed that for the in-plane analysis, the horizontal sliding of stones was the only significant failure mode as the diagonal cracking failure and the failure mode due to combined action of axial force and bending moment at the base of the wall are insignificant. For out-of-plane failure mechanisms, the horizontal timber beams provided resistance to overturning behaviour by providing tie-action and to the bending behaviour by acting as bond beams. The tie action provided by the horizontal timber bands results in the angle of wall rotating around the horizontal cylindrical hinge being larger than the angle of sliding mechanism, rendering the in-plane failure more significant than the out-of-plane failure. The study found that for the estimated vertical acceleration, no uplifting of the structure occurred and the Bhatar buildings are well capable of withstanding PGA values of 0.5g (Carabbio et al., 2018).

6.3.2. Equivalent static lateral force analysis

Rautela and Joshi (2009) conducted Equivalent static lateral force analysis of the Koti Banal structure according to the IS:1893 (part 1) (2002). The authors distributed the computed design base shear force for a particular building along its height. The resulting design lateral force at each floor was then distributed to individual lateral load resisting element depending upon diaphragm action. Koti Banal construction system uses wooden beams, joists and 20-22 mm thick planks for floors. In their research, they theorize that these highly flexible diaphragms have sufficient strength and stiffness to transfer the lateral loads from the transverse walls to the side shear walls. The design shear force was calculated to be around 23% of total seismic weight of the building. Additionally, their study also conducted radiocarbon dating of the wooden samples from the panels used in the subject buildings. The samples were dated to be around 880±90 years and 728±60 years from the year of study – 2009. This also establishes that the structures have undergone two major earthquakes – Kumaon Earthquake of 1720 and Garhwal Earthquake of 1803 without, suffering any major damage. It also points out to the fact that the highly seismic region of Uttarakhand had developed a seismic culture in their buildings at least 1,000 years ago.

6.4. Experimental Studies

6.4.1. Experimental Study on Seismic Capabilities of Dhajji-Dewari Frames

Dar et al. (2012), tested various configurations of timber-frames with different bracing patterns derived from field surveys. In their experiment, where they adapted a vertical loading frame to simulate static horizontal load, they found that joints were critical for assessing resistance of such frames. From the test results, they also concluded that "increasing the bracings … and strengthening the joints by iron straps increased the load carrying capacity of the frame by 300%." The properties of the materials and the frame used for the experiments are listed in Table 6.1.

Table 6.1: Material properties of Dhajji Dewari frames used for experiments

1	Poison's ratio	0.318
2	Modulus of elasticity	10900 KN/m2
3	Cross section of vertical main posts	0.1016m × 0.1016m
4	Cross section of bracings	0.1016m × 0.0508m
5	Height of frames	1.4478m
6	Width of frames	1.3716m

6.4.2. In-Plane Behaviour of the Dhajji-Dewari Structural System (Wooden Braced Frame with Masonry Infill)

Ali et al. (2012) conducted experimental and numerical investigations on the in-plane lateral load response of dhajji-dewari buildings. The experimental study consisted of in-plane, quasi-static lateral cyclic load tests on three full-scale wall samples (two with different ratios of stone-mortar in infill and one without infill) to determine properties such as in-plane lateral strength, ductility, stiffness, energy dissipation and damage mechanism. This was done by subjecting the walls to incremental quasistatic cyclic displacement as the horizontal load. The force deformation behaviour and damage mechanism of the three wall samples was compared by the lateral load versus drift ratio envelope curves, cyclic response and damping ratio versus drift ratio curves. It was found that the lateral load capacity of the system was dominated by the connections, while the main contribution of the masonry infill was to increase the energy dissipation capacity with higher viscous damping for the wall with infill. Therefore, tension and bending tests were conducted on different configurations of connections. The elastoplastic curves obtained from these tests were used to idealize the behaviour of connections in the nonlinear static pushover numerical model conducted to evaluate the lateral load capacity of dhajji-dewari structures. The FEM software SAP2000 was used to obtain the capacity (pushover curve) and a good match was observed between the pushover curve and the experimental curve stressing the point that nonlinear static pushover analysis can be used to calculate the in-plane capacity, if the dhajji-dewari walls are assumed to act like rigid diaphragms. The study finally concluded that the dhajji-dewari walls "*possess tremendous resilience against lateral forces*", a conclusion derived from the drift ratios. Another inference from the experimental study was the contribution of masonry infill only to the damping ratio while not contributing significantly to the in-plane lateral load capacity which is dominated by the connections (Ali et al., 2012). Mechanical properties of the timber used in the construction of the walls are listed in Table 6.2.

