Exploration and analysis of low-cost seismic retrofit measures to improve box-action for traditional brick masonry houses in Nepal
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Exploration and analysis of low-cost seismic retrofit measures
to improve box-action for traditional brick masonry houses in Nepal

THESIS
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Picture front page:
Vertical crack at the junction of perpendicular walls of a traditional brick masonry house in Nepal
(Source: own image)

Picture this page:
Box-action (Source: Touliatos, 1996)
This document contains the report of my thesis for the master’s degree in Civil Engineering at the Delft University of Technology. The origins of the research stem from a study-related visit to Nepal. There it became clear how much damage the traditional brick masonry houses had suffered during the recent earthquakes (Gorkha 2015). The vast amount of damage of this typology demonstrated also the vulnerability of the houses which are still standing. All made me wonder what could be done to improve the condition of these houses. With the help of Jitendra Bothara and Prem Nath Maskey I have formulated a thesis research aiming to study low-cost retrofit measures which could improve the structural integrity of the traditional brick masonry houses. The research was performed under the guidance of Delft University of Technology and the engineering firm Arup from November 2015 until October 2016.

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Emilie van Wijnbergen
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Figure 0.1 Elevation of a street in the heritage settlement Bungamati, drawn in 1968, Nepal (Source: Jorgen Thomson)
SUMMARY

Nepal and the Kathmandu Valley lie in a highly seismic region. The traditional brick masonry houses built with mud-bound masonry and timber are an important part of the local heritage and Newari culture. These houses are characterized by thick load bearing walls, very flexible timber diaphragms and decorated timber features in the façades. Deformation capacity in the mud mortar and connections allows the buildings to sway during an earthquake and absorb some of its energy. The configuration and design of the vernacular houses demonstrate a thorough knowledge of basic seismic principles, but the low-strength masonry and lack of sufficient connections between building elements make these structures vulnerable to earthquakes. Research is needed on how to make these houses more earthquake resistant, preferably without interfering with the original architecture. Tying the building elements together and thereby improving the box-action of vernacular houses could increase their overall seismic performance. However, a complicating factor for the seismic retrofit is that the majority of Nepalese people have very limited financial means and existing solutions are not tailored to the Nepalese circumstances.

At the time of writing there are no numerical or experimental studies on upgrading connections or diaphragms for these vernacular houses. Therefore, the main question asked in this research is: “How to improve the connections and diaphragms between building elements of traditional brick-masonry buildings in Nepal for future earthquakes?” This question is approached from the following perspectives: “which retrofit techniques are affordable, accessible and integrable?” and “what is the effect of the retrofit measures on the global seismic performance?”

By means of interviews with structural engineers, a list is set-up of retrofit concepts and Nepal-specific design- and construction criteria to improve connections and stiffen diaphragms. The principle criteria are: structural efficiency, impact on architecture, durability, constructability, cost-effectiveness and material availability. For box-action, both upgrading of connections, as stiffening of the diaphragm is required. Therefore integral solutions are preferred which incorporate both. A colour-coded rating system is used to evaluate which are suitable and feasible techniques. Outcome suggests different approaches for urban/rural areas versus remote areas. Urban and rural areas are generally accessible by road for transportation of building materials and have more access to construction knowledge and equipment; allowing solutions including widely available materials such as reinforced concrete. A horizontal bandage around the perimeter of the building can tie the building together comparable to a belt. The diaphragm can be stiffened with an in-situ concrete floor overlay. Standard rebar meshes can be used. Reinforcement rods, laid into the in-situ concrete can function as wall-ties connecting the overlay and external bandage. Remote areas generally have very low accessibility and less access to knowledge and technology, favouring the use of local or light materials and more low-tech solutions. Wall-ties secured by plate-anchors are suggested to connect wall-to-floor, combined with diagonal timber bracing of the diaphragm with timber planks. The wall ties are made with galvanized steel perforated strips, for which no welding is needed. Timber slats are nailed to the strip-end outside of the wall as plate anchors.

A series of numerical analyses is done with the aim to investigate the effect of the retrofit strategies on the seismic performance of the vernacular houses. Nonlinear time history analysis is chosen as calculation method; this method is believed to yield the most accurate results as it can capture nonlinear material behaviour, dynamic and duration effects. As the nonlinear behaviour of the composite masonry material is complicated, some confidence is needed to establish if the behaviour of the existing low-strength masonry buildings can be captured sufficiently with the chosen modelling strategy. Therefore a comparison is made of a numerical model to a shake table test on a scaled low-
strength masonry structure from literature, by Sathiparan (2016). The purpose is to evaluate the capacity and limitations of the applied modelling method for this application. In the experiment from literature, a box-shaped model with- and without a roof were subjected to 43 runs of sinusoidal acceleration cycles in one direction. The first model consisted only of four walls, a window and a door opening. A second model is built with a timber roof structure fixed to the walls. For this research, a numerical model is built in LS-DYNA, in which the masonry walls are modelled with shell elements and the masonry behaviour is represented by a smeared crack material model, developed by Arup. The shaking table test is simulated, using test-specific material properties and applying the same input motions as the experiment. Results show that global crack patterns are reasonably well reproduced, and the main collapse mechanisms are captured. The analyses also show that the structure without roof is excited mostly in primary (out-of-plane) modes. As the walls bulge out- and inwards, it is hard to capture a representative measurement for storey drift. It can be clearly seen that the second model with roof has less out-of-plane damage, and more in-plane, similar to the experimental results. Sensitivity studies show that the model is sensitive to the input parameters Young’s Modulus and Diagonal Tension value. It is recommended to apply a Young’s modulus of $\sim 300\, MPa$ to better approach the lateral stiffness, and to increase the diagonal tension parameter. To optimize simulation results, the material model could be calibrated to masonry-type specific component tests.

Secondly, a full-scale, one storey model, comparable to one bay of a vernacular house is modelled to simulate the effect of the retrofit measures on the global seismic behaviour. Initially the unstrengthened model (friction only connections, no diaphragm) is compared to a model with a stiff diaphragm and fully fixed connections. Modal analysis shows that the flexible unstrengthened model is excited in primary modes (out-of-plane) by many (+38) modal shapes, whereas the tied model only demonstrates 3 main modes, representing an in-plane 3D structural response. As a next step, nonlinear time history analyses are performed for a typical ground motion (El Centro) on the two extreme cases (no diaphragm and friction only connections), versus the stiff diaphragm and fully fixed connections. Subsequently several intermediate variations with retrofit measures are studied.

The unstrengthened model sees many cracks in out-of-plane walls, high out-of-plane displacement and relatively low storey displacement. The model with stiff diaphragm and fixed connections shows that the damage concentrates at the fixation of wall-to-floor. The models with implemented retrofit measures show a significant increase in storey displacement, but a drastic decrease in out-of-plane diaphragm displacement compared to the unstrengthened model. Comparison of damage propagation shows that the damage is shifted towards the in-plane walls. The in-plane cracks differ from the typical X-cracks. As the mortar shear strength is very low, and the upgraded diaphragm relatively stiff compared to the mud-bound masonry, severe horizontal bed-joint sliding is seen at the line of the diaphragm. These cracks cause softening of the model, and result in high storey drifts.

From the analyses it can be concluded that the amount of damage is not necessarily lower, but changed in nature from out-of-plane to in-plane failure modes. This type of damage is less critical, and is seen as more favourable as it is more ductile. In this way, the retrofits do provide an improved situation. Since in-plane damage is expected, especially if the masonry is of low quality, the retrofit of only connections and diaphragms may not be sufficient. It seems reasonable that the minimal interventions are not a panacea, solving all vulnerability. It might prove beneficial to distribute the load over the height of the wall, instead of only fixing at floor level. Furthermore, the connections might be too stiff (in the way that they are designed, or modelled). If more deformation is allowed, (such as in the timber wedges of original construction) less damage concentration is expected at the fixation of the diaphragm, and more energy could be dissipated.

As an overall retrofit approach it is suggested first to perform an assessment of the initial conditions in order to decide if a retrofit is feasible at all. When the masonry wall is of sufficient quality, with sufficient interlocking between wall wythes, retrofit may be feasible. It is suggested to better distribute
the seismic loads over the height of the wall, and to allow more deformation and ductility in the connections. Vertical timber strongbacks and vertical mesh strips can provide extra anchoring possibilities. The timber anchor plates can be carved to blend in the traditional façade, and the horizontal bandage can be designed as a decorative ridge at storey level (which is not uncommon for these houses). When retrofitting in dense urban areas with arrayed houses, there are advantages of ups scaling the retrofit to multiple houses. For example: due to horizontal alignment of storey heights the bandage can be extended to the houses of the neighbours.

If the masonry wall is vulnerable to delamination and is of very low quality it might be better to (partially) rebuild the structure. Because in this case the walls would need intensive strengthening measures, leading to high costs and the risk of excessive interference with the architectural identity of the house. Rebuilding in traditional style fits into the cultural building tradition. Materials can be partially re-used. If properly maintained and seismic provisions are well-incorporated traditional construction can be resilient up to three stories. Rebuilding with modern concrete frames will provide opportunity to offer modern day ceiling heights and higher buildings. However, this is only recommended if the knowhow for adequate concrete detailing is present.

For future research, more experimental tests are required on mud-bound masonry components and connections in order to calibrate numerical models for low-strength masonry structures. As this research was focused on simplified box-models, it is recommended to study the effect of upgrading connections and diaphragms on a numerical model of a full sized vernacular house with multiple stories.
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Chapter 1 Introduction

1 INTRODUCTION

1.1 Context

Over the ages masonry has extensively been used as a building material because of its local availability, economical construction and thermal insulation properties. Low-strength masonry, containing bricks bound with mud-mortar, has been frequently adopted for housing in areas with high seismic risk, such as Iran, Nepal and Peru. However, the inherent properties of the material (heavy and brittle) make masonry structures vulnerable for seismic excitations. Due to the collapse of structures, earthquakes have caused a tremendous loss of life and assets in these regions.

The recent earthquakes in Nepal have clearly demonstrated the seismic vulnerability of unreinforced masonry buildings. This threat is ongoing, as Nepal is a highly seismic area. The collision of the Indian and the Eurasian tectonic plates causes a continuous build-up of tectonic stresses in the Himalayan range. Large and devastating earthquakes tend to occur in cycles of 75-100 years (Pradhan, 2000), (EERI, 2016). The seismic events of 2015 caused more than 8790 casualties and 22,300 injuries, due to building collapse and landslides. The prevalent building methods in Nepal are low-strength masonry, cement-based masonry and reinforced concrete frames. In the affected districts low-strength masonry structures, built with natural stone or clay brick and mud-mortar, make up most (58%) of the housing stock (NPC, 2015a). This building type has suffered the largest extent of destruction and partial damage (Table 1.1).

<table>
<thead>
<tr>
<th>Construction type</th>
<th>Fully collapsed or beyond repairs</th>
<th>Partially damaged</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low-strength masonry</td>
<td>474,025 (95%)</td>
<td>173,867 (67,7%)</td>
</tr>
<tr>
<td>Cement-based masonry</td>
<td>18.214 (3,7%)</td>
<td>65,859 (25,6%)</td>
</tr>
<tr>
<td>Reinforced concrete frame</td>
<td>6.613 (1,7%)</td>
<td>16,971 (6,7%)</td>
</tr>
<tr>
<td>Total</td>
<td>498,852</td>
<td>256,697</td>
</tr>
</tbody>
</table>

Table 1.1 Building typologies and damage (Source: National Planning commission, PDNA report vol.B)

![Figure 1.1 Seismic hazard zones of the world (Source: NG MAPS)](image)
Traditional masonry buildings

In the historical settlements of the Kathmandu valley, traditional masonry houses have experienced a substantial amount of damage. In these settlements, houses from the traditional Newari architecture are common and have significant heritage value, some being more than 200 years old. The buildings are characterized by masonry facades with elaborately carved timber window frames – demonstrating excellent craftsmanship in woodworking amongst the Newars.

A vast continuity in building style has resulted in the construction of many traditional brick masonry buildings with a relatively similar building configuration and material use (brick, wood and tiles) over the ages. In urban areas these buildings generally count 3 to 4 stories, whereas in remote areas 2 stories are more common. The main load bearing system is formed by unreinforced mud-bound brick masonry walls. The plan is divided by an internal spinal wall (duāga) which in upper stories is sometimes replaced by a timber colonnade. The floors are built up out of closely spaced beams joists, spanning from façade to the spinal wall, topped with planks. Timber beams form the trussed roof, which is covered by a thick layer of clay and tiles.

Vulnerabilities

The most important vulnerabilities of these buildings are the lack of connections between building elements and the inherent properties of their main materials. Unreinforced masonry is brittle and has limited capacity to cope with earthquakes. The bricks are laid in mud-mortar, which has low shear strength and very limited tensile strength (Bothara et al., 2004), making masonry prone to cracking when subjected to seismic excitations. However, as the mortar starts to yield under relatively small accelerations, energy could be dissipated through frictional sliding, if the walls are tied well together. The thick walls offer stability and resistance, but also induce high inertia forces. The walls are generally built in multiple wythes ¹, making them vulnerable for delamination.

Due to insufficient connections between structural elements many houses have low structural integrity. The orthogonal walls are simply butt jointed without sufficient interlocking. Buildings mostly have extremely flexible floors and roofs with negligible in-plane stiffness. Timber planks or bamboo strips are simply lying on top of the timber joists, without having any nailed connection (Bothara et al., 2004). A thick layer of mud on the floors provides excessive mass, without contributing to the load bearing system. The floors are not anchored to the walls, nor are the gable walls tied to the roof, which causes them to act as freestanding cantilevers. Over time, bad maintenance has further deteriorated the strength of timber features such as lintels and occasional shear locks in beams (Maskey, 2012) which play an important role in the seismic resistance. Also modern modifications such as additional stories, the addition of larger windows and heavy concrete slabs increase the vulnerability of these buildings.

These deficiencies have led to multiple forms of damage (NPC, 2015b). ‘Gable wall toppling, delamination of (multi-wythe) low-strength masonry walls, out-of-plane toppling of walls, corner separation, various types of wall failures under in-plane loading such as diagonal cracks, sliding cracks, crushing of piers, failure of spandrels; and collapse of floor and roof due to loss of vertical load-bearing elements such as walls’ were reported by NSET² after the earthquakes of 2015.

Despite the encountered damage and seismic deficiencies, it is notable that the basic concepts of seismic resistant design were very well understood by the Newars, who were bound to use the materials which were available at the time (brick, mud, timber). Regular building plans and elevations, limited window openings, weight reduction towards the top and allowing the houses to be flexible,

¹ A wall wythe or leaf is a vertical section of masonry with the thickness of one unit. It may be independent or interlocked with adjacent wall wythes. https://en.wikipedia.org/wiki/Wythe
² National Society for Earthquake Technology
are examples of the employed strategies to minimize seismic loads. An overview of seismic deficiencies and resilient features of the Newari building style is given in Appendix A1.

**Need for retrofit**
The building industry is increasingly choosing modern construction methods and materials, causing a rapid change in appearance of traditional settlements (Korn, 2007). However, although many unreinforced masonry structures have collapsed in the earthquakes of 2015; traditional brick masonry buildings are still widely in use. Furthermore it is expected that people will keep constructing low-strength brick masonry buildings or keep on living in dangerous or damaged buildings with deficient seismic performance, where financial means are limited. Many of these buildings are of insufficient performance to cope with future earthquakes, which causes a risk to their inhabitants and a threat of losing valuable heritage structures (Shrestha et al., 2012).

Although for some structures it can be argued that demolition is a better option than repair and strengthening, the replacement of all unsafe masonry structures is not feasible (Shrestha et al., 2012), nor desirable. As Macuabuag (2010) argues: ‘they are often the embodiment of local culture and tradition’. Therefore it is necessary to consider the feasibility of seismic retrofit measures for existing structures.

A complicating factor for the seismic retrofit is that the majority of Nepalese people have very limited financial means, especially for retrofit, as they are faced daily with more urgent basic needs. However, such retrofit interventions could in the long term prove to be the cheaper option when compared to the scenario ‘doing nothing’ and then having to reconstruct after the next large earthquake. Due to the aforementioned economic context of Nepal, many existing solutions in reference research from other countries are too expensive, using materials such as FRP (Fibre Reinforced Polymers) or steel sections. Furthermore, foreign solutions are not tailored for the specific building style and culture of Nepal. This leads to the need for know-how on low-cost and specific upgrading methods of traditional structures for traditional Nepalese brick masonry houses.

Since many of these buildings have insufficient connections and diaphragms, addressing their structural integrity might be a powerful improvement. Enhancing the box-action can optimize the load transfer and activate (favourable) in-plane structural elements in the response to seismic excitation. It is widely agreed upon that improving the box-action can significantly increase the overall seismic performance, making such methods cost-effective. In principle, retrofit should be an integral exercise, in which all possible failure modes of the building are addressed, including strengthening of the walls. If one would highly strengthen the connections, but the wall would fail instead, the retrofit would be purposeless. As the walls of the vernacular houses are thick, this research project will focus on potential retrofit of the buildings connections and diaphragms.

![Figure 1.2](source: own image)
1.2 Objectives

This thesis research deals with the retrofit of connections and diaphragms of the traditional masonry buildings typical to the Kathmandu region in Nepal. It will focus specifically on low-cost and low-tech measures to improve the seismic performance. The following classification is made:

1. Connections wall-to-diaphragm
2. Connections between perpendicular walls
3. Connections between diaphragm elements

The objectives of this research are formulated as follows:
1. To investigate and evaluate low-cost options for seismic retrofit of connections and diaphragms of traditional Newari brick masonry houses.
2. To perform numerical studies to study the effect on their global seismic performance.

Due to the specific context of Nepal, besides enhancing seismic performance, the solutions must satisfy certain requirements. The main focus will be on the following design and construction criteria:

<table>
<thead>
<tr>
<th>Design criteria</th>
<th>Construction criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural efficiency</td>
<td>Impact on architecture</td>
</tr>
<tr>
<td>Impact on architecture</td>
<td>Durability</td>
</tr>
<tr>
<td>Durability</td>
<td>Construct-ability</td>
</tr>
<tr>
<td>Construct-ability</td>
<td>Cost-effectiveness</td>
</tr>
<tr>
<td>Cost-effectiveness</td>
<td>Material availability</td>
</tr>
</tbody>
</table>

Above objectives lead to the following research question:

“How to improve the connections and diaphragms between building elements of traditional brick masonry buildings in Nepal for future earthquakes?”

This question is approached with the following sub questions:
1. Which retrofit techniques are affordable, accessible and integrable?
2. What is the effect of the retrofit measures on the global seismic performance?

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3 Paragraph 6.2 contains a more elaborated outline of the design- and construction criteria for the retrofit.
1.3 Outline and scope of thesis

As a starting point and problem definition, a descriptive analysis is conducted of the traditional brick masonry houses considering the typical style, building elements and possible failure mechanisms in Chapter 2. The continuation of the research will consist of four main parts: a literature review, an evaluation of possible retrofit measures, numerical analyses and implementation proposals.

PART I: Literature review
The literature study addresses the existing research on the seismic behaviour and modelling of low-strength masonry structures. It starts with low-strength masonry material in Chapter 3, proceeds with connections and diaphragms in Chapter 4 and brings prior topics together in Chapter 5, addressing masonry structures without box-action.

PART II: Connections and diaphragms upgrading
A set of specific retrofit criteria is made in Chapter 6 based on literature and interviews with structural engineers. On the basis of these criteria, a selection of existing retrofit measures on connections and diaphragms is evaluated qualitatively by means of a “traffic light” scoring system in Chapter 7. The overview of the rating results is given in Appendix B. Two retrofit combinations are proposed which have the best overall score. For the proposals a distinction is made between remote areas (local or transportable materials, low-tech) and urban/rural areas (making use of concrete and reinforcement bars).

PART III Numerical analyses
Numerical analyses are performed to study the effect of the implementation of the retrofit proposals on the global seismic performance of a structure. The analysis strategy is laid out in Chapter 8. For the analyses of the masonry walls, the smeared crack material model is applied, newly developed by Arup, using LS-DYNA software. First, in Chapter 9, numerical models are compared to the results of an experimental shaking table testing campaign by Sathiparan (2016) on a scaled masonry model. The motivation is to evaluate the capacity of the numerical modelling strategy to capture the seismic behaviour of low-strength masonry. The most important objectives of the comparison are reproduction of overall damage propagation and collapse mechanisms.

Then in Chapter 10 a simple one storey box-model with comparable dimensions and characteristics to one bay of a vernacular house is modelled to simulate the effect of the proposed retrofit measures. The fixed-base models are loaded sequentially in X- and in Y-direction with El Centro motions which are scaled to 0,5g. Firstly the unstrengthened situation is compared to a model with a fully connected, stiff diaphragm. Secondly the retrofit methods proposed in PART II are modelled into the case study. Results are reviewed on crack patterns and displacement of masonry walls.

PART IV Integration case study
In Chapter 11, the results of the prior numerical study on a simple masonry box structure are linked to a full vernacular house, suggesting an overall retrofit approach. Proposals are done on how to integrate the retrofit measures into the architecture of a typical (3-4 storey) traditional brick masonry house.

The main findings and conclusions are given in Chapter 12, followed by recommendations for further research.
1.4 Limitations

The following aspects pose limitations on this research:

PART II – Exploration of connection and diaphragm upgrades

Chapter 7 and 8:
- The evaluation of retrofits measures with a three-color grading system is inherently subjective. It is nevertheless a useful tool to compare the performance of retrofit measures on multiple criteria. These criteria are set-up with the input of several structural engineers from Arup. The applicability and feasibility of retrofits for the Nepalese situation is evaluated with feedback by the Nepali born seismic engineer Jitendra Bothara.
- The dimensions of structural members and spacing of components of the retrofit proposals are not engineered, but based on suggestions from literature.

PART III – Numerical analyses

Chapter 9:
- The newly developed shell masonry material model should be further calibrated to tests on specific low-strength masonry sub assemblages to better approach the nonlinear behavior of the material. However, these tests on low-strength brick masonry were not found in literature.
- The walls of the shake table test are single wythe, whereas Nepalese walls are generally multiple wythes.
- The force-displacement input used for the modelling of nailed connections is calibrated to diaphragm tests from literature by Arup. There has been no specific validation in this research for the modelling of connections and diaphragms.

Chapter 10:
- For simplicity, a numerical model represents a one story, one bay of a vernacular house. The structural response and failure mechanisms associated to a full vernacular house will be different.
- The numerical models are fixed-base, no soil structure interaction.
- The masonry material model is not able to account for delamination of wall leaves, although a significant share of the low-strength masonry walls is expected to have little interlocking.
- Several timber and steel elements are modelled elastically as these are not expected to be governing in failure. Furthermore, the nonlinearly modelled nails do not have a descending branch leading to failure.
- Earthquake motions are applied in one horizontal direction at a time for clarity in interpreting the results. In reality, the earthquake will have vibration components in 3 main directions.
- The analyses are performed with only one earthquake record, although the results can be highly dependent on the frequency content of the chosen record. The well-known El Centro record is chosen with comparable frequency content to the Nepalese code spectrum.
- The typical earthquake record has a duration of >30seconds. Due to time-constraints incremental dynamic analysis was not possible. The models are all analyzed for one earthquake motion at a fixed PGA of 0.5g. The models do not experience collapse at this ground motion, and no information about collapse is therefore obtained.
2 DESCRIPTION OF NEWARI HOUSES

The traditional Newari settlements are found in the Kathmandu valley, a bowl-shaped and densely inhabited urban area lying in the midst of Nepal’s mountain ranges. The built environment in the Kathmandu Valley can be broadly classified into: urban cores (centre of Kathmandu), urban outskirts, urban historical settlements, rural villages and remote villages. Villages are called remote if they lie deep into the mountainous areas, and are only accessible by foot. A large number of dwellings ranging from the Malla period (200 years old) are found in the three main cities Kathmandu, Lalitpur and Bhaktapur (D’Ayala & Bajracharya, 2003; Pradhan, 2000) and in several large towns or fortresses such as Kirtipur and Bungamati. Houses with comparable construction method, but with less storeys and more modest decoration of the façade are found in the rural and remote areas of Kathmandu Valley.

2.1 Building style

The Newari building style has remained relatively unchanged from the 16th century until the second half of 20th century, when Nepal started to open up to western influences (Beckh, 2006). This continuity has resulted in the construction of many buildings with a relatively similar building configuration and material use (brick, wood and tiles) over the ages. Early on, the buildings used to be only one storey high adobe structures. Houses in remote and rural areas are still one to two storeys high. As urbanization occurred and the price of land increased, houses were being built with multiple stories (Nienhuys, 2003). Newari houses in urban areas are mostly 3 or 4 stories high, where some have modern 4th floor additions. The houses with multiple storeys were associated with wealth. Special façades with hairline joints between burnt bricks and elaborately decorated timber elements contributed to this projection of riches and craftsmanship. The buildings typically have a rectangular floor plan with a depth of 6 m and a length varying from 3 to 10 m (D’Ayala & Bajracharya, 2003). Storey heights are relatively low, lying between 1,8-2,4 m, with an average of 2,2 m. The gravity and lateral load resisting system are formed by thick unreinforced brick masonry walls. The 6 meter depth plan is divided by an internal spinal wall (duāga). On higher floors the spinal wall can be replaced by a row of timber columns (dalān), providing a more open space, better ventilation and reducing building mass towards the top.

Housing typologies

The houses are arranged in several typologies. The basic house in rural and remote villages is rectangular and detached. In more dense urban environments, houses are clustered in arrays or around courtyards. It is common for adjacent houses to have matching storey heights, creating a horizontal alignment and minimizing the effect of pounding. It is observed that houses in array or courtyard configuration have a better seismic performance than the detached houses (Malla, 2015). Malla suggests that the adjacent buildings might have some stabilizing effect, enlarging their redundancy.

Figure 2.1 Houses in array in urban areas vs. detached houses in rural areas with comparable construction method, only two storeys in height (Source: own pictures)
Figure 2.2 Map of the Kathmandu valley showing the historical Newari settlements (Source: Korn, 2007)

Figure 2.3 Bird view sketch of a part of historical town Bungamati - housing typologies: (a) detached, (b) around a courtyard and (c) in array (Source: adapted from Jorgen Thomson)
Figure 2.4 Elevations and sections of a typical Newar house (Source: Survey by Danish architects in 1968)
Chapter 2 Description of Newari Houses

Figure 2.5 Detailed section of a typical Newar house (Source: Survey by Danish architects in 1968)

(a) Timber roof structure with timber peg connections
(b) Window and door frame with transversal elements connecting inner and outer frame

Figure 2.6 Typical features of good construction practice (Source: Survey by Danish architects in 1968)
2.2 Building elements

The following section will give a brief overview of the structural elements of a Newari house. Typical dimensions and configurations are referenced from Maskey (2012).

Foundation

The traditional masonry buildings have shallow strip foundation systems, usually made with rubble- or field stone footing. The foundation is placed at depth of 150 cm or more below ground level. The configuration is stepped, starting with a maximum width of 75 cm at the bottom and continued by brickwork into the walls of the superstructure.

Walls

The walls are built out of adobe (sun-dried) or fired bricks, bound by mud-mortar. The walls thickness generally ranges from 40 to 75 cm and tends to be reduced upwards, to decrease the mass towards the top. The large thickness provides the walls with more lateral resistance and stability against overturning, but it also induces higher inertia loads. Due to the very weak bonding strength of the low-strength mud mortar (Table 3.1), rapid cracking can occur in the masonry walls. The mud-mortar is significantly less strong than the masonry unit, therefore cracking mostly occurs in the mortar joints. As the mortar starts to yield under relatively small loads, energy can be dissipated through cracking and frictional sliding. In this way, the weak mud-mortar provides some flexibility to the masonry walls, allowing some of the earthquake impact to be absorbed. The weight of the masonry walls, increases the friction force ($\mu \cdot \sigma_d$) and contributes to its resistance. Newer mortars contain more cement which provides higher strength, but also a higher tendency to brittle cracking and a lower deformation capacity. For example; the young’s modulus of cement mortar is high (8-10 GPa) which indicates little elastic deformation (Nienhuys, 2003) where dried clay has significant lower value of around 0.5 GPa.

The thick walls are built up out of multiple wall wythes. In poor construction practices, the walls lack through-stones (header) and interlocking between the wythes. The exterior wythes can be made with properly dimensioned brick units, whereas the middle layer is filled-up with rubble bricks. These walls generally have a decreased integrity and high vulnerability to delamination. For some buildings the outer leaf is built with fired bricks (ma apa), whereas the inner leaf is constructed with unburned adobe bricks (kaci apa). The greater stiffness of the exterior wall leaf can cause bulging out failure mechanism. For prosperous inhabitants and houses with a certain status, a special wedge shape brick (daci apa) was used to create a hairline joint facades (Figure 2.7b). Besides the preferred aesthetic, these bricks prevent moisture intrusion to the timber interior elements and degrading of the mud-mortar.

Timber wedged connections

The applied seismic strategy was to make the houses flexible, as making buildings strong enough was not feasible with the mud-bound masonry. Key elements for this strategy are wedged timber pegs (Chukul). These connections are loose-fitting to allow some movement and absorb impact of the earthquakes. In good construction practice, these wedges are applied at multiple locations throughout the structure where timber elements meet such as at the cross ties of window frames, and at beam to wall (Maharatta, n.d.).

![Figure 2.7 Typical mud-based brick masonry walls in heritage settlements](image)

(a) Masonry walls in Bungamati and Kirtipur, Nepal  
(b) Delamination of multi-wythe wall

Figure 2.7 Typical mud-based brick masonry walls in heritage settlements  (Source a: own picture, source b: Nienhuys, 2003)
These connections provide the houses with high flexibility, allowing them to sway heavily during earthquakes (Rana, 1934).

Window size and frames
The walls are characterized by a limited amount and size of window- and door openings, which is favourable as there is less weakening of the wall as a diaphragm. The symmetrical arrangement of the wall openings contributes to the continuity of load transfer. The openings have double wooden lintels above and beneath, which extend up to 200 mm into the surrounding masonry wall. The lintels are connected by vertical and transversal elements – creating a box-like frame. The windows have no glass infill but are fully or partially covered by a timber grill of lattice work.

Dalan structure
The brick load-bearing façade is sometimes replaced at the ground floor by a timber colonnade called *Dalan*, to create an open threshold for shops or workplaces. A double row of columns is used with dimensions ranging from 100x100 mm to 150x150 m, spaced 100 to 150 cm. The pinned timber columns stand on stone piers, to protect them from splashing rain. A pin on top of the column pierces through a hole in the (decorated) wooden column head, and the beam above, creating a pinned connection. The open façade with respect to the relatively closed upper brick facades poses the risk of a soft storey mechanism. A *Dalan* structure is also applied in the upper stories of the building. The heavy spinal wall is replaced to decrease the weight of the building towards the top.

Floor system
The floor is supported by timber joists (*dhalin*) spanning 3m, running from external wall to the spinal wall. Timber floor joist dimensions are usually 50 to 100 x100 mm, closely spaced at a distance of 200 to 300 mm. The joists vary from rectangular timber beams of Nepalese hard wood, to crude tree or bamboo trunks. The joists are covered by tils, wooden planks or pieces of bark, without having any nailed connection. On top, a layer of 10cm clay is applied for insulation and cultural purposes. Modern 4th floor additions have reinforced concrete cast in-situ slabs as floor system, adding excessive mass to the building.

Stairs
The wooden stairs consist of two inclined wooden beams. Due to the limited storey heights the next floor is reached by means of one flight, resulting in narrow and steep stairs.

Roof structure
The roofs structure is built with a system of timber rafters spanning from the façade to the interior *Dalan* columns. Transversal beams lie on these rafters and support the roof joists. The roof is covered with a thick insulating mud layer which ensures the waterproofness and protection of interior wooden elements. The mud is covered with overlapping ceramic tiles (*jhinjati*). The roofs structure has an overhang of 1 meter, strutted by carved wooden brackets (called *tunalas*) which are placed at an angle of 45° and 2 m spacing. The roof overhang and the occasional skirt roof at lower stories helps protecting the wall from rain erosion and the timber elements from rotting (Nienhuys, 2003).

Figure 2.8 Typical features of vernacular house (Source a: Korn, 2007, b: Bonpace & Sestini, 2003, c: Nienhuys, 2003)
2.3 Overview of the current situation of building connections

In this section an overview is given of the current situation of the connections and floors, which are of main influence to the box-action the masonry structures:

1. Connections wall-to-diaphragm
2. Connections between perpendicular walls
3. Stiffness of diaphragms

2.3.1 Wall-to-diaphragm

Wall-to-diaphragm connections include wall-to-floor and wall-to-roof. As earthquakes are multidirectional, the connections must be able to withstand both in-plane shear force parallel to the wall and out-of-plane forces perpendicular to the wall.

Wall-to-floor

The roof- and floor joists span from the façades to the internal (spinal) wall. The timber floor joists are simply embedded into the masonry where they are generally not held or anchored at the end. The floor joists may be supported by half of the wall width (b) or pierce fully through the wall (a), making the joist ends show in the façade. A wall-plate introduces the loads uniformly to the wall. The embedded joist provides resistance to shear forces in-plane of the wall. For pull-out loads the current joist-pocket connection relies only on friction and some mortar cohesion between the joist and masonry.

In good construction practice shear locks in the form of timber wedges (Chukul) are provided on the interior and occasionally on the exterior of the façade preventing dislodgement of floors and walls.