Table 6.2: Mechanical properties of timber used in the construction of the walls for the experiment

Timber sample #	Compressive strength parallel to grain (MPa)	Tensile strength perpendicular to grains (MPa)	Modulus of rupture (MPa)	Modulus of elasticity (MPa)
1	23.54	2.35	65.90	4439.67
2	27.85	1.57	70.22	2988.18
3	27.07	1.57	66.19	3441.55
4	28.93	2.35	62.57	3296.02
Average	**26.87**	**1.96**	**66.19**	**3345.24**

6.4.4. Experimental study on traditional assam-type wooden house for seismic assessment

Assam-type or Ikra houses is another typology from Eastern Himalayan region which has received attention because of its superior seismic performance. Monotonic and slow cyclic lateral loads exerted on full scale frames demonstrate that the wooden frame, either with Ikra infill walls or without the infill walls display excellent drift and ductility behaviour due to the high lateral displacements achieved without substantial decrease in lateral load carrying capacity (Chand et al., 2017). Quasi-static cyclic tests and pull out tests to assess the performance of different types of connections in a typical Assam-type house also showed positive results with high deformability and ductile behaviour (Chand et al., 2020b).

According to Chand et al. (2017), the main vertical posts not being rigidly connected to the foundation lead to limited rotation in one direction while preventing it in other direction. This property along with absence of any rigid connections in the entire frame results in lesser likelihood of failure of main joints.

7. Seismic analysis: Analytical considerations for a Bhatar building

Bhatar buildings are one of the most common types of buildings found in the Himalayan region. Their popularity ranges Karakoram (Gilgit-Baltistan, Kashmir), Western Himalayas (Kashmir, Himachal Pradesh, Uttarakhand) to Central Himalayas (Nepal). They exist in both rural and urban regions. Consisting of dry stone masonry, the walls are interspersed with horizontal timber bands at regular intervals. Carabbio et al. (2018) conducted a full analytical study into the structural behaviour of a Bhatar building. Dimensions of the building, material properties and loading cases were characterized. In-plane seismic analysis conducted and based upon these characteristics yielded an acceleration of 0.5 g under which in-plane failure mechanisms developed. This was based on assumption that no vertical ground motion existed. Moreover Carabbio (2016) interprets that the energy dissipation in buildings built by Bhatar technique mainly happens through the friction between the stone layers. The shear strength of the stone was determined through Barton's rockfill model. The analytical study presented in the current study derives certain aspects like material properties, geometry and Barton's model from the research conducted by Carabbio (2016). However, in addition to the seismic analysis of a Bhatar building, this study compares the behaviour of an unreinforced stone masonry building with a comparable Bhatar building with horizontal timber bands. The effect of a traditional heavy mud roof is also compared to the effect of a lighter roof with timber beams and planks. In addition, the effects of geometry on the seismic capacity of the buildings are studied.

Some assumptions made for this analytical model are presented below:

- Vertical ground acceleration is ignored.
- The contribution of out-of-plane walls to the seismic behaviour of the entire structure, when loaded in-plane, is neglected.
- The roof beams are assumed to be connected to only the main walls (in-pane walls). This scenario is also closer to the existing buildings.
- The contribution of the joints between the orthogonal timber beams is neglected in the in-plane seismic capacity.

7.1. Materials

The major components of the construction of a Bhatar building – foundation, wall, floor, roof and openings, use wood and stone as the major materials. As with other forms of vernacular construction, this technique also developed heavily dependent on the local materials available in the region. Limestone is often the most appropriate stone for construction in the region as it is easily available locally and has good strength properties. Properties of limestone and timber considered for this study are presented in Table 7.1.