For the wall-to-floor connection parallel to the joists (d) there seems to be no structural connection at all. The lack of structural connection of the floor to the side walls has caused a lot of local failure mechanisms such as out-of-plane toppling of the walls (Figure 2.13 and Figure 2.14).

![Figure 2.9 Section of several types of masonry joist pocket connections](Source: own image)
Wall-to-roof
The primary roof beams are simply supported by the masonry wall (Figure 2.10). This connection (perpendicular-to-joists) relies purely on friction and some cohesion of the mud-mortar. The beams are pinned into the columns of the Dulan colonnade. The transversal beams are pinned to the primary roof beams with wedged timber elements. The roof joists are simply supported by the transversal beams and spinal beam in the middle, and by diagonal struts at the overhang. For the attachment of the roof to the ‘gable’ wall (parallel-to-joists) there is no structural connection at all. Out-of-plane toppling of gable walls is a frequently observed failure mechanisms in these buildings.

2.3.2 Perpendicular walls
Connections between perpendicular walls include the corner junction of the façade with the side walls, and the T-junction of the spinal- and side walls. The connections are often simply butt-jointed with no or little interlocking of the perpendicular bricks.

2.3.3 Stiffness of diaphragms
The floor joists are covered by tils, wooden planks or pieces of bark mostly without nailed connections, forming the subsurface for a heavy 10cm thick layer of clay ballast (Figure 2.11). The floors basically account for no diaphragm action at all and the mud provides excessive mass to the structure.

Figure 2.10 Cross sections of the wall-to-roof connections3D (Source: own sketches)
Figure 2.11 Damaged buildings showing wall-to-floor(joist) connection in Bungamati (Source: own picture)

Figure 2.12 Old brick masonry houses in Bungamati. The floor joist ends are showing in the façade (Source: own picture)

Figure 2.13 Damaged house in Sankhu. Out-of-plane failure of the side walls. (Source: own picture)
Figure 2.14 Collapsed side façade of old brick masonry house in Sankhu (Source: own image)

Figure 2.15 Timber wedges as shear locks (a) interior wedges (b) exterior wedges (Source: own image)

Figure 2.16 Wall-to-roof connection old brick masonry house in Bungamati (Source: own picture)
2.4 Building failure modes

The main deficiencies of the houses are the lack of structural integrity and the inherent weakness of the building materials. The following building failure mechanisms are identified after past earthquakes:

a) **Rapid cracking of the low-strength masonry;**

b) **Delamination of multi-wythe masonry wall;**

c) **Heavy earthen topping of floors and roofs causing high inertia loads. As the floors are brought into movement, the perimeter walls can bulge outwards;**

d) **Out-of-plane toppling of walls:**
   - In cases where there are flexible diaphragms and no proper connections between walls, in-plane walls deliver little contribution to the performance of the out-of-plane walls (Karantoni, 1992a).
   - Where in-plane walls are far apart and there are too little supporting cross walls, the middle part of the out-of-plane wall can topple over (Bothara et al., 2004). The failure of one wall also enlarges the vulnerability of the perpendicular wall as the supporting ‘flange’ falls away.

e) **Parapet and gable wall toppling,** since these walls behave as cantilevers due to a lack of fixation at the top (Ramesh Guragain et al., 2009).

f) **Collapse of non-structural elements** such as cornices, chimneys;

g) **Various types of wall failures under in-plane loading:**
   In case of sufficient box-action, in-plane failure modes are more likely to occur. Several observed modes are:
   - Flexural failure in masonry piers.
   - Shear failure (diagonal cracking ‘X-cracks’) usually originating at the corners between openings.
   - Sliding of piers at the base (Bothara et al., 2004)

h) **Collapse of floor and roof:** the failure of vertical load-bearing elements (such as walls) or the failure of connections can cause the collapse of the floors and in this way the total collapse of a structure.

i) **Corner separation of walls:** earthquake excitation can cause vertical cracks to develop at junctions (NSET, 1998). When separation of the in-plane and out-of-plane walls has occurred, the isolated walls can experience rapid failure (Bruneau, 1994);

j) **Poor anchorage,** weak connections between building elements. Once the anchorage has failed and the building loses structure integrity, cause rapid failure;

k) **Soft storey mechanism in case of timber colonnade in façade (Dalan).** The Dalan frame creates a large opening in the façade. The reduction in stiffness of the wall makes the building vulnerable to in-plane failure and lateral overturning.

The following failure modes are rarer:

l) **Compressive failure of piers;**

m) **Foundation failure:** mostly the superstructure fails before the foundation is overstressed (NSET 1998)
a) Severe cracking of walls  

b) Delamination  
c) Bulging of floors  

d) Out-of-plane toppling of walls  
e) Gable toppling  
f) Non-structural elements  

g) In-plane failure  
h) Collapse of floor / roof  
i) Corner separation  

j) Poor anchorage  
k) Dalan frame: potential soft storey  

Figure 2.17 Failure mechanisms of traditional brick masonry buildings in Bungamati and Sankhu (Source: own pictures)
PART I: LITERATURE STUDY

An overview is made of the existing research on the seismic behaviour and modelling of low-strength masonry structures without box-action. This part commences with low-strength masonry (Chapter 0). Low-strength masonry structures have several types of connections between building elements and generally quite flexible floor diaphragms (Chapter 4). Together, the masonry walls, connections and diaphragms behave as a total structure (Chapter 5). In each chapter both seismic behaviour and modelling methods of the addressed topic are outlined. This part serves as the basis of the chosen modelling assumptions for the numerical analyses of Part III.
3 LOW-STRENGTH MASONRY

This chapter addresses the seismic behaviour and the modelling methods masonry, focussing specifically on low-strength unreinforced masonry which is typical for developing countries. This type of masonry is characterized by very weak mortar bonds and relatively thick walls. Firstly, an introduction is given on the masonry components and construction methods. Secondly, a review is done on typical material properties and the seismic behaviour of the masonry. Lastly, several methods are discussed to model masonry material.

3.1 Masonry classification

Masonry components
Masonry is a composite construction material, built-up out of masonry units and mortar. Masonry units are building blocks which can be made of a variety of materials. In developing countries such as Nepal, masonry is mostly built with solid adobe (sundried) bricks, fired clay bricks or natural stone. Stone masonry units can be dimensioned to regular shapes or left in original form (rubble stone). The units are bound together by mortar, which is a mixture of aggregates, binder and water. Mortar can be made of clay (mud) and based on lime/cement or various other compositions. The properties of mortar are highly dependent on the ratio of its components. Masonry which is bound by mud-mortar is generally referred to as low-strength masonry.

Masonry construction methods
There are several ways to construct with masonry material:

- **Unreinforced masonry** is purely built up with brick units and mortar.
- **For reinforced masonry** the walls are strengthened with reinforcing steel and concrete.
- **Confined masonry** employs a system of confining the load-bearing masonry walls with reinforced concrete members.
- **Reinforced concrete frame** construction is different from confined masonry, as the RC-frame is the main load-bearing structure and the masonry acts as infill walls that influence the stiffness of the structure.

Low-strength masonry is mostly built unreinforced.

3.2 Masonry properties

Masonry is a non-homogeneous material, as it is built-up of two constitutive elements: blocks and mortar. The structural behaviour of masonry is complex and depends highly on the composite action between its components. With knowledge on the mechanical properties of the individual elements, it is hard to estimate the composite behaviour of the masonry. This means that for verification of the structural capacity the mechanical properties of the composite masonry/mortar action should be used, which are derived from experimental tests on standard masonry wall samples (Tomaževič, 1999). In the following section first the general composite properties of masonry are listed, after which several tests are reviewed on masonry components with properties representative or comparable to low-strength masonry.
3.2.1 General masonry properties

The following mechanical properties are relevant to determine the structural behaviour of masonry, as specified in EC6:

- The compressive strength, \( f_s \)
- The shear strength, \( f_v \)
- The flexural strength, \( f_u \)
- The stress-strain relationship, \( \sigma - \varepsilon \).

Additional properties needed for numerical verification are according to Tomaževič (1999):

- The modulus of elasticity, \( E \)
- Shear bond strength, \( f_s \)
- Shear modulus, \( G \)
- Flexural bond strength \( f_t \)
- The friction coefficient, \( \mu \).

The compressive strength of masonry is derived by applying an increasing vertical pressure load at a uniform rate on a masonry wallet until failure occurs (Figure 3.1a). From the compressive test, the relationship between the stress and the strain in the masonry can be derived (Figure 3.1b). This relationship illustrates the highly non-linear behaviour of the masonry. Initially the masonry exhibits linear-elastic behaviour, until cracking starts to occur.

![Masonry wall sample for compression test](a) Typical stress-strain curves of a masonry unit, prism and mortar (Source: Paulay & Priestley, 1992)

The modulus of elasticity \( E \) is a measure of resistance to being deformed elastically. The Young’s modulus can be derived from the experimental stress-strain relationship, up till 1/3 of the maximum vertical load which is applied during a compression test (Tomaževič, 1999).

The shear strength of masonry is defined as the resistance to shear forces. It is a combination of the initial shear strength (the shear bond strength) without any normal load, increased by an additional frictional resistance defined as the frictional coefficient \( \mu \) multiplied by the compressive stress on the shearing member \( (\tau = c + \mu \sigma) \). The initial shear strength \( f_{v0} \) represents the cohesion of the masonry bond \( f_s \) and is measured with a triplet test. In this test, the middle brick of a rotated stack of three bricks is pushed downwards. Shear stress should be developed in the contact interfaces.

The shear modulus \( G \) describes the response (lateral deformation) of a material to shear stress. The parameter is hard to define for masonry by means of experimental testing, as other mechanisms are active in the test set-up. The G-modulus for isotropic materials is described by: \( G = E/(2(1+\nu)) \). Assuming a Poisson’s ratio of 0.25 delivers a G-modulus of approximately 40% of the elasticity modulus. Bosiljkov et. al. (2005) state that masonry stiffness differs highly from soft lime mortars to...
hard brittle mortars. As found through experimental values, the shear modulus may differ from 6 to 25% of the measured Young's modulus of the masonry (Tomažević, 1999).

*The diagonal tensile strength* is an alternative way of describing shear strength of masonry. The value is determined by means of a diagonal compression test, representing a situation of combined lateral and compressive stress in the masonry.

For out-of-plan loads the strength of masonry in bending is the governing parameter. Anisotropic masonry material has different flexural strengths in the two orthogonal directions. The flexural strengths $f_{sk1}$ and $f_{sk2}$ correspond respectively to the bending parallel to the bed joints and perpendicular to the bed joints.

The *flexural bond strength* $f_t$ describes the tensile cohesion of the brick-mortar bond and can be evaluated by performing a bond wrench test on a stack-bonded prism.

### 3.2.2 Typical properties of low-strength masonry properties

Considering the complex behaviour and wide variety of masonry it is recommended to perform mechanical properties tests for the specific masonry which is evaluated. But, there are limited experimental tests done to retrieve the material properties of low-strength brick masonry in developing countries. An overview is given of several relevant studies on masonry with comparable low mortar capacity, and some field tests on mud-bound masonry structures. Results of the studies will be summarized in Table 3.1.

**Laboratory tests**

Parajuli (2012) has performed laboratory tests on thick-walled *mud-bonded fired brick* masonry wallets. The test samples were composed from old bricks from the Malla period which were collected from dismantled buildings in Nepal. Three kinds of tests were performed: compression, shear, combined shear and compression loading tests. A compressive strength is found of 1,82 MPa, diagonal shear strength of 0,126 MPa, a Young's modulus of 789 MPa and a Poisson's ratio of 0.25. Parajuli indicates that there were no lab investigations on the mechanical properties mud-bonded brick masonry reported up to their study.

![Figure 3.2 Set-up of masonry component tests](Source: Parajuli 2012)

Sathiparan (2005) has performed tests on the mechanical properties of scaled masonry wallets for both *fired clay bricks and sun-dried adobe units* representative for developing countries. The tests were part of a larger experimental campaign to test the masonry retrofit of PP-band meshing. Bricks were made with Japanese materials, which were actually too strong compared to low-strength masonry. However, it is argued that the overall strength of these masonry walls is governed by the bonding strength of the mortar joints, and a very weak mortar mix was applied with the proportions cement: sand: lime in 1: 2.8: 8.5 to replicate the low-strength masonry of developing countries. Compressive strength of masonry bricks were found 4,36 MPa and 21,78 MPa for adobe and fired bricks respectively. Shear bond strengths of 0,0056 and 0,0075 MPa (5,6 kPa and 7,5 kPa) for fired bricks, and a flexural bond strengths of similar range of 0,0051 and 0,0055 MPa (5,1 and 5,5 kPa).

---

4 Part of the Disaster Risk Management for the Historic City of Patan, Nepal – Final Report of the Kathmandu Research Project
Since the tests of Sathiparan were done on scaled masonry wallets, the effect of scaling on masonry properties is reviewed on the basis of the literature study of Petry and Beyer (2012). For reduced size bricks, fired bricks are experienced stronger than the prototype after burning (Egermann, R., Cook, D. & Anzani, 1991). The high compressive strength of the fired bricks measured by Sathiparan (2005) of 21,78 MPa might be explained by this phenomenon. The deviation does not arise for adobe (unburnt bricks). The properties of scaled mortar joints are influenced by the sucking tendency of bricks. For thin mortar joints there is relatively more suction of the water, increasing the mortar strength in comparison to thicker joints with the same mortar mix (Drysdale & Hamid, 2008). Shear bond strength and flexural bond strength are assumed to be scale-independent failure mechanisms.

**Field tests**

Kiyono and Kalantari (2005) have performed in-situ tests on the remains of both mud-bonded sun-dried brick walls as fired brick walls in Iran to study the bonding strength of mortar joints in shear and tensile directions. They found very weak bonding strength, for shear respectively 2,9 and 9,7 kPa for sun-dried and baked bricks. And for the flexural bond strength they found relatively similar values for sun-dried and baked bricks: 4,6 and 5,1 kPa. The researchers suggest these low bonding strength values as the main cause of collapse of many low-strength masonry structures.

Parajuli and Kiyono (2015) performed similar pull-out in-situ tests as Kiyono and Kalantari on mud-bonded stone-masonry in Nepal. It was found that the shear bond strength of the masonry was 1,137 kPa and the friction coefficient 0,6.

Guragain et al. (2012) have performed in-situ tests on the shear resistance of masonry walls for Nepal and Bangladesh, including the initial shear strength and the factor associated with frictional resistance. The tests are performed for masonry bonded by cement mortar and mud-mortar, which are both assumed to be weak with respect to the brick units. A shear wall resistance of 0,106 MPa is found for mud-bounded structures.
Summary of experimental research

There is little experimental research on material properties typical for low-strength masonry. The results of the tests which were found in literature are combined in a table. As comparison, experimental values are added of tests done in Pavia for bricks representative for the North of the Netherlands.

The experimental campaign of Parajuli (2012) on fired bricks dismantled from traditional buildings, bound by mud-mortar is the only lab campaign found on brick masonry bound by mud-mortar. For the application of numerical modelling, this campaign misses experimental data on the shear- and flexural bond strength of the mortar joints, which are crucial for simulating the seismic behaviour of masonry. Other studies are therefore reviewed from Parajuli and Kiyono (2015), Guragain et al. (2012), Sathiparan et al. (2005, 2006), which show very low cohesion, in a reasonably similar range. Shear bond strengths lie in the range of 0.001 to 0.01 MPa, where flexural bond strength is measured of around 0.0050 MPa. Friction coefficients all lie around 0.6. When comparing the values representative for low-strength mud-based masonry to experimental values for Dutch cement-mortar masonry, it is clear that the young's modulus and compressive strength are lower. Values for the mortar bond strength are significantly lower, in the range below 5% of the values for Dutch masonry.