Table 7.1: Material properties of timber and limestone

	Specific weight of the stones (Υ_{stone})	26.86 kN/m^3
	Specific weight of the rubble stones (Υ_{stone})	19.88 kN/m^3
	Specific weight of timber (Υ_{timber})	9.00 kN/m^3
	Void ratio	0.26
	Porosity (n%)	20

7.2. Geometry and dimensions

A single room structure was considered for this analysis with the dimensions of 3.60 m x 3.60 m x 3.0 m (length x breadth x height). The 3.60 m walls were considered for the in-plane analysis. The thickness of the wall is 450 mm as observed in the field. Though, it is to be noted that conventional Bhatar buildings are not of this particular size as they are more often than not multi-storeyed With 3 storeyed being a more common occurrence. However this geometry is adopted for the purpose of this analysis given the time constraints for this additional graduation project. Considering the geometry that is closer to actual buildings will yield more realistic insight into the behaviour of the Bhatar buildings. For example, the additional storeys added will result in higher overburden stress while also increasing the seismic weight.

7.3. Unreinforced stone masonry

A large number of structures in the Himalayan region are built using random rubble masonry. Irregularly shaped stones are gathered locally and assembled into masonry by local artisans or stonemasons. Reinforcing elements to carry bending and tension like steel or timber are not used. These structures have been seen to have substantial failures, even collapsing at times, in the past earthquakes. The walls are assumed to have enough stiffness to behave as a wall.

7.3.1. Failure modes

Three global failure modes are considered for this unreinforced stone masonry building: 1) Sliding at bottom, 2) Rocking and 3) Toe crushing. The overburden force on the wall, the self-load and the allowable acceleration is also reported in Table 7.2 and Table 7.3. This study shows that even for the most critical failure mode – bottom sliding, the allowable acceleration is only 0.235. This is lower than the recently seen PGA values in the Himalayan region reaching up to 0.5. This also exemplifies that under the normal earthquakes, this stone masonry will fail leading to substantial loss of life or property.

Table 7.2: Weight of roof and walls of unreinforced stone masonry

Weight of 1 IP wall	9847.040	kg	96.566	kN
Weight of 1 OOP wall	7385.280	kg	72.425	kN
Weight of 4 walls	34464.641	kg	337.983	kN
Weight of roof (3.6m x 3.6m), W_{roof}	10444.630	kg	102.427	kN
Total weight of system, (W_{total})	44909.271	kg	440.410	kN
Weight on 1 IP wall	22454.636	kg	220.205	kN

Table 7.3: Global in-plane seismic resistance of loaded wall for unreinforced stone masonry

$\sigma_{c,90,d}$	71.600	Mpa	
Weight of roof, F	51.213	kN	
Self-weight of wall, W	96.566	kN	
Length, l	3.600	m	
Height, h	3.000	m	
Width, w	0.450	m	
Friction coefficient, μ	0.350		
			allowable acceleration factor
Bottom sliding , V_{sl}	51.723	kN	0.235
Rocking, V_r	88.668	kN	0.403
Toe crushing, V_t	88.517	kN	0.402

7.4. Bhatar – timber reinforced masonry

To analyse the contribution of the horizontal timber bands in the Bhatar structures, the in-plane seismic capacity is calculated for comparison with that calculated for an unreinforced stone masonry wall with similar geometry and material properties but with no timber bands.

7.4.1. Assumptions

- Only the behaviour of the in-plane wall is characterized through this analytical model.
- The roof is assumed to be resting only on the in-plane walls. This means distributing all the load of the roof and floors to the two in-plane walls.
- The contribution of the out-of-plane walls is neglected in the calculation of the in-plane capacity of the system.

- The diagonal cracking is assumed to not occur in this typology because of two major reasons – a) continuous interjection of the masonry mass by the horizontal timber beams, and b) “already cracked conditions” created by the lack of any sort of mortar.

Due to these assumptions, it can be hypothesized that the failure would occur because of the horizontal sliding mechanism. The failure under this mechanism will in turn be dependent on the friction between the two major materials – stone and timber. This can further be broken down into the friction occurring at two significant interfaces – stone-stone and stone-timber interfaces.

7.4.2. Shear strength of rockfill – Barton’s model

Carabbio (2016) also considers the Barton’s model to describe the shear behaviour in rock joints. The shear strength of an in-plane stone masonry wall is largely dependent on the friction and interlocking of the stones in the wall.

$$\tau_p = \sigma_n * \tan\left(R * \log_{10}\frac{S}{\sigma_n} + \phi_r\right),$$

Where,

τ_p = peak shear strength
σ_n = applied normal stress
R = Roughness
S = Strength
ϕ_r = residual friction angle

The properties for the interface of stone-stone and stone-timber are listed in the Table 7.4. These properties are derived from literature (Carabbio, 2016).