<table>
<thead>
<tr>
<th>Material property</th>
<th>Mortar type</th>
<th>Dutch masonry</th>
<th>Sathiparan</th>
<th>Kiyono and Kalantari</th>
<th>Parajuli</th>
<th>Parajuli</th>
<th>Guragain</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density [kg/m³]</td>
<td>Brick</td>
<td>1768</td>
<td>1850</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Mortar</td>
<td>1705</td>
<td>1650</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Masonry</td>
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<td>1768</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Young's Modulus [MPa]</td>
<td>Brick</td>
<td>3874</td>
<td>1680</td>
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<tr>
<td></td>
<td>Mortar</td>
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<td></td>
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<tr>
<td></td>
<td>Masonry</td>
<td>5760</td>
<td>1160</td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>Poisson's ratio</td>
<td>Brick</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Mortar</td>
<td>0.25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Masonry</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength [MPa]</td>
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<td>45.81</td>
<td>11.03</td>
<td>0.05</td>
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<td></td>
<td></td>
</tr>
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<td></td>
<td>Mortar</td>
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<td>1.58</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Masonry</td>
<td>11.32</td>
<td>1.82</td>
<td>4.36</td>
<td>21.78</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wall shear [MPa]</td>
<td>V</td>
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<td></td>
<td></td>
<td>0.723***</td>
<td>0.106***</td>
<td></td>
</tr>
<tr>
<td>Diagonal shear [MPa]</td>
<td>Masonry</td>
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<td>0.047</td>
<td>0.077</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear bond strength [MPa]</td>
<td>Mortar</td>
<td>0.15</td>
<td>0.0056</td>
<td>0.0075</td>
<td>0.0029</td>
<td>0.0097</td>
<td>0.001137</td>
</tr>
<tr>
<td>Flexural bond strength [MPa]</td>
<td>Mortar</td>
<td>0.158</td>
<td>0.0051</td>
<td>0.0055</td>
<td>0.0046</td>
<td>0.0051</td>
<td></td>
</tr>
<tr>
<td>Friction coefficient</td>
<td>µ</td>
<td>Brick-mortar</td>
<td>0.7</td>
<td>0.62</td>
<td>0.62</td>
<td>0.54</td>
<td>0.6</td>
</tr>
<tr>
<td>Shear modulus [MPa]</td>
<td>G</td>
<td>Masonry</td>
<td>318</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* These values are representative for Dutch masonry
** There is no information on the overburden of the wall, so nothing can be said on initial shear strength
*** The type of brick was not explicitly mentioned, but it is stated that the test is done for masonry with relatively strong units and weak mortar joints.

Table 3.1 Overview of mechanical properties derived for low-strength masonry

---

5 Experimental campaign on masonry representative of the Groningen building stock by a collaboration of TU Delft, EUCentre, Arup and Pavia
3.3  Seismic behaviour of masonry

Earthquake vibrations are multi-directional. Failure modes of walls are distinguished as in-plane and out-of-plane failure modes. If the building is tied well together, in-plane modes are mostly governing.

![Diagram](image)

Figure 3.5 Typical crack patterns for a masonry building (Source Tomaževič, 1999)

3.3.1  Out-of-plane modes

Out-of-plane failure is a first mode, local response mechanism. It is a brittle mode, as portions of the wall can suddenly lose their equilibrium and cause partial or total collapse of the structure. Out-of-plane modes are caused by a lack of sufficient connections, long unsupported walls or horizontal thrust forces, which are frequent features in low-strength masonry buildings. Typical out-of-plane failure modes are depicted in Figure 3.6 and summarized as:

a) Partial or complete overturning (of the perimeter walls such as gable walls and facades),
b) Composed overturning (taking a part of the wall),
c) Corner failure,
d) Vertical flexural failure (causing horizontal cracks),
e) Horizontal flexural failure (causing vertical cracks),
f) Gable wall failure,
   -or a mixture of above.

![Diagrams](image)

Figure 3.6 Out-of-plane failure modes(Source: adapted from Milano et al., 2009) .
3.3.2 In-plane modes

In-plane failure is characterized as the second mode response, activated if the building develops a global response. There are several types of in-plane mechanisms. Typical in-plane modes are:

- Bed joint sliding horizontal (a1)
- Bed joint sliding diagonal (a2)
- Diagonal tension (b)
- Toe crushing (c1)
- Rocking (c2)

![In-plane failure mechanisms](image)

Figure 3.7 In-plane failure mechanisms according to ASCE41-13

The ratio of the shear- versus compressive stress in a pier influences which in-plane failure mechanism is likely to occur, as depicted in Figure 3.8. This ratio contains aspects such as the normal load (overburden), eccentricity, slenderness ratio and friction.

Horizontal bed joint sliding (a) occurs when there is low vertical load and little friction between the horizontal layers. A crack will form at the interface and the horizontal layers can start to slide. Due to the weak shear cohesion of mortar, bed joint sliding is one of the prevalent failure mechanisms. Diagonal tension failure (b) occurs when the resultant of horizontal and tensile stresses exceed the tensile strength of masonry. As the mortar of low-strength masonry is weaker than the brick, the cracks will mostly propagate along the mortar joints, not through the bricks. When there is a high compressive stress, failure may occur due to toe-crushing and rocking (c). For small buildings, rectangular buildings, or buildings with no slender piers compressive failures are rare.

![Shear-stress ratio and occurring failure mechanisms](image)

Figure 3.8 Shear-stress ratio and occurring failure mechanisms (Source unknown)
3.4 Modelling of masonry

The modelling of existing masonry buildings is particularly complicated, due to the following difficulties, according to Parajuli (2012b):

- The inherent properties of the material, being anisotropic and highly nonlinear.
- The composite material has several varying parameters such as the joint dimension, material properties of the constituents, the arrangement of bricks, and quality of workmanship.
- For existing structures there is generally a lack of data on geometry, inner part of structural elements, variation of mechanical properties due to age and workmanship, degradation of fatigue and strength and accumulated damage.

Extra difficulties arise for low-strength masonry:

- As the mud mortar is very weak, cracking starts at low levels of acceleration. Therefore the material already displays highly nonlinear behaviour in early stages of loading.
- The walls of low-strength masonry are generally very thick, displaying various configurations of bricks. The wall is built-up out of several wall-wythes, which can have different properties and capacities.
- Due to the weak coherency between wall-wythes the walls are vulnerable to delamination, which is an extra failure mode to take into account.

For finite element analysis, standard material models can simulate the behaviour of structural materials in the linear-elastic range. Due to the complexity of modelling the nonlinear material, there is no widely accepted computational model for masonry. However, in ongoing research specialized material models are developed for the modelling of non-linear behaviour such as crack propagation and collapse. Ideally material models should be able to capture the following principle local failure mechanisms: unit tensile cracking, joint tensile cracking, joint slipping, unit-joint diagonal cracking and unit-joint crushing. However, as crack patterns are mostly in the mortar joint and compressive failure is rarely seen, unit-cracking and unit joint crushing are not governing.

A classification of modelling strategies can be made with respect to the connectivity between elements. For masonry models a distinction is made by element connectivity into FEM models with continuous elements, DEM models with discontinuous elements, and the AEM models which apply discrete blocks connected by multiple springs or dashpots. FEM models tend to apply meshed shell surfaces, whereas DEM and AEM apply solid elements.

The masonry models can also be classified means of scale, dividing the methods into micro-, meso- and macro modelling. Micro-models are more time-consuming for modelling and computation, but give more local and accurate results. Macro-models are more descriptive and provide a prediction of global behaviour, but are a favourable trade-off for computational and modelling speed.
3.4.1 FEM, DEM and AEM method

- **Finite element modelling (FEM)** is a technique which is used to find ‘approximate solutions to boundary value problems for partial differential equations’ (Wikipedia). In FEM analysis a problem is broken down into small parts, called finite elements which are described by simple equations. These parts are then assembled as continuum elements to approximate the total solution of the whole system. Masonry as a continuum component can also be called macro-modelling, in which the masonry properties are smeared out over a mesh surface.

- **Discrete Element Method (DEM)**, also referred to as the Distinct Element Method is used essentially for modelling the behaviour of a large amount of small particles. The method is widely applied for engineering problems concerning granular or discontinuous materials (Lemos, 2007). Masonry is modelled as a series of assembled (non-deformable) solid blocks, for which the contact forces and displacements at the interfaces are evaluated via the equations of motion (Parajuli, 2009), which are a set of constitutive laws. The DEM method can be considered as a form of simplified micro or meso-modelling.

- **Applied Element Method (AEM)** combines features of both FEM and DEM. It is similar to FEM as it also divides a problem into a series of smaller elements. The method differs in the way the elements are joined. Masonry units are modelled as rigid block like elements, as in DEM, which are connected at their contact surfaces by a series of nonlinear springs (normal and shear springs) (Guragain et al. 2012), or as a multiple springs and dashpots (Furukawa, A., Kiyono, J., Toki, 2011). AEM is capable of simulating structural collapse through all stages of loading with reasonable accuracy, allowing it to capture crack initiation and propagation with satisfying computational speed, according to Meguro and Tagel-Din (1997).

![Figure 3.10 Element connectivity of the AEM method versus the FEM method](Source: Guragain et al., 2012)
3.4.2 **Macro and micro modelling**

Masonry modelling strategies can be classified on the basis of scale with the terms micro-modelling, simplified micro- or meso-modelling and macro-modelling (Lourenco, 2002; Rots et al., 1997). The micro-model is most detailed model. The macro-model is 10-100 times larger of scale. The simplified micro-modelling operates on a scale in between the micro- and macro models and is referred to as simplified micro-modelling.

- **Micro-modelling:** The components of masonry (units and mortar) are modelled explicitly as continuum elements and the unit-mortar interface by discontinuous elements. The non-linear behaviour is concentrated in the mortar interface. This strategy models the masonry brick-by-brick. Building the model is time-consuming; therefore this technique is mostly used for component simulation.

- **Simplified micro-modelling:** The masonry units are modelled as continuum elements. The mortar joint is reduced to zero-volume, whereas the brick units are expanded to secure the geometry of the masonry. The mortar and unit-mortar interface are modelled with averaged properties, as potential crack joints.

- **Component (macro-element) models:** The elements of masonry (units, mortar and unit-mortar interface) are modelled as one continuum homogeneous element, in which the aspects of masonry material behaviour are smeared out over the material model. This method is more descriptive and represents a prediction of global material behaviour. As the model is easier to construct than prior mentioned methods, it is a compromise between accuracy and computational speed. Various methods have been developed to link the macro material model to the actual material behaviour; on the basis of specific experimental tests, two-step, RVE, Multi-scale. Frequently named in literature is the smeared crack model (Mersch, 2015), which originates form the modelling of concrete.

- **Equivalent frame modelling:** The masonry is modelled as a blocked frame. The walls are subdivided into pier- and spandrel masonry panels (modelled as 2-noded nonlinear beams) connected by rigid areas. Earthquake damage is rarely seen to occur in these rigid areas. These kinds of models are applied in the program Tremuri for Push-over analyses.

![Masonry modelling strategies](image)

Figure 3.11 Masonry modelling strategies (Source: adapted from Mersch, 2015 and Lagomarsino et al., 2013)
Chapter 4 Connections and Diaphragms

4 CONNECTIONS AND DIAPHRAGMS

It is widely recognized that connections and diaphragm stiffness influence the overall seismic response of a building. Understanding their behaviour is therefore of high importance for the numerical modelling of existing structures, but also for design of retrofit measures (Lin & LaFave, 2012). However, limited research is done on the behaviour of connections between structural components (Carles, 2012; Lin & LaFave, 2012; Moreira et al. 2012).

In this chapter firstly a classification is made of different connection types, prevalent in masonry buildings with timber diaphragms. Then a review is given of several studies on the structural behaviour and modelling of the several connection types.

4.1 Existing connections and improvements
Connections are considered in this chapter which influence the box-action of unreinforced masonry structures. A distinction is made between connections in the as-built connection, and devices which can be used to improve the connectivity. The following classification is made in Table 4.1 Classification of existing and improved connections Table 4.1.

<table>
<thead>
<tr>
<th>Classification of connections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connection type</td>
</tr>
<tr>
<td>Wall-to-diaphragm</td>
</tr>
<tr>
<td>Perpendicular walls</td>
</tr>
<tr>
<td>Diaphragm elements</td>
</tr>
<tr>
<td>Belt</td>
</tr>
<tr>
<td>Wall-wythes</td>
</tr>
</tbody>
</table>

*) Only in good construction practices

Table 4.1 Classification of existing and improved connections
4.2 Behaviour and modelling of connections

Suitable modelling is needed of the behavior of connections, including their ductile or dissipative action, in order to capture the impact of connections on the overall seismic behavior of a structure more accurately.

Plumier (1994) makes a distinction between explicit and global representation of the modelling of connections. The connection is modelled explicitly with a three dimensional finite-element model, meshed into a large number of elements. This method can provide accurate behaviour, but it is very time-consuming. A global representation applies an idealization of the connection zone by (a series of nonlinear) springs (and dashpots) with a prescribed force-displacement behaviour. Global representation seems appropriate in combination with macro-element masonry models, as both aim for a decent representation of the global behaviour of the structure (Cross & Jones, 1994). For the global representation of connections in numerical models, experimental data on the structural behavior of connections in the form of force-displacement curves or approximate stiffness values are essential input. An overview is made of several studies done on the behaviour and modelling of connections and diaphragms:

<table>
<thead>
<tr>
<th>Model</th>
<th>Type</th>
<th>Model elements</th>
<th>Load</th>
<th>Researchers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Numerical</td>
<td>building</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Joist-pocket with steel strap</td>
<td>Experimental</td>
<td>-</td>
<td>Monotonic, Quasi-static, and Cyclic loading</td>
<td>Lin &amp; Lafave (2012)</td>
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<td></td>
<td>Numerical</td>
<td>SDOF</td>
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<tr>
<td>Joist-pocket with anchor</td>
<td>Experimental</td>
<td>-</td>
<td>Monotonic loading</td>
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<tr>
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<td>-</td>
<td>Time history</td>
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<td>SDOF &amp; MDOF</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nailed connection</td>
<td>Experimental</td>
<td>-</td>
<td>Time history</td>
<td>Bracci &amp; Hueste</td>
</tr>
<tr>
<td></td>
<td>Numerical</td>
<td>MDOF</td>
<td>Pushover, Time history</td>
<td>Arup (2011)</td>
</tr>
<tr>
<td>Nailed connection</td>
<td>Experimental</td>
<td>-</td>
<td>Shake table test</td>
<td>GOM (1998)</td>
</tr>
<tr>
<td>Wall component with reinforced mortar overlay</td>
<td>Experimental</td>
<td>-</td>
<td>Monotonic loading</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Numerical</td>
<td>AEM</td>
<td>Monotonic loading</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.2 Overview of research on connection behaviour and modelling
4.2.1 Wall-to-floor connections

For existing buildings with timber diaphragms the floor joists meet the wall by means of a joist-pocket connection. During earthquake excitations, the beam can slide out of the wall and eventually fall of its support. This connection can be improved with straps and anchors.

Joist-pocket connections – Cross and Jones (1993)

The behaviour of joist-pocket connections of timber beams into masonry walls relies basically on friction and impact (Cross & Jones, 1993b). The friction between the beam and masonry pocket provides resistance to pull-out or push through forces and energy dissipation via frictional sliding. As vertical earthquake motions can cause uplift of the beams, the decrease of normal force lowers the frictional resistance. To approach the behaviour of a joist-pocket connection the well-known Coulomb model for friction is used, containing a frictional force proportional to the normal force:

\[ F_f \leq \mu_s \cdot N; \quad \text{for } v = 0 \]  \hspace{1cm} (4.1)

\[ F_f = -\mu_k \cdot N \cdot \text{sgn } v; \quad \text{for } v \neq 0 \]  \hspace{1cm} (4.2)

Impact occurs when the end of the beam hits the end of the joist pocket while crushing the mortar. The coefficient of restitution describes the kinetic energy which remains after the rebound of an object and the amount that is lost by deformation. The value \( e \) ranges from \( e = 0 \) (perfectly plastic) to \( e = 1 \) perfectly elastic. The reality will lie somewhere in between.

\[ n \cdot v_s = -e \cdot n \cdot v_A \]  \hspace{1cm} (4.3)

Cross and Jones (1994) have developed a modelling method that accounts for both friction and impact behaviour. The frictional component is set-up as follows: if the beam moves a distance of \( > x_{\text{edge}} \) towards the outer edge of the wall it will fall off and if it moves a distance larger than \( x_{\text{wall}} \) it will contact the endface of the wall pocket. This behaviour is represented by a discrete beam element, which initially has zero length, but deforms when loading is applied to the structure. The impact component is an equivalent spring-damper which is only active for the duration of the impact, and captures the energy dissipation by means of the coefficient of restitution.

Several numerical analyses were performed by Cross and Jones to test the numerical models. These analyses have shown that the applied connection model gives a good approximation of the frictional behaviour of the joist-pocket connection. The model also manages to capture some energy loss due to the impact of the beam end and wall.
Wall-to-joist nailed strap – Lin & Lafave (2012)

Lin & Lafave (2012) studied the behavior of joist masonry-pocket connections with and without steel strap connection. The main mechanisms of force transfer are frictional resistance between wood joist and brick and the mechanical connection of the nails. The specimens are loaded in the longitudinal direction of the joist, by means of static monotonic loading, quasi-static cyclic loading and dynamic cyclic loading.

Under monotonic loading, the nails and friction connection show initially elastic states. The connection softens as the nail slips and starts prying into joist. A yield plateau is reached until the first nail shears off. The connection is then moderately stable until the second nail shears off and only friction capacity is left. For quasi-static loading failure modes are seen of nails shearing off but also additional modes of nails pulling-out. For dynamic loading failure occurs only due to nails shearing off.

A simple numerical model is proposed for the connection by Lin and Lafave, including nails, friction and contact. The nails are modelled by parallel springs. The second nail is prescribed with a more ductile displacement curve to simulate the one-by-one shearing of the nails. An average friction is proposed, combined with a contact model which describes the wood joist moving into the masonry pocket and crushing the mortar. The numerical results are found to well estimate the overall force-displacement behavior of the wall-diaphragm connection.
Wall-to-floor plate anchor connection - Moreira et al. (2012)

Moreira et al. (2012) studied the behavior of improved joist-pocket connections, representative for old masonry buildings in Lisbon (‘Gaioleiro’ and late ‘Pombalino’). The connection relies on friction, adhesion and shear resistance of the masonry for load transfer. Several specimens were strengthened means of a steel angle and tie-rod, anchoring the beam to the exterior of the wall with an anchor plate. During the test, the joist is monotonically loaded with a pull-out force.

Failure modes were predicted: pull-out of the masonry cone, masonry crushing under the anchor plate, failure of the steel tie rod, failure of the connection of the steel anchor and the joist, ripping of timber. For each specimen, parameters as the bolt parameter, the fixation of the tie rod, and the wall thickness were varied to evaluate the influence of these parameters on the occurring failure mode. Results show the alteration in test set-up indeed influences the failure mechanisms per specimen. The force-displacement graphs show that pulling out of timber nails results in yield plateau (WF.40.U.1A), where stripping of threads of the bolt is a very brittle failure mechanism (WF.40.A.1B).

(a) Expected failure modes  (b) Strengthening solution  (c) Connection wall specimen

Figure 4.5 Wall to floor connection (Source: Moreira et. al 2012)

<table>
<thead>
<tr>
<th>WF.40.U.1A</th>
<th>WF.40.A.1A</th>
<th>WF.40.A.1B</th>
<th>WF.40.A.1C</th>
<th>WF.60.A.1A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic model, unstrengthened</td>
<td>4 Φ6 bolts were used to connect angle to beam</td>
<td>Φ6 mm bolts replaced by Φ10 mm bolts</td>
<td>Tie rod was tightened with 3 nuts</td>
<td>Wall thickness was increased to 60 cm</td>
</tr>
</tbody>
</table>

Beam sliding out of wall, pull-out of nails  Crushing of timber and shearing of bolts  Stripping of threads of steel tie-rod  Pull-out cone of masonry wall  Crushing of wall-plate

<table>
<thead>
<tr>
<th>WF.40.U.1A</th>
<th>WF.40.A.1A</th>
<th>WF.40.A.1B</th>
<th>WF.40.A.1C</th>
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</tr>
</tbody>
</table>

Table 4.3 Failure modes and force-displacement graphs of the test specimens (Source: Moreira et. al 2012)

4.2.2 Perpendicular walls

The bricks of perpendicular walls can be interlocking by means of corner stones, or simply butt-jointed. The corner connectivity can be improved by means of stitches, anchors or belts tying the walls together. No specific research on the behaviour or modelling of wall-to-wall connection is found. For brick-by-brick models the interlocking or lack thereof can be modelled explicitly. When using a smeared crack model with mesh surfaces, the material model inherently assumes an interlocking pattern of bricks. So when the perpendicular mesh surfaces meet at coinciding nodes, an interlocking connection is implied.
4.2.3 Diaphragm behaviour

The components of timber diaphragms are joists, sheathing panels or planks and nailed connections. Timber floors vary from being very flexible to acting as a rigid diaphragm. Modelling gains complexity for flexible or semi-flexible diaphragms. It is widely agreed upon that the nonlinear behaviour of timber diaphragms is dominated by its nailed connections. The idealization of this connection type is therefore important.

Lourenço (2011) has studied literature on flexible diaphragms by Brignola et al. (2008), Yi (2004), Paquette and Bruneau (2000) and Tomaževič et al. (1996). The most important conclusions are that flexible diaphragms; act as spring supports for the walls, have large deformation capacity, behave highly non-linear and hysteretic under high accelerations and that strengthening only in-plane stiffness is not necessarily enough to improve global building response.

Nailed sheeting connection – Judd and Fonseca (2007)

Judd and Fonseca (2007) have studied hysteresis modelling for nailed connections of timber diaphragms. The cyclic response of a nailed sheathing diaphragm is characterized by hysteresis loops. Initially the force-displacement relationship is linear, as the components wood fibres, sheathing and nails deform but remain in the elastic range. Nonlinearity occurs when the wood and sheathing fibres start to crush and/or the nails start to yield. At reverse of loading, the connection loses stiffness as the nail can move through the gap which is created, until it hits the wood again on the other side.

The flexible behaviour of the diaphragm can be modelled by an equivalent SDOF model or explicitly by a Finite Element model. The equivalent SDOF system approach uses a spring and dashpot system. The model can be calibrated to the force-displacement relationship from experimental data, or extrapolated from a scaled model.

For the explicit model, modelling the nails as elastoplastic, embedded into the nonlinear sheathing and timber joints requires knowledge on all nonlinear mechanical properties of the components (foschi 2000, He, Lam and Foschi 2001). The alternative is to model the timber joists and sheathing panels as linear elastic elements and the nailed connections as nonlinear springs. The response of the connection is modelled implicitly by defining force-displacement behaviour. To evaluate this strategy, Judd and Fonseca have done numerical analyses on both finite element model and SDOF model to test the influence of several alternative hysteresis models. Results show that for both analyses methods strength and stiffness degradation and pinching behaviour parameters are important in the modelling of hysteresis to achieve modelling accuracy.

![Figure 4.6 Hysteresis modelling of nailed diaphragm connections](Source: Judd & Fonseca, 2007)
Retrofits for flexible diaphragms – Bracci & Hueste (2004)

Bracci & Hueste (2004) have studied the in-plane behavior of existing and retrofitted timber diaphragms (typical for the USA) in the pre-1950ies. Several retrofit methods were subjected to a quasi-static reverse cyclic loading and compared to the performance of an existing diaphragm. Applied retrofits included: steel strapping with enhanced shear connectors, steel truss, unblocked and blocked wood panel overlays. All retrofits are found to increase in-plane stiffness. The addition of a steel truss delivered the most significant increase in strength and stiffness of the model. The measured in-plane lateral response is used to generate force-displacement curves. The existing diaphragms are very flexible to in-plane loading. The high deformation allowance is proposed as the reason for the lack of damage, as there was no failure up to a displacement of 76 mm.

Nailed connections for Dhaji Dewari construction – Arup 2011

Arup (2011) has performed numerical analyses on the seismic performance of the traditional Dhaji Dewari construction found in the western Himalayas. The basic structural system is a timber braced frame with stone masonry infill. The stone masonry blocks are modelled as incompressible solid elements with contact surfaces between infill pieces to represent friction behaviour. The flexible mud-mortar has not been modelled explicitly. The corrugated roof sheets are attached to the timber roof joists with nails. The nails are represented by nonlinear discrete beams with prescribed inelastic force-displacement behaviour for shear and pull-out forces.

The numerical model of a wall is compared to component tests by UET Peshawar. Wall layout and amount of nails of numerical and experimental were not identical. Comparison shows that the numerical model can broadly reproduce the behaviour of the timber braced wall. Hysteresis loop demonstrate that the displacement matches well, whereas the strength of the physical model is initially stronger by 50%. Pushover and nonlinear time history analyses are performed on the full building. Comparison is done between models with nailed connections between horizontal and vertical frame members and compared with the model without nailed connections. The addition of the nailed connections clearly has beneficial effect on the out-of-plane behaviour of the walls, as they provide extra confinement to the infill pieces.
4.2.4 Belts

‘Seismic belts’, in the form of horizontal bands or bandages, span around the perimeter of a structure, tying the walls together.

**Horizontal bandage – GOM (1998)**

A shake table test is performed by GOM (1998) in response to the Maharashtra earthquake in 1993 in India, as referenced by (Bohara & Brzev, 2011). The aim of the test is to study the effect of horizontal bandage on the seismic behaviour of unreinforced rubble-stone masonry buildings. The retrofitted model with bandage at roof level did not collapse after 12 test runs as the walls are tied together by the seismic belt. The unreinforced model is clearly on the verge of collapse, where it has already experienced various out-of-plane failure mechanisms.

![Figure 4.9 Evaluation of the bandage retrofit method through shake table testing](Source: GOM 1998).

4.2.5 Wall leafs

The behaviour and modelling of masonry walls are discussed in chapter 3. Walls with multiple wall wythes can be constructed with interlocking bricks or header bricks, or with no interlocking and infill of rubble masonry between the outer leaves. Both for walls with little coherency and walls with insufficient strength, through anchors, overlays or jacketing strategies with ties connecting inner and outer leaf can be used to increase in-plane and out-of-plane strength.

For brick-by-brick models the interlocking within a wall can be modelled explicitly. When using a smeared crack model with meshed surfaces, a composite part can be used to define several masonry leaves with deviating strength. However, delamination and debonding of masonry leaves and connections from inner to outer leaf are not possible to model. Ongoing research is focused on mesh-type retrofitting of masonry walls, evaluating multiple (low-cost) mesh-type retrofits with PP-band, bamboo, grocery bag and geomesh.

**PP-band mesh overlay – Sathiparan (2005)**

Sathiparan et. al (2005) have studied the potential beneficial effects of retrofitting method with PP-band mesh. Both in-plane and out-of-plane experimental static tests were done on masonry wallets of burned brick and low-strength mortar. Orthogonal bands were connected at intersecting nodes. The PP-band material was tested and found to have a very large deformation capacity. Diagonal compression tests show that the PP-band provides a lot of residual strength for in-plane loading after the first cracking of the masonry. The load bearing capacity increases by 2.5 and the deformation capacity by 45. For the out-of-plane bending test, the influence of the mesh also becomes notable after the cracking of masonry. Capacity is increased twice and deformation capacity by 60.
Macuabag et al. (2012) simulated a diagonal compression test on a retrofitted masonry component. The AEM technique is used to simulate component tests of the retrofitted masonry. Brick units are modelled as incompressible blocks, connected by multiple springs representing the mortar interface. The numerical model provided failure mechanisms comparable to experimental, although failure loads were found to be lower than actual tests. It is stated that more calibration could lead to better results. A more extensive numerical study is done by Mayorca and Meguro (2001). Their simulation of mesh retrofit shows very good resemblance in peak-strength and force-displacement behaviour.
5 MASONRY HOUSES WITHOUT BOX-ACTION

In the prior chapters the behaviour and modelling of low-strength masonry, connections and diaphragms are discussed. This section deals with the seismic behaviour and modelling of a masonry structure. First the behaviour of masonry buildings without box-action is addressed. The lack of box-action has an impact on the applicability of the seismic calculation methods, which is discussed secondly.

5.1 Behaviour of masonry buildings without box-action

Masonry walls and timber diaphragms act as the main loadbearing systems for lateral and vertical loads. In case of box-action, the walls and diaphragms are tied well together, so the building can develop a global response to seismic excitations.

Structures with box-action

In case of proper connections and stiffened diaphragms, lateral loads are distributed proportionally to the lateral stiffness of the walls. A larger share of the load is then transferred to the in-plane lateral load resisting walls. In-plane mechanisms are governing in this case and out-of-plane modes and failure mechanisms are limited. In-plane mechanisms contain shear, sliding-shear, diagonal or rocking/crushing failure mechanisms. These modes are preferred above out-of-plane mechanisms as they are less brittle (Carles, 2012), and allow more energy dissipation before collapse. With box-action, the building can produce a more global, three-dimensional structural response. This allows to utilize more fully the potential seismic resistance and energy dissipation capacity of the structure (Tomaževič, 1999). Poor masonry quality and significant percentage of wall openings are deficiencies to keep in mind for structures with an in-plane response.

Flexible diaphragm and insufficient connections

Structures without box-action are not tied well together. No proper load path is ensured from the diaphragms to the vertical load-resisting structural elements. In this case, the seismic load of is distributed per tributary area. The governing wall modes are out-of-plane modes. Flexible diaphragms and insufficient connections to the walls can lead to excessive displacements at the floor levels and out-of-plane mechanisms of the walls such as the overturning of the perimeter walls. Out-of-plane failure mechanisms are commonly described as a primary failure modes and are unfavorable because of their brittle nature (Brignola et al., 2009). The local failure of walls can cause the collapse of the floors, and in this way the total collapse of a structure.

Figure 5.1 Behaviour of masonry building during seismic excitation. (Source: Tomaževič, 1999)

Flexible floors, without ties
(b) Flexible floors and tied walls
(c) Rigid floors and tied walls

Flexible diaphragm and insufficient connections

Structures without box-action are not tied well together. No proper load path is ensured from the diaphragms to the vertical load-resisting structural elements. In this case, the seismic load of is distributed per tributary area. The governing wall modes are out-of-plane modes. Flexible diaphragms and insufficient connections to the walls can lead to excessive displacements at the floor levels and out-of-plane mechanisms of the walls such as the overturning of the perimeter walls. Out-of-plane failure mechanisms are commonly described as a primary failure modes and are unfavorable because of their brittle nature (Brignola et al., 2009). The local failure of walls can cause the collapse of the floors, and in this way the total collapse of a structure.

6 (Tomaževič et al., 1996), (Lin & LaFave, 2012), (Lourenço et al., 2011), (Sathiparan, 2016)
The amount of diaphragm stiffness needed differs per building. The objective is not to make the diaphragm fully rigid but to make it stiffer. For buildings with thick walls, the weight is concentrated in the masonry. Transferring all the out-of-plane loads to in-plane via the diaphragm is then an extensive task. The stiffness required for such a diaphragm would add a significant addition of mass, increasing inertia forces. Leaving a building able to move in multiple modes has potential benefit: perhaps the multiple modes can partially cancel out each other’s movement. Whereas for boxed buildings, the energy is concentrated in a limited amount of modes. This consideration requires more research.

**Enhancing connections without diaphragm action**

The floors of vernacular houses in Nepal have no diaphragm action. Floor beams generally have no nailed connections to the planks or topping. Connecting the wall to diaphragm then basically means connecting the wall to one floor beam, which doesn’t seem very effective.

**Stiffening floor without appropriate connections**

Invasive retrofits have been executed in the past to improve the in-plane stiffness of diaphragms. Timber floors have been replaced by new reinforced concrete (rigid) diaphragms. However, connections to the wall were not always executed sufficiently. Increasing only the in-plane stiffness of the diaphragm is not enough to improve global response (Lourenço et al., 2011). It can even lead to unfavorable mechanisms (Brignola et al., 2009). Lemme et al. (2008) have observed overly stiff floors leading to the expulsion of building corners. The distribution of stress in a diaphragm can result in tension and in compression diagonals. A concentration of outward thrust forces can push the corners outwards.

**Experimental tests**

There are limited experimental tests on masonry structures without box-action. Brignola (2009) mentions shake table tests of (Bothara, 2004), (Cohen et al., 2004), (Paquette et al., 2004). He concludes from these studies that masonry buildings with flexible diaphragms do not behave as SDOF systems, but as a 2DOF or more, where the additional degree of freedom is associated with the in-plane flexibility of the timber diaphragms.

Sathiparan (2016) has studied the effect of roof diaphragms on unreinforced masonry structures subjected to dynamic loading. A scaled box-model, built with low-strength masonry, was tested with three different roof conditions on a shaking table test. Results show that the connection of wall-to-roof-diaphragm influences the type of failure modes. The model with roof connection demonstrates a more global response, whereas the model without roof shows local out-of-plane damage mechanisms. The model with a proper roof connection has a better seismic performance with respect to ‘shear resistance, lateral stiffness, maximum strength, yield displacement and ductility’ (Sathiparan, 2016).

![Figure 5.2 Experimental studies on the effect of roof diaphragm (Source: Sathiparan)](image-url)
5.2 Modelling of seismic behaviour of buildings without box-action

The effectiveness of a retrofit technique is tested by simulating seismic excitations on a structure with and without the implementation of retrofit. The effects of seismic loads are addressed by calculations, (computational) modelling or experimental testing, on full-size, scaled structures or components.

5.2.1 Aspects influencing the applicability of calculation methods

The characteristics of the structure and the associated seismic hazard influence the applicability of seismic calculation methods. Key aspects for low-strength masonry with flexible floors are:

- **Highly non-linear behaviour of the materials.** Nonlinear behaviour is when irreversible deformations occur in the materials (such as cracking or yielding). The weak mortar of low-strength masonry starts cracking at low levels of acceleration. Also flexible diaphragms show highly nonlinear behaviour when subjected to high accelerations.
- **High levels of ground motion** increase the structural damage, causing more nonlinear behaviour.
- **Large displacements** cause geometrical non-linearity, which concerns the influence of second order moments and change of stiffness as the geometry of the structure changes.
- **Diaphragm flexibility.** Due to the flexible diaphragm behaviour the structure can’t be discretized as an SDOF system, as the response can contain multiple dominant modes (Nakamura et. al, 2014).
- **Out-of-plane modes/ failure.** Without box-action a building is more prone to out-of-plane failure. The calculation method should be capable to sufficiently capture the out-of-plane modes.
- **Out-of-plane response and diaphragm flexibility** are important aspects to capture in seismic analysis. Research is under development on how to properly evaluate these effects.
- **Duration effects.** Earthquakes in Nepal are relatively long (30-50 seconds). The number of cycles with large amplitude contains a lot of seismic energy. The long duration causes strength and stiffness degradation of the structure as the shaking progresses. The degradation of the structure influences its dynamic response (Bommer et al., 2004; Oyarzovera & Chouw, 2008).