Table 7.4: Interface properties for stone-stone and timber-stone interfaces

	Stone-stone Interface properties	
1	Porosity (n%)	20
2	Roughness (R)	10
3	Particle size diameter (d_{50})	100 mm
4	Unconfined compressive strength, UCS (σ_c)	71.3 MPa
5	Strength (S)	0.7 σ_c = 50 MPa
6	Residual friction angle (ϕ_r)	13°, 18°, 24°
7	Variation for R and S	±20%
8	Coefficient of variation for UCS	14%
9	S/ σ_c	0.7
10	Variation in shear strength due to roughness (R) variation	+25%
11	Shear strength increase due to friction angle	+50%
	Timber-stone Interface Properties	
12	Area reduction factor ($\xi = A_{t-s}/A$)	0.57

This empirical formula Barton gives the peak shear strength and the friction coefficient for both stone-stone interface and the timber-stone interface, based on the normal stresses for the specific layers, as reported in Table 7.5 and Table 7.6. To calculate the shear strength of the wall (dependent on the stone-stone and timber-stone interface, the Barton’s empirical formula is used.

Table 7.5: Normal stresses, peak shear strength and friction coefficient for each stone-stone layer

Layer	Normal stress (kN/m2)	σ_n (N/mm2)	τ_p (N/mm2)
σ_1	31.613	0.032	0.066
σ_2	35.757	0.036	0.079
σ_3	44.726	0.045	0.096

σ_4	56.490	0.056	0.112
σ_5	68.254	0.068	0.128
σ_6	80.018	0.080	0.141
σ_{ground}	90.004	0.090	0.145

Table 7.6: Normal stresses, peak shear strength and friction coefficient for each timber-stone layer

Layer	Normal stress (kN/m2)	σ_n (N/mm2)	τ_p (N/mm2)
σ_1	33.000	0.033	0.063
σ_2	39.030	0.039	0.072
σ_3	50.540	0.051	0.090
σ_4	62.040	0.062	0.105
σ_5	73.540	0.074	0.122
σ_6	85.040	0.085	0.137
σ_{ground}	91.010	0.091	0.146

7.4.3. Shear resistance

The friction coefficient for each layer, when multiplied with the vertical load on each wall gives the resisting shear force for the corresponding layer. The layer with lowest shear force will correspond to the critical value for the allowable amplification factor for the structure. The shear resistances are calculated for both possible failures of interactions – stone-stone and timber-stone. Moreover, the application for the load has three possibilities – at the top of the wall, linearly varying distributed load, uniformly distributed load; as reported in Table Table 7.7, Table 7.8 and Table 7.9 for stone-stone interfaces and Table 7.10, Table 7.11, and Table 7.12 for stone-timber interfaces, respectively.

Table 7.7: Shear resistance and allowable acceleration factor for stone-stone interface for force applied at the top of the wall

Different layers	Load on each layer (Normal stress * area) W_i (kN)	R_{si} = Resisting shear force on each layer (kN) τ_p * A	allowable PGA F_s * PGA < τ_p * A
W1	51.213	106.920	0.486
W2	57.926	127.980	0.581
W3	72.456	155.520	0.706
W4	91.514	181.440	0.824
W5	110.572	207.360	0.942
W6	129.629	228.420	1.037
W7	145.806	234.900	1.067

Table 7.8: Shear resistance and allowable acceleration factor for stone-stone interface for linearly varying distributed force applied

Different layers	Weight of each layer W_i (kN)	Height of the layer from ground H_i (m)	W_i * H_i (kN)	Distribution factor = β_j	R_{si} = Resisting shear force on each layer (kN) τ_p * A	allowable PGA F_j * PGA < τ_p * A
W1	19.31	3.150	60.822	0.532	106.920	0.912
W2	4.844	2.930	14.192	0.124	127.980	0.885

W3	6.353	2.460	15.627	0.137	155.520	0.890
W4	6.353	1.860	11.816	0.103	181.440	0.919
W5	6.353	1.260	8.004	0.070	207.360	0.974
W6	5.392	0.660	3.559	0.031	228.420	1.040
W7	1.646	0.150	0.247	0.002	234.900	1.067

Table 7.9: Shear resistance and allowable acceleration factor for stone-stone interface for uniformly distributed force applied