5.2.2 Main methods

The main calculation methods for modelling the seismic behaviour of a structure are: the lateral force analysis, the modal analysis, pushover analysis and the nonlinear time history analysis. The methods can be divided into linear or non-linear and quasi-static or dynamic.

Linear methods are not able to take into account physical or geometrical non-linearity, and quasi-static methods are not able to evaluate the dynamic response of the structure. Linear methods are applicable for lower values of acceleration, where material behaviour is expected to remain in the elastic range and non-linear behaviour of materials and structure is not significantly influencing the results. Uncertainty on elastic methods grows for inelastic demands; therefore a higher level of conservatism needs to be maintained in the load assumptions. Static methods lose their applicability when higher response modes and the seismic duration effect are of significant influence.
Lateral force analysis
The lateral force analysis is a linear, static method. The response of the building is read from a design spectrum for a certain estimated natural frequency. The maximum expected lateral force at the base is calculated (1) with the response and weight of the building (5.1). This force is distributed over the height of the buildings and its elements (2), either according to either the first mode shape or linearly over the height. The capacity of the elements is checked in the elastic domain (3).

\[ F_b = S_d(T_1) \cdot m \cdot \lambda \]  

(5.1)

\[ F_i = F_b \cdot \frac{s_i \cdot m_i}{\sum s_i \cdot m_i} \]  

(5.2)

Modal analysis
The modal analysis is a linear, dynamic method. The structure is modelled as a multi-degree-of-freedom (MDOF) system. Equivalent lateral loads are determined per modal response shape (1). The total response of the structure is computed by superposition of all lateral loads per mode. The force is then distributed over the height of the building according to either the first modal shape or linearly over the height (2). Then, the elements are checked in the elastic domain. The method can take into account structural ductility indirectly by applying global force-reduction factors. The response is calculated in the time-domain; therefore dynamic factors such as phase information can be included.

Pushover analysis
The pushover analysis is a nonlinear, quasi-static analysis method. The structure is subjected to a monotonic loading (push) (1) which proceeds through the elastic and in-elastic domain until the ultimate capacity is reached (3). The structural response is registered by a capacity curve which plots the displacement of a control node at the center of gravity on the highest floor (2) to the base shear force. The capacity curve is compared to the demand curve (4).
Nonlinear time history analysis
The nonlinear time history analysis is a nonlinear, dynamic method. A detailed structural model is subjected to a record of motions (1). During the analysis, the internal forces and displacements are calculated at the nodes by solving the associated equation of motion for every time step.

\[ m\ddot{u}(t) + c\dot{u}(t) + ku(t) = -m\ddot{g}(t) \]  \hspace{1cm} (5.3)

Dynamic analyses can be performed by means of an explicit or implicit solver. As for nonlinear analysis the geometry and materials can change, each increment in time requires an update of the stiffness matrix. Implicit solvers use both current state at \( t \) and later stage \( t + \Delta t \) of the system, and therefore require iterations for each step to reach equilibrium of internal forces. The stiffness matrix needs to be inverted per time step, which can take a lot of time for complex models.

Explicit solvers calculate the later state of a system \( t + \Delta t \) only by means of the current state. However, the time step must be small enough to attain accurate results. The time step is governed by the Courant condition, stating the time step must be lower than the time a stress-wave takes to propagate through the length of one element. For implicit analysis the time step can be several orders of magnitude larger to obtain accurate results. However, the method can have some difficulties converging when collapse occurs, whereas the explicit solvers deal with nonlinearity and collapse with relative ease (LS-DYNA Support, 2016; Wikipedia, 2016).

\[ Y(t + \Delta t) = F(Y(t)) \]  \hspace{1cm} (5.4)

\[ \dot{G}(Y(t), Y(t + \Delta t) = 0 \rightarrow Y(t + \Delta t) \]  \hspace{1cm} (5.5)

Figure 5.3 Schemes of the main calculation methods for seismic analysis (Source credits: Arup)

Additional research on the applicability of the push over method
The pushover analysis is an attractive method for seismic analysis as it captures nonlinear structural behaviour, while it maintains the simplicity of static analyses. Several studies are reviewed to assess the applicability for structures with flexible diaphragms.

The Pushover is developed for diaphragms remaining rigid in-plane, as it essentially represents an SDOF system (Nakamura et al. 2014). Several adaptations are proposed in literature to account for limitations in the procedure. Lourenco (2011) argues that these methods should be used with caution for masonry buildings without box-behaviour as more research is necessary on the topic.

Galasco et al. (2006) used a simplified equivalent frame model and TREMURI software to evaluate the pushover analysis method. The model is built-up with macro elements representing piers and lintels, connected by rigid parts. Pushover tests are performed in-plane on separated wall systems. Numerical results are in good agreement to the experimental tests, with respect to force-displacement curve and damage progression. It is acknowledged that this method has several limitations for buildings with flexible floors: flexural behaviour of the diaphragms and out-of-plane response of the walls are not taken into account. A tool is presented to overcome the limitations of the pushover analysis for existing masonry buildings, this tool is not validated to nonlinear dynamic analysis or experimental tests.
Nakamura et al. (2014) studied the applicability of nonlinear static methods for structures with flexible diaphragms with simple 2DOF systems. Two stick-models represent the in-plane walls, including associated mass, stiffness and damping. A coupling spring is used to represent the flexible diaphragm. Several alternatives of pushover methods are employed (N2, MPA and ACSM procedures) and compared to time history analyses. Results of the pushover became less reliable when the diaphragm increased in flexibility. Underestimations were made in the calculation of ductility. The shortcomings were alleged to the assumption of first mode response; the inability of the control node to represent the movement of the whole structure as it has no uniform diaphragm behavior.

Lourenco et al. (2011) have compared the results of (scaled) shake table tests and non-linear dynamic analysis to the results of pushover tests. A four storey “Gaioleiros” building (unreinforced masonry with no box-action) was chosen as case study. Results indicate that traditional, adaptive or modal pushover analyses did not capture correctly all the mechanisms observed in the non-linear dynamic analysis or experimental observations. Cycle and rigid block behavior (rocking) and the out-of-plane behavior were not captured.
The pros and cons of the main calculation methods are outlined for the seismic analysis of URM masonry buildings with flexible floors in Table 5.1.

<table>
<thead>
<tr>
<th>Method</th>
<th>Pros</th>
<th>Cons</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral force method</td>
<td>Relatively easy and fast estimation</td>
<td>Applicable only to regular and low-rise structures dominated by first mode</td>
</tr>
<tr>
<td>Adopted by most building codes</td>
<td>Assumption of rigid diaphragm (SDOF)</td>
<td></td>
</tr>
<tr>
<td>Pier-by-pier checks</td>
<td>No account for contribution of higher modes</td>
<td></td>
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<tr>
<td>Little input needed for calculation</td>
<td>No dynamic or seismic duration effects</td>
<td></td>
</tr>
<tr>
<td>Account for irregularity by torsional modification factors</td>
<td>No direct account for non-linear material behaviour and ductility</td>
<td></td>
</tr>
<tr>
<td>Account for structural ductility by applying force-reduction factors</td>
<td>Application of conservative factors (applicable to many buildings) may lead to inappropriate retrofit design</td>
<td>Lumping of loads in the floors is not representative for thick walled structures</td>
</tr>
</tbody>
</table>

| Modal analysis               | MDOF model provides contribution of higher modes                      | No direct account for non-linear material behaviour and ductility |
|                             | Applicable for more complex geometries, containing discontinuities in form and stiffness, or flexible floors | Lumping of loads in the floors not representative for thick walled structures |
|                             | Account for structural ductility by applying certain force-reduction factors | Application of conservative factors (applicable to many buildings) may lead to inappropriate retrofit design |
|                             | Dynamic effects are taken into account                                 | No seismic duration effects                                         |

| Pushover analysis           | Accounts for stiffness degradation                                     | Limitations to obtain accurate results for buildings with flexible diaphragms |
|                             | Analysis continues through the non-linear range                       | Requires expertise of user to predict structural hinge locations |
|                             | Insight into damage propagation                                        | Uses first mode, SDOF system                                         |
|                             | Accounts for second-order effects                                      | In-plane analysis only                                               |
|                             |                                                                        | No dynamic or duration effects                                        |

| Nonlinear time history      | Account for stiffness degradation                                      | Requires a lot of input information (F.E. geometry and material properties) necessary to benefit from accuracy of method |
|                             | Analysis continues through the non-linear range                       | Time consuming and difficult to interpret results                     |
|                             | Most extensive and accurate method, Least conservative method         | Results are sensitive to characteristics input ground motions (frequency content, duration, PGA). Therefore: representative motion, and favourably multiple ground motions needed |
|                             | Combination of in-plane and out-of-plane loads                        | Variability in masonry properties (even within a structure) to which extent of accuracy is possible? |
|                             | Accounts for the duration (amount of cycles) of seismic event          | Computationally expensive                                            |

Table 5.1 Pros and cons of calculation methods for masonry buildings without box-action
The aim of Part II is to assess suitable retrofit methods for the traditional brick-masonry houses with flexible timber diaphragms in Nepal. First several general seismic retrofit concepts are outlined, followed by specific design and construction requirements which are defined for the Nepalese circumstances in Chapter 6. Then, an overview is given of possible retrofits techniques for connections and diaphragms from literature in Chapter 7. These retrofits techniques are evaluated on the design- and construction criteria. On the basis of the outcomes, two suitable retrofit combinations are proposed for further numerical studies in Part III.
6 RETROFIT REQUIREMENTS

Seismic retrofit is a strategy to improve the seismic performance of a building by designing a set of modifications. Making a building stronger or stiffer is not always the most effective, economic or elegant solution. Strong elements require a lot of material and the extra weight will induce higher inertia forces. Furthermore, as the maximum load of a potential earthquake is never exactly known it can be risky to design for peak load. Alternative strategies to improve a building’s seismic performance are enhancing ductility, reducing the seismic demand or improving the distribution of forces. An overview is given of several general concepts and strategies.

6.1 Retrofit concepts

- **Reduce irregularity**: Irregularity in plan and elevation causes additional stress concentrations in certain parts of the building. Reducing irregularities lowers the seismic demand on these parts.
- **Improve box-action**: Improving connections, the stiffness of diaphragms and the continuity of load path will tie the building components together and provide a more favorable global structural response of the structure.
- **Design as one detail / integral solutions**: Seismic retrofit could include multiple interventions such as upgrade of wall-to-wall and wall-to-floor connections, improving floor- and roof diaphragms. Combining multiple upgrades into one integrated design can be very effective, such as: wall anchors embedded in concrete overlay or continuous connector beams which also function as diaphragm chords.
- **Remove and prevent excessive mass**: Removing or lowering mass causes a decrease of the inertia forces, and hence of the seismic demand.
- **Ensure adequate stability**: Increasing stability means improving the resistance to overturning at element and global level, by improving connections at element level, creating a broader base, adding buttresses, pre-stressed anchorage into the foundation or improving box-action. Increasing local stability of elements means lowering susceptibility to overturning in case of masonry walls.
- **Empower existing structural system**: If possible, it is favourable to let the existing structural system perform in a better way, instead of adding a new system such as steel or concrete frame caging. New structural systems could have compatibility problems with the existing, due to difference in pre and post-cracking stiffness. Activating the existing structure mostly requires less intervention, making this strategy suitable for both low-cost and historical interventions.
- **Enhance redundancy**: Redundancy of a building structure can be enhanced by providing alternative load paths. It helps to prevent progressive collapse when individual members fail.
- **Increase lateral strength and stiffness**: The aim of stiffening a structure is to reduce its lateral deformation, by adding lateral load resisting systems such as braced frames or shear walls.
- **Allow movement**: Allowing the building to sway and have some flexibility in order to absorb some of the earthquake energy and thereby reduce the seismic demand.
- **Energy dissipation**: Some kinetic energy of the earthquake can be absorbed, for example via frictional sliding or structural yielding of elements. In masonry building, this happens primarily due to cracking of the masonry.
- **Enhance ductility**: Ductility is defined as the ability to deform plastically, without experiencing an instant significant loss in load carrying capacity. Ductility prevents sudden collapse of members, and contributes to energy dissipation.
6.2 Design and construction and criteria

Criteria are defined to assess which seismic retrofits are suitable for improving the box-action of the vernacular houses in Nepal. Retrofit measures will be scored on these criteria to compare their performances with respect to the different categories. The six criteria are divided into two main categories “design” and “construction”:

<table>
<thead>
<tr>
<th>Design criteria</th>
<th>Construction criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural efficiency</td>
<td>Impact on architecture</td>
</tr>
</tbody>
</table>

The list is set-up on the basis of literature and interviews with several structural engineers from the firm Arup, who have had experience in design and implementation of wall-to-floor and floor stiffening and strengthening upgrades. Per criteria several key points are outlined:

6.2.1 Design criteria

Structural efficiency
- **Enhance seismic performance**: for existing buildings it is unfeasible to reach current code level requirements, but any well-designed mitigation strategy can yield better seismic performance and help reduce risk.
- **Technically robust and redundant**
- **Spread the load**: spreading the load over a larger surface ensures a more fluent and well-distributed introduction of the forces into the brittle masonry.
- **Avoid or account for eccentricity**: when the connection is eccentric with the force (diaphragm) it could cause additional moments. Eccentricity can be avoided by designing in-line of the strengthened diaphragm or symmetrical (with double anchors).
- **Avoid concentration of stiffness**: concentration of stiffness in one part of the structure can result in damage concentrations in another part (Tomažević, 1999).
- **Account for change of load path**: changed load distributions due to retrofit strategies can subject members to loading which they were not designed for. (Cross & Jones, 1993a)
- **Be aware of the role of overburden on the masonry wall**: it is generally favourable to remove excessive mass. However, the overburden provides more lateral load resistance to friction connections and mortar joint interfaces through frictional resistance (Arup, 2011; Cross & Jones, 1993b). It also has a pre-stressing effect on masonry walls, providing extra capacity to overcome some tensile forces.
- **Balance capacity of retrofit measures**: As the earthquake will ‘search for the weakest link’; there is no use making one element stronger than its counterparts in the load path.
- **Compatibility with the existing structure**: Additional lateral load resisting structures can be much stiffer or much more flexible than the masonry walls. Stiffer elements attract most of the loads, resulting in large dimensions of elements. Flexible frames will only be activated after the stiffer masonry has already cracked.

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**Impact on architecture**

The category architectural design evaluates which impact the retrofit has on the existing architecture. Interventions can affect the architecture in multiple ways: the functionality of the building, the external appearance of the façade and roof, internal appearance of the walls and diaphragms.

- **Interventions should be in harmony with the original architecture** (NPC, 2015b; Pradhan, 2000). Several strategies for more harmonious intervention are:
  - Design integrated measures supported by knowledge of the architectural features and historical identity of the building.
  - Design interventions with minimal intrusion or the possibility to cover up.
  - Preservation of historic material or “avoidance of new work that will not allow for future intervention” (Langenbach, 2000).

- **Tolerance for interventions dependent on status of building:** Since the housing stock does not have the same high cultural status of temples and palaces, there is more tolerance in modification of the exterior (Nienhuys, 2003) and more freedom in the use of modern materials for interventions in housing stock, such as plywood, steel, reinforced concrete and plastics. For monuments only traditional materials are considered, limiting possibilities to bricks, wood, clay and tiles.

**Durability**

The category building physics evaluates how measures contribute to maintaining the building physical properties of the house.

- **Prevent moisture infiltration:** one should take great care to prevent moisture intrusion (Langenbach, 2000; Nienhuys, 2003), as wall penetrations for improved connections can bring in water and leakage.
- Masonry has a high water absorption capacity; therefore timber elements in direct contact with the masonry, without ventilation possibility are susceptible to rotting.
- **Non-corrodible:** when exposed to the elements, materials should be resistant to corrosion and decay by insects or moisture. Materials such as galvanized metal, treated timber or plywood or reinforced concrete with sufficient concrete cover are favoured.
- **Low-maintenance:** an extended return period of maintenance and need of repair.

**6.2.2 Construction criteria**

**Constructability**

- **Constructability:** feasibility in terms of construction. Limiting damage to the existing building during the installation. For example, drilling into poor quality masonry. Especially into the top layer of masonry, such as the gable wall, can be challenging. Bricks are prone to falling out of the wall since this part has little overburden.
- **Simplicity in materials and methods:** use low-tech and light-weight materials, so no special experts or equipment (machines) are needed for the installation.
- **Adopt techniques that match local expertise:** design for local methods to limit the amount of training needed for implementation and the risk of incorrect execution. Adopt techniques that are known, but improve performance due to improved lay-out, materials, structural features (Parajuli, 2009).
- **Choose materials which allow for on-site flexibility:** materials and methods which can be adjusted on site to fit unexpected geometry or configuration: in this aspect cast-in-situ concrete or timber beams are favoured, with respect to steel members and laminated timber which have fixed dimensions.
- **Reduce the impact of the works on comfort:** in terms of noise, vibrations, functionality.

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9 Goodwin, Tonks and Ingham (2011) – paragraph on ‘heritage and conservation’. Langenbach (2000) has developed a specific set of guidelines for conservation and seismic retrofit for the Kathmandu Valley Preservation Trust.
- **Time-efficient solutions for large scale implementation**: for example plywood sheeting as floor-stiffening method is more time-efficient than nailing of single planks.

**Cost-effectiveness**
- **Low-cost**: in terms of materials, equipment, workmanship, transportation, implementation, maintenance. Cost estimation is challenging; since the costs are dependent on many variable factors as material price, scale of implementation, duration.

**Material availability**
The availability of materials depends highly on the accessibility of the building location. Urban and rural areas can be reached by motorized vehicle, whereas some remote areas are only reached via a long journey by foot. Availability is also influence by factors as deforestation, and political situations such as blockades.

(Local) availability of building materials is important for the feasibility of a retrofit technique.
- **Use of local materials**: local materials are favourable for locations with bad accessibility for motorized vehicles, and it will lower transportation costs. Furthermore, local workers will more probably know how to deal with these materials. As local materials are often also incorporated in the houses, they could possibly be easier to integrate into the existing buildings. Using regional materials also ensures independence of conflicts with neighbouring countries and delivers income for local manufactures. If certain materials could be imported on large scaled and are easy handle these should not be ruled out.

The issues regarding the main potential materials for retrofit are outlined:
- **Timber**: Prices are high due to a scarcity of the material. Commercially available timber comes mostly from the south of the country (the Terai). The sourcing of timber is restrained due to deforestation and forests in the mountains are hard to access. Softwood (pines) is more widespread, however this is not typically used for structures due to durability concerns, as there are little timber treatment plants. In remote mountain areas timber and bamboo are a locally available material. Measures which could benefit the availability of timber are the development of local or regional timber treatment plants and a temporary allowance of more timber sourcing for reconstruction.
- **Cement mortar / concrete**: Concrete is widely available in the urban areas. Reinforced concrete frames is the dominating construction method for reconstruction. Although workers are familiar with the material, the detailing is complex, and this part is therefore prone to construction errors. For remote areas the use of concrete is extremely challenging from a logistic point, since every building material has to be carried there by foot.
- **Steel**: Steel is not a local building material, and all steel sections need to be imported (from India) for high prices.
- **Modern materials**: New materials may be used to strengthen traditional housing. FRP are expensive, and need special technical expertise. But stainless steel, galvanized nails and wires, bituminous paper and PVC foil as water proofing may prove adequate in withstanding corrosion or decay.
- **Low-cost materials**: There are several very widespread and easily available low-cost materials such as bamboo, polypropylene (packing material), plastic carrier bag, rubber tire straps, steel rods and wires. These materials can be used in wire mesh grid or strips. Bamboo however is not viewed as a material for permanent construction.
7 RETROFIT EVALUATION AND PROPOSALS

7.1 Evaluation of existing methods

In this section an overview is given of existing measures to enhance the structural integrity of building elements for unreinforced masonry buildings. Retrofit options are classified into subgroups as:

1. Wall-to-diaphragm connection
2. Stiffening of diaphragm
3. Connecting perpendicular walls
4. Seismic belts
5. Strengthening of wall
And combined methods

First the possible techniques are briefly discussed in this chapter. Then the measures are evaluated on the basis of six design- and construction criteria. On the basis of this evaluation two preferred retrofit combinations will be proposed for further research.

<table>
<thead>
<tr>
<th>Design criteria</th>
<th>Construction criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural efficiency</td>
<td>Impact on architecture</td>
</tr>
<tr>
<td></td>
<td>Durability</td>
</tr>
<tr>
<td></td>
<td>Construct-ability</td>
</tr>
<tr>
<td></td>
<td>Cost-effectiveness</td>
</tr>
<tr>
<td></td>
<td>Material availability</td>
</tr>
</tbody>
</table>

A three-color grading system is used for the evaluated. Green is the most positive score and red the most unfavorable. For the criterion material availability an alternative rating is applied. The applicability with respect to material availability depends on the location of the house of implementation. For this category a measure is scored as appropriate for remote, rural/urban or urban core. The overview of the scores per retrofit measure is given in Appendix C1.

<table>
<thead>
<tr>
<th>Design and construction requirements</th>
<th>Material availability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Favorable solution</td>
<td>Feasible material in remote areas, either local or transportable by foot</td>
</tr>
<tr>
<td>Uncertain or neutral solution</td>
<td>Feasible and known material in urban and rural areas</td>
</tr>
<tr>
<td>Unfavorable solution</td>
<td>High-tech material, mostly imported and suitable for high-end buildings in urban cores</td>
</tr>
</tbody>
</table>

(a) For design- and construction requirements
(b) For the material availability

Table 7.1 Scoring system for the design and construction requirements

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10 A main reference for this section has been the Stone Masonry manual by Bothara and Brzév (2011) which contains an overview of possible retrofit techniques for non-engineered stone masonry buildings.
Chapter 7

Retrofit evaluation and proposals

Wall intersections

- Stone stitching
- Metal stitching
- Through-wall-anchor
- Anchored splint overlay

Belt

- Parallel timber bands
- Diagonal timber band
- Split- and bandaging
- Containment reinforcement
- Concrete band
- Horizontal bandage

Glass fibre overlay clamped under plywood floor

RC-diaphragm chord with shear locks

Strengthening wall

- Wire mesh
- Polypropylene mesh
- Bamboo mesh, geomesh, plastic carrier bag
- Interior concrete cage
- Strongbacks

For the source credits of the pictures, refer to the end of paragraph C1.
7.1.1 Wall-to-diaphragm

A wall-to-diaphragm connection may be divided into three components: the wall anchor, the connector element and the connection to the diaphragm (Arup, 2014). The connection must withstand both in-plane (shear) forces parallel to the walls and out-of-plane (pull-out/ push-through) perpendicular to the wall. Solid, thick masonry walls generally enable simple wall anchoring systems such as adhesive anchors and plate anchors. The anchor tie can be made with a bolt, rod or reinforcement bar.

- **Adhesive anchors**
  The anchors are drilled into the masonry. Anchorage is provided by means of adhesive bond and mechanical interlock. Epoxy resin is used as an adhesive. The strength of this connection is largely dependent on the quality and bonding to the masonry. Adhesive anchors have no effect on the façade appearance, as they do not protrude the exterior of masonry. Because of the high seismic loads and low-strength masonry, adhesive anchors are not sufficient with respect to anchorage capacity.

- **Anchorplate**
  The wall-tie can be secured by an anchor plate on the exterior of a masonry wall. Anchor plates have a higher resistance to pull-out forces than adhesive anchors, forming a larger pull-out cone in the masonry. The load is spread over the perimeter of the plate. The anchorplates are placed on the exterior of the masonry and impact the aesthetics of the façade. Exterior elements can be hidden behind a decorative plastered ridge or integrated into the façade by designing an aesthetically pleasing anchor plate.

The design of the ductile plate can contribute to energy dissipation. Carles (2012) has performed experimental and numerical studies on a curved petalled steel anchor plate, intended to dissipate energy through the yielding of the petals. Dissipation through plate yielding was less than expected. Dissipation through friction is suggested to be more effective and also more durable, which means increasign the contact surface of the plate and wall.

- **Connector element**
  The interface element between wall-anchor and diaphragm is called a ‘connector’ element. The connector is placed underneath or above the floor, depending on the location of the diaphragm strengthening. Connectors can be made of steel angle cleats, timber blocking or concrete sections. Steel sections are expensive, and have low workability on site.

- **Anchor to joist**
  The joist can be directly anchored to the wall via steel straps, joist anchors, joist plates, joist ties or even flattened rebar. This method is independent of the method for stiffening of the floor. It is feasible where no increase in diaphragm stiffness is needed; for example in low-seismicity areas. In the direction parallel to the joists the anchors can be fixed on the side of the beam. In perpendicular orientation the anchor has to penetrate the beams or has to be fixed at the bottom of the joists. It is hard to drill closely near the joists due to the dimensions of the drilling machine, creating a gap/eccentricity.

- **Fixation to diaphragm**
  The anchor or connector element must be fixed to the existing or strengthened diaphragm. Depending on the surface of the diaphragm, nails, screws or bolts are used. For concrete in-situ floors or concrete overlays, the fixations can be made directly with wall-tie elements which are laid into the wet concrete.
Failure mechanisms which may occur for wall anchors are listed below (Arup, 2014). Some failure mechanisms are prevented by using an anchor plate.

- **Pull out**
  a. Bond failure between anchor and grout
  b. Bond failure between grout and masonry
  c. Cone failure of masonry
  d. Wrench failure of masonry
  e. Steel tensile failure

- **Shear**
  a. Bearing failure on grout
  b. Bearing failure on masonry
  c. Anchor shear failure
  d. Anchor bending failure

![Figure 7.1 Several types of anchor failure](Source: adapted from Arup 2014)
### 7.1.2 Stiffening and strengthening of diaphragm

Diaphragm stiffness can be increased by modifying the existing floor or replacing the floor by a new diaphragm. The strength and stiffness required differ per building.

- **Nail plates**
  Plates can be nailed to connect existing adjacent timber boards. This method is labour intensive for the retrieved strength and stiffness (Figure 7.2 and Figure 7.3). Advantages are the little added mass and easy construction.

- **Diagonal straps**
  Diagonal straps are placed diagonally on the existing floor boards. Straps are made of flat straps of steel, timber planks or alternative material with high tensile strength such as FRP (Figure 7.3). In this solution the ties take the tensile forces and the floor boards the compressive force. The material of the FRP is relatively expensive, and requires specialized workers. It is light-weight and of high strength. Added mass is relatively low.

- **Plank or plywood overlay**
  Additional layer(s) of planks are nailed upon the existing floor, in perpendicular of diagonal direction with respect to the original boards to contribute to stiffness in multiple directions. Single or double additional planks are possible; though adding more layers will also add unfavourable extra mass. Nailed panel sheathing and (additional) layer or multiple layers of plywood can serve both as vertical load carrier and resist the in-plane diaphragm forces. The plywood panels are nailed or glued to the joists. Gluing the plywood yields high increased stiffness (Figure 7.2), but it is a time consuming activity. The method ensures a leveled surface for the floor. The plywood can also be applied underneath the floor joists; this will cause a lowering of the storey height and create a new ceiling, covering the traditional appearance of timber joists. Plywood can be hard to access in remote areas.

- **Truss bracing**
  Additional truss bracing can be applied underneath the floor by means of steel or timber sections. These should be members with a minimal section height, seen the already limited storey height of the old masonry buildings. Steel sections require specialized labour and high costs and are therefore not feasible.

- **RC-overlay**
  A *reinforced concrete overlay* delivers a very high increase of diaphragm stiffness (Figure 7.2). The topping must have a thickness of minimally 40 mm, and be adequately fixed to the existing floor by means of shear connectors (steels studs)). If sufficient shear connectors are applied composite action can be achieved. This solution adds significant rigidity, but also enlarges the demand due to the added weight. As the timber floors of these buildings are quite flexible and allow significant deformation, a thin concrete layer might be prone to cracking. The concrete overlay can contribute as vertical load carrier working composite with the timber. The in-situ overlay provides a good opportunity to connect the anchors by laying them into the wet concrete.

- **Replace diaphragm**
  In case it is not feasible to structurally improve the existing timber floor, a *new diaphragm* can be constructed above, below or instead-of the existing floor. Replacing the existing floor by a concrete floor does add a significant amount of mass to the building, increasing inertia forces. A lighter alternative is replacing the floor or roof with a steel braced diaphragm or horizontal truss. However, steel sections require specialized labour and high costs.
Figure 7.2 shows a comparison of the range of achievable increase in relative stiffness of several upgrading measures, as calculated by Arup (2014). The increase is presented relative to the stiffness and strength of a typical existing timber diaphragm with nailed planks, as calculated by Arup from the ASCE 41-13.

\[
\frac{K_{req}}{K_{as-built}}
\]

Figure 7.2 Range of increase in relative stiffness for several strengthening measures (Source: Arup, 2014)

Figure 7.3 shows a comparison of the range of achievable increase in relative strength for several upgrading measures, as calculated by Arup (2014).

Figure 7.3 Range of increase in strength for several strengthening measures (Source: Arup, 2014)
7.1.3 **Perpendicular walls**

The connection between perpendicular walls is improved by means of stitches or corner overlays. This connection is also enhanced by means of seismic belts, as described in paragraph 7.1.4.

- **The diagonal stitching of metal ties**
  Diagonal steel bars are drilled with an inclined pattern into the walls. Injection of grout provides the necessary bond of the bars with the masonry. This method is hard to execute due to the diagonal drilling in the masonry.

- **Stone stitching, metal stitching**
  For stone stitching new stones are placed diagonally embedded into the wall. The alternative is done with metal stitching, containing steel strips welded to anchor plates which are shifted into the wall at the intersection (Gettu & Santhanam, 2007)

- **Anchored splint overlay.**
  Vertical splints in the form of L-shaped mortar overlays (Jitendra Bothara & Brzev, 2011) can be used to strengthen the intersections of perpendicular walls.

7.1.4 **Belts**

The box-action of a building can be enhanced by creating bands or bandages around the perimeter of the building. This type of reinforcement acts as a sort of belt. The belt ties the walls at floor- and roof levels preventing separation of perpendicular walls (Bothara & Brzev, 2011).

- **Band**
  Horizontal continuous bands are made with reinforced concrete, timber or bamboo. Besides tying the building elements together, the band provides a uniform introduction of loads from roof to wall (Tomaževič, 1999). The placement of the band is most effective at lintel or roof level (Jitendra Bothara & Brzev, 2011). For newly constructed buildings multiple bands can be placed over the height of the outer wall. For retrofit a portion of the existing wall needs to be removed for the intervention. Therefore these methods seem more appropriate for new construction. Edging the reinforced concrete band with bricks can function as formwork for the pouring of concrete and exterior cover-up of the roof band.

- **Containment reinforcement**
  Containment reinforcement is installed by making grooves in the mortar joints (usually horizontal channels at both sides). Steel bars, which can be concrete reinforcement bars, are then inserted into the grooves and covered up by mortar (Gettu & Santhanam, 2007). For better durability stainless steel bars should be used. If the bars are threaded at the endings, the bars can be bolted and anchored by steel end plates. The big advantage of containment reinforcement is that it is not visible in the façade, but it is hard to construct. Vertical containment could function as pre-stressing of the masonry. The overburden prevents tensile forces in the masonry. However, this requires the drilling of long vertical slots in the masonry.
- **Bandages, split- and bandaging**

  Bandages, "seismic belts" are thin reinforced mortar overlays around the perimeter of a building. Bandages do not require demolition of a part of the wall since it is applied externally (Jitendra Bothara & Brzev, 2011). A distinction can be made between horizontal bandages, and the split-and-bandaging technique which contains additional vertical splints adjacent to openings (Gettu & Santhanam, 2007). The band can be made of ferrocement with galvanized welded steel wire mesh. As a low-cost alternative the bandages can be made with a closely spaced mesh of polypropylene straps, a material which is easily available since it is used for packing (Paola Mayorca & Meguro, 2004). Bands and bandages can be integrated into the brick façade by creating a decorative ridge or appearance.

### 7.1.5 Strengthening of wall

- **Jacketing**

  Jacketing increases the strength and ductility of unreinforced masonry walls. Mesh type retrofits contribute to the in-plane and out-of-plane flexural and shear capacity under lateral loads. Wires or strings connect the mesh on both sizes of the wall, tying the wall leafs together. The mesh is covered with a mortar overlay. Total packaging of the walls has a large impact on the brick façade exteriors and is a laborious task.

Sathiparan (2015) summarizes the critical issues and possible strategies for several types of mesh-materials: steel mesh, PP-band mesh, polymer mesh, bamboo mesh and plastic carrier bag. **Steel cage mesh** increases the seismic performance, but it is relatively expensive and susceptible to corrosion problems. **Polymer mesh** is compatible with the deformation of the earthen walls. **PP-band** has the advantages that it is very low-cost, durable and widely available, as it is a material which is used for packaging. **Plastic carrier bag mesh** is also low-cost and widely available, as it is made from ordinary plastic bags. **Bamboo mesh** is also cheap and low-tech, however the bricks do not seem to give proper protection to the bamboo. Charleson (2011) proposes a method of strapping the building with strips cut from used car tires. This low-tech recycled material can be applied by the house-owners. However, the straps are vulnerable to corrosion.