Different layers	Weight of each layer W_i (kN)	Height of the layer]H_i (m)	W_i * H_i (kN)	Distribution factor = β_j	R_{si} = Resisting shear force on each layer (kN) τ_p * A	allowable PGA F_j * PGA < τ_p * A
W1 + W_{roof}	19.31	3.150	60.822	0.384	106.920	1.264
W2	4.844	2.930	14.192	0.096	127.980	1.209
W3	6.353	2.460	15.627	0.126	155.520	1.163
W4	6.353	1.860	11.816	0.126	181.440	1.123
W5	6.353	1.260	8.004	0.126	207.360	1.095
W6	5.392	0.660	3.559	0.107	228.420	1.072
W7	1.646	0.150	0.247	0.033	234.900	1.067

Table 7.10: Shear resistance and allowable acceleration factor for stone-timber interface for force applied at the top of the wall

Different layers	Load on each layer (Normal stress * area) W_i (kN)	R_{si} = Resisting shear force on each layer (kN) τ_p * A	allowable PGA F_s * PGA < τ_p * A
W1	53.460	56.916	0.258
W2	63.229	65.160	0.296
W3	81.875	80.964	0.368
W4	100.505	94.689	0.430
W5	119.135	109.350	0.497
W6	137.765	123.714	0.562
W7	147.436	131.409	0.597

Table 7.11: Shear resistance and allowable acceleration factor for stone-timber interface for linearly varying distributed force applied

Different layers	Weight of each layer W_i (kN)	Height of the layer from ground H_i (m)	W_i * H_i (kN)	Distribution factor = β_j	R_{si} = Resisting shear force on each layer (kN) τ_p * A	allowable PGA F_j * PGA < τ_p * A
W1	18.11	3.150	57.365	0.502	56.916	0.515
W2	3.330	3.025	10.073	0.088	65.160	0.501
W3	6.350	2.700	17.145	0.150	80.964	0.497
W4	6.350	2.100	13.335	0.117	94.689	0.502
W5	6.350	1.500	9.525	0.083	109.350	0.528
W6	6.350	0.900	5.715	0.050	123.714	0.567
W7	3.290	0.300	0.987	0.009	131.409	0.597

Table 7.12: Shear resistance and allowable acceleration factor for stone-timber interface for uniformly distributed force applied

Different layers	Weight of each layer W_i (kN)	Height of the layer]H_i (m)	W_i * H_i (kN)	Distribution factor = β_j	R_{si} = Resisting shear force on each layer (kN) τ_p * A	allowable PGA F_j * PGA < τ_p * A
W1 + W_{roof}	18.11	3.150	57.365	0.384	56.916	0.713
W2	3.330	2.930	9.757	0.096	65.160	0.690
W3	6.350	2.460	15.621	0.126	80.964	0.662
W4	6.350	1.860	11.811	0.126	94.689	0.631
W5	6.350	1.260	8.001	0.126	109.350	0.615
W6	6.350	0.660	4.191	0.107	123.714	0.601
W7	3.290	0.150	0.494	0.033	131.409	0.597

The final allowable amplification factor for acceleration are reported in the Table 7.13 .

Table 7.13: Allowable acceleration amplification factor

Allowable acceleration amplification factor			
Unreinforced Stone masonry			
	Bottom sliding	Rocking	Toe-crushing
	0.235	0.403	0.402
Bhatar - timber-reinforced masonry			
Stone-stone interface between stones			
Layers	At the top of the wall	Linearly varying distributed load	Uniformly distributed load
W1	0.486	0.912	1.264
W2	0.581	0.885	1.209
W3	0.706	0.890	1.163
W4	0.824	0.919	1.123
W5	0.942	0.974	1.095
W6	1.037	1.040	1.072
W7	1.067	1.067	1.067
Stone-timber interface below timber bands			
Layers	At the top of the wall	Linearly varying distributed load	Uniformly distributed load
W1	0.258	0.515	0.713
W2	0.296	0.501	0.690
W3	0.368	0.497	0.662
W4	0.430	0.502	0.631
W5	0.497	0.528	0.615
W6	0.562	0.567	0.601
W7	0.597	0.597	0.597

The results demonstrate the clear contribution of the horizontal timber bands to the increased seismic response of the Bhatar in-plane walls. While the shear resistance of unreinforced stone masonry shows that the wall would fail through the bottom sliding mechanism at a PGA of 0.235, it is probable that the wall doesn't slide from the bottom due to the non-cohesive nature of the masonry and it might explode instead. The results also show that out of the two interfaces, stone-timber interface below the timber band has lower resistances for each layer and hence is more critical than the stone-stone interfaces between the stone layers. This is obvious due to the lower friction coefficient and lower contact surface area for the former.

It is also noted that while the acceptable acceleration factor for the bottom sliding at the bottom of the wall is equal due to seismic weight for all three distributions of loading – at the top of the wall, linearly varying and uniformly distributed; it varies significantly at the top three layers of the wall with the point load being considerably more critical than linearly varying load. While the topmost layers are most critical for linearly varying load, in the case of uniformly distributed load, the bottom layers have lesser seismic resistance. This can be explained by the fact that linearly varying load has a significantly higher lateral load on the top layers, while having lesser overburden stress.

Another point of significance is that for linearly varying distributed load, the top most layer is not the most critical, but rather the second and third layers are the most critical. This might be due to the topmost layer having less seismic weight, while the bottom layers having high overburden stresses and hence more seismic resistance.

The linearly varying distributed load is a closer representation of the lateral loads caused by earthquake excitation however effects of both uniformly distributed load and linearly varying distributed load are recommended for pushover analysis.

8. Conclusion

Traditional building systems have evolved over centuries as a part of local seismic culture in the seismically active Himalayan region. The high frequency of earthquakes in the Himalayas lead to loss of thousands of lives every decade. The study of seismic response of traditional masonry structures with timber structural elements is crucial in enhancing the capabilities of communities living in this region and prevent loss of lives.

Failure modes of unreinforced masonry are studied in detail to explore the critical details and points of weaknesses. In-plane masonry is found to be a more suitable failure mode as it enables a ductile failure. This also results in higher capacity of the overall structural system. In-plane failures have been observed in the field mainly as diagonal or shear and flexural cracks which gives the inhabitant enough time and warning to exit the building. This is in contrast to the brittle out-of-plane failure modes which have a tendency to occur suddenly like overturning of out-of-plane walls or gables.

A review conducted on the state-of-the-art of experimental investigation revealed that very little scientific research has been carried into understanding the behaviour of earthquake resistant traditional building typologies. Although a few experimental studies are conducted on the seismic capacity of Dhajji Dewari or Kath Kuni walls, there's a need for more tests and experiments to develop and validate simple analytical models that would help in easier design and assessment of such structures.

An analytical model was used to assess the seismic capacity of a Bhatar wall. Bhatar is a building technique used across Northern, Western and Central Himalayas. It includes horizontal timber bands at regular intervals in a stone masonry wall. Only in-plane behaviour was studied due to time constraints of this project. The material properties were derived from the literature and Barton's model was used to calculate the peak shear strength of the rockfill joint. This model was used for its appropriateness for delivering similar values for the random rubble stone masonry as the experiments do. The shear resistance of an unreinforced stone masonry wall was compared with that of the Bhatar building.

The analysis confirmed that the horizontal timber bands had a significant positive impact on the in-plane wall by increasing its shear resistance substantially. However, it also revealed that instead of the topmost layer, second or third layer from the top is the most critical when it comes to the shear resistance of the layer under linear varying uniformly distributed load. It was also found that the linearly varying distributed load had a more critical factor for allowed acceleration. Finally, it is found that the most critical allowable acceleration factor for an in-plane Bhatar wall is 0.497, which is also close to acceptable value of allowable PGA of 0.5 in Himalayan region. PGA value of 0.5 is also close to the excitations of the recent earthquakes. However, it should be noted that this value is for an in-plane wall which didn't consider the contribution of out-of-plane walls. Additionally, since masonry structures lie on the plateau of the response spectrum, the seismic resistance of the entire system is expected to be much more than that of a single in-plane wall.

It is to be noted that the analytical consideration was limited to only an seismic analysis of the in-plane Bhatar wall. In order to obtain a more realistic estimation of the seismic resistance of such a structural system, it would be more appropriate to build a numerical model. However, this was beyond the scope of this thesis.

Future research can look into additional analysis of an entire building by modelling joints and connections as well as the out-of-plane walls. In terms of additional details future studies can assess effects of different types of floors (concrete, timber and traditional mud-based floor), behaviour of multi-storeyed buildings and behaviour of cluster of buildings.

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