- **Strongbacks**

  The addition of internal strongbacks supports walls with long spans against overturning and out-of-plane toppling. Wooden beams and columns are a feasible retrofit option, especially in cases where concrete, steel and FRP’s are not allowed due to heritage considerations. Parajuli (2012) has done numerical analyses on a traditional masonry building and evaluated the addition of wooden elements at weak zones, near the openings and gable walls and fixing to the floor. The modification delivered a significant contribution to strength and decrease in deformation when subjected to the El Centro 1940 earthquake. Whilst the retrofit did prevent collapse, the building was not serviceable anymore.

- **Additional ‘cage’ structure**

  Additional frame structures are placed on the interior of the structure. As the masonry walls collapse, the frame might save the inhabitants inside. However, the additional frame structures might be stiffer or more flexible than the existing structure. The retrofit material and original must be compatible; otherwise they will tend to separate. In case of flexible structures, additional structure will contribute to the load resisting capacity after the masonry has started to crack. Reinforced concrete frames are much stiffer, and will therefore attract most of the forces. The heavy masonry walls will act only as dead load and so the concrete frame will have to be substantially (over) dimensioned (Nienhuys, 2003).
7.1.6 Combined methods

As this research aims to find a retrofit improving both connections and diaphragms, it is beneficial if solutions can be integrated. This section discusses combined solutions.

- **Steel anchor strap integrated with plywood diaphragm stiffener**
  A steel strap which stiffens the diaphragm (tensile forces) is fixed underneath a plywood sheet. The steel strap extends through the wall to function as an anchor.

- **Concrete overlay can be combined with smart wall anchors**
  A concrete overlay is combined with smart wall anchors embedded into the concrete, avoiding the need for a connector element. The connection is made by folding the reinforcement grid upward and fixing it to the wall with anchors, or by laying a wall tie into the wet concrete topping.

- **Mesh retrofit fixed to the diaphragm**
  Glass fibre meshes are frequently used for refurbishments of walls. The wall mesh is applied to the wall, folded over the floor and nailed to the diaphragm (Nienhuys, 2003). In this way both the walls can be strengthened, and the connection wall-to-floor is made. However, the one-side glass-fibre mesh connection has a weak connection to the wall. When the building starts deforming, it will pull-off the wall, as it is not attached to the exterior leaf.

- **Shear keys combined with a RC ring beam**
  A concrete ring beam is placed at the perimeter of the floor. Tapered shapes are cut-out of the masonry, and the ring beam is anchored to the wall by means of shear locks. The ring beam can tie the walls together as a diaphragm chord, where the 'tapered' shear locks offer resistance to both in-plane and pull-out forces for the wall-to-diaphragm connection. Reinforced concrete is a well-known and frequently used building material in Nepal. However, the detailing and construction of the shear locks is very challenging. The concrete will also add extra mass to the floor.

7.1.7 Summary of evaluation

The scoring of the measures per criteria is given in Appendix C1. A summary of the evaluation of suitable retrofit measures is given below.

**Wall-to-floor connection**
Adhesive anchors are not feasible with respect to anchorage capacity. Plate anchors provide more resistance to pull-out forces. Plate anchors are visible on the exterior, but a decorative ridge made of bricks or plaster can cover up the protruding floor-tie connections and also provide protection against corrosion. Joist anchors are independent of the diaphragm upgrading, and therefore only suitable for low-seismicity areas. Connector elements in the form of steel sections are expensive, have low workability on site and are stiff, whereas flexible connections are preferred for these structures. Timber blocks parallel to the wall can be used as diaphragm chords and to resist push-through forces.

**Stiffening of diaphragm**
Several diaphragm options are very labour intensive, such as nail plates between the planks, and diagonal plank overlay. Plywood, planks, concrete overlay and concrete floors provide a levelled surface and contribute to the vertical load bearing system, whereas bracings only resist the in-plane diaphragm forces. Solutions with concrete add mass to the structure, but concrete is well-known and widely available. Steel sections and FRP are very expensive and require specialized workers and equipment. Timber bracings and planks seem feasible in remote and hard to access areas.
Connecting perpendicular walls
Several options for connecting perpendicular walls are more feasible for newly built structures, such as timber parallel or diagonal horizontal bands. Several options such as diagonal corner stitches and anchors are hard to construct. Anchored splint overlays have positive effect, but bandages (seismic belts) seem more effective.

Seismic belts
Containment reinforcement has no interference with the appearance of the exterior, but is hard to execute correctly. Horizontal bands are feasible for new construction, but more challenging for existing construction, as parts of the wall should be removed. Horizontal bandages are feasible and effective. The bandage can be integrated into the façade by means of a decorative plastered ridge.

Strengthening of wall
The jacketing of walls on the exterior has a very large impact on the architecture and appearance of the façade. Strong backs can deliver a propping for long unsupported out-of-plane walls, or spread the load transfer. The addition of structural frame systems is not efficient due to the lack of compatibility with the existing structure (either too flexible or too stiff compared to the masonry).

Combined methods
Steel strips can function as wall-ties and be connected to the upgraded diaphragm. In-situ concrete overlay systems offer a good opportunity to connect anchor systems by laying the anchors into the wet concrete. Interior mesh which is clamped to the floor is hard to fix to the wall. A reinforced concrete diaphragm chord is feasible in urban areas, but the detailing of wedge shaped shear locks is very challenging.

The measures with the best overall scores and no unfavourable grading for any criterion are: plate anchors, steel strip anchors nailed to the diaphragm, concrete overlay, timber diaphragm bracing, plank overlay, and horizontal bandage.

Figure 7.4 Measures chosen for further research (Source: multiple, refer to end of Appendix C1)
7.2 Proposals for suitable retrofit methods

The evaluation of possible retrofit measures shows that local circumstances highly influence the appropriate retrofit strategy. Main aspects are budget, availability of materials, and accessibility to motorized vehicles, knowledge and equipment. Two categories of solutions are distinguished:

- **Remote areas**
  Remote areas are by definition hard to reach and sometimes totally inaccessible by motorized vehicles. Budgets are generally lower than in urban areas, as is the access to construction skills and equipment. Therefore a low-tech retrofit combination with local or light materials is proposed: timber floor bracing and wall plate anchors.

- **Urban or rural areas**
  Urban or rural areas are generally accessible by road allowing mechanical transportation of building materials. Budgets are generally higher and there is better access to equipment and construction skills. Although concrete adds weight to the structure, concrete and reinforcement are widely available and frequently used building materials in reconstruction. A seismic bandage is proposed, in combination with a reinforced concrete floor overlay.

The next section will elaborate on typical dimensions, material use and construction aspects for the proposed retrofit strategies.

7.2.1 Proposal for remote areas - plate anchors and diagonal bracing

**Wall-to-floor connection**
A workable and local option is to use (hard) wood blocks for the anchor plate. The ties are spaced at a distance of 600 to 900 mm (Arup, 2014; Nienhuys, 2003). The wall-ties can be made with reinforcement rods or steel strips:

- **Reinforcement rods**
  The anchor plates can be secured a rebar-nut connection at the exterior of the wall. On the interior the rod is welded to a steel perforated strip which is bolted to the diaphragm. An alternative, as proposed by Nienhuys (2003), is to flatten the rebar at the end, drill in holes so the flattened part can be nailed to the diaphragm. The steel rebar ties can be of 10 mm thickness.

- **Flat steel strips**
  An alternative is to use perforated steel strips. An elongated cut is made in the masonry. The steel strip is directly used as the wall tie. In this case no flattening or welding is necessary and the strip can be directly bolted to the diaphragm and the anchor plate. The anchor plate can exist of two timber slats nailed to the steel strip. The tie-connection must resist forces in-plane and out-of-plane of the walls. The anchor plate will resist pull-out forces, where timber blocks or slats can be nailed to the floor beams to resist push-out forces. Considering in-plane forces, in the direction parallel to the walls, the timber floor joists restrain movements in this direction. However, parallel to the joists wall-ties will
have little out-of-plane resistance. Therefore the anchors or strips are placed diagonally on the diaphragm, to provide resistance to the wall in-plane forces.

- **Anchor plate**
  
  The traditional facades are characterized by carved wooden elements. Timber blocks or slats as exterior wall-plates will fit in better with the façade. As there are many skilled wood workers in Nepal, depending on the budget the anchor plates can be carved with decorative pattern.

  As masonry tends to hold on to moisture, timber to masonry surface is susceptible to rotting. The timber plate could be carved out (concave) to minimize the contact surface. However, the carved shape also minimizes the friction surface, lowering the energy dissipation through friction (Carles, 2012). For the fixation, galvanised nails and screws are preferred.

---

Figure 7.6 Sketches of retrofit details

(a) Timber block anchorplate with rebar terminator
(b) flattening of rebar end, section of wall-to-floor connection,
(c) Rebar rod welded to a stone drill to make drills into the thick masonry walls (Adapted from: Nienhuys, 2003),
(d) Timber block attached directly with steel strip,
(e) section of wall-to-floor connection,
(f) concave block to minimize contact surface masonry timber
(g) Diagonal steel straps to provide some in-plane shear resistance (Source all except e: own pictures).11

---

**Diaphragm stiffening**

The timber floor can be upgraded by nailing timber planks diagonally to the bottom of the existing floor or roof joists. At the crossing of diagonals a steel strip can be applied as connector of the traversing beams. Due to the low ceiling heights of vernacular houses, it is favourable to minimize the height of the timber members to approximately 2 cm. The timber section does not need a lot of height to withstand in-plane forces. A thicker member is placed around the perimeter of the floor. This member functions as a diaphragm chord and helps to withstand push-through forces of the joist.

Possible alternatives to the timber bracing are timber planks, steel strip bracing or steel wire floor bracing. Steel strips are being used extensively as windbracing of roofs. The floors can also be braced by means of multiple low-cost galvanized steel wires. The wires can be pre-tensioned. As the thin wires are stressed and start to deform, they could possibly absorb some of the energy from the earthquake.

Figure 7.7 Timber diagonal floor retrofit (a) Stiffening of the floor with diagonal timber planks (Source: own picture) (b) Stiffening of roof (Source: Bothara & Brzev, 2011) (c) Diaphragm chords withstand push through forces (Source: own picture)

### 7.2.2 Proposal for remote areas – bandage and reinforced concrete overlay

**Concrete overlay**

A light-weight concrete topping, with a thickness of 40-50 mm, can be added on top of the existing timber floor. Although concrete adds weight, by removing the existing layer of clay topping, the weight is compensated:

- 100 mm of clay with a density of 1700 kg/m³: unit weight 170 kg/m²
- 50 mm of reinforced concrete overlay with density of 2500 kg/m³: unit weight 125 kN/m²

Generally the reinforcement is a welded wire mesh with a diameter of 5-6 diameter and a spacing of 200 mm (Baldessari, 2010; Brignola et al., 2009). Steel connectors between timber and concrete can ensure composite action. The shear connection between concrete topping and timber joists can be realized in several ways: L-shaped elements, shear studs, or slanted nails. These connectors are generally spaced 200 to 300 mm cc (14 mm diameter). Additional reinforcement (for example 3 bars of 14 mm diameter) can be placed around the perimeter as diaphragm chord.
nails as studs for concrete overlay (Source: UNIDO, 1983)  (b) Concrete overlay details (Source: Baldessari, 2010)

Figure 7.8 Concrete overlay on existing timber diaphragm

**Horizontal bandage**

A horizontal bandage at floor level ties the walls together. Typical bandage width are 200 to 400 mm, where the mortar thickness ranges from 40 to 50 mm (GOM, 1998). The mortar is applied in two layers; first a layer of mortar, then the mesh, covered by the outer layer of mortar. The mortar material can be reinforced cement, sand plaster or micro concrete (Bothara & Brzev, 2011). Ties, in the form of simple rebar rods, are laid into the wet concrete floor to secure the tie to the diaphragm. On the exterior the rebar is bent around bandage mesh.

![Figure 7.8](image)

**Alternative mesh materials**

The costs of the wire mesh can be lowered by using low-cost meshing material such as polypropylene. The dimensions of the structure will determine if these low-cost options are feasible.

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Figure 7.9 Bandage retrofit of wire mesh as seismic belt combined with an RC-overlay floor (Source: own picture)
PART III: NUMERICAL ANALYSES

Numerical analyses will be used to study the effect retrofit measures of connections and diaphragms on the global seismic performance of a vernacular structure.

A set of modelling assumptions and strategies are chosen based on the literature research (Part I). The overall analysis and modelling strategy is outlined in chapter 8. In order to evaluate the chosen modelling assumptions, a simple numerical model is set out against an experimental shaking table testing campaign from literature. Results and conclusions are presented in chapter 9. Moving forward from the lessons learned on modelling strategies, a simple one storey box-model is modelled with and without retrofit measures in Chapter 100. The simple model is comparable to one bay of a vernacular house.
Chapter 8 Analysis strategy

8 ANALYSIS STRATEGY

The objective of the numerical analysis part is to simulate the effect of the proposed retrofit strategies (Part II) on the global seismic performance of a vernacular house. Since the structure of an entire vernacular house is quite complex, it could be hard to establish the direct effects of the modifications. For this reason, simple box-shaped masonry models are chosen for the analyses. A set of modelling strategies is chosen based on the following criteria:

- Applicable for low-strength masonry structures
- Applicable for the chosen retrofit strategies
- Reasonable trade-off between accuracy and computational and modelling speed
- Feasible within the scope of this thesis

The analysis approach basically consists of two parts:

1. A numerical model is set out against a scaled experimental low-strength masonry model tested on a shaking table campaign from literature to evaluate the capacities and limitations of the numerical modelling strategies.
2. A full-scale simple box-model is modelled with- and without retrofit measures to evaluate the effect on the global seismic performance of the structure.

The chosen modelling methods are outlined for respectively: low-strength masonry, connections and diaphragms and masonry buildings without box-behaviour.

8.1 Low-strength masonry

- Masonry properties
For the comparison of the numerical model with the shaking table test from literature (chapter 9), the main masonry properties are taken from the associated component tests. A sensitivity study is done on several parameters to evaluate their influence on the model.

For the simulation of retrofits on the simplified box-model (chapter 10), the main material properties are taken from the experimental tests from Parajuli (2012). This campaign is executed specifically for the mud-bound masonry buildings of the Kathmandu Valley. For the missing values on the mortar bond strength, appropriate values are chosen referencing other tests from literature on mud-mortar brick masonry.

- Masonry modelling
The masonry walls are modelled with mesh-elements, as is current practice at Arup Amsterdam. The masonry behaviour is represented with a smeared crack material model. The smeared crack approach is chosen as a trade-off between accuracy and relative computational speed, since multiple models will be made and compared. Secondly, as the overall research goal is to assess the influence of connections on the global seismic behaviour, it is justified that a descriptive (global) modelling approach is chosen for the masonry.

An LS-DYNA material model for shell elements (MAT_SHELL_MASONRY) is applied, which is newly developed by Arup. The masonry model is still under development, so the version considered best practice at 25th of May 2016 is used for this research. Refer to Appendix APPENDIX D for a description of the material model MAT_SHELL_MASONRY, the considered failure modes and failure criteria.
At the time of research, the masonry model is capable of modelling several failure modes including: tensile and shear failure of mortar joints, combinations of both, and compression failure such as toe-crushing and rocking. Up to this point, the model does not include cracks through bricks. As for low-strength masonry the mortar is much weaker than the bricks, cracks through bricks are estimated not to be governing.

The material model has been compared and calibrated to single-leaf masonry components representative for the Groningen region (a province in the north of the Netherlands). The performance of the material model (version 12th October 2016) is also evaluated by Arup for thick, historic masonry walls by comparing brick-by-brick numerical models to shell-type models. The shell model is found to perform realistic or conservative for multi-leaf walls with interlocking. Without interlocking, for one loading case the shell model produced non-conservative values with respect to the brick-by-brick model (Arup, 2016). Multi-leaf layers (for example with rubble infill) sometimes have varying masonry qualities throughout a wall, and are vulnerable to delamination of wall wythes. However, with the shell-approach it is not possible to model explicitly the delamination of these layers. Walls with a non-homogenous build-up, without coherency between wall leaves, are left out of the scope of this research.

AEM-models seems promising for modelling low-strength masonry as the method allows explicit modelling of bricks although with satisfying computational speed. Within the limits of this thesis it was not possible to look further into this method, but for further research this method is recommended.

### 8.2 Connections and diaphragms

- **Masonry joint friction connections**
  The masonry joint-pocket connections are modelled by means of discrete beams, developing a frictional resistance, based on the vertical force read in the node.

- **Lintel**
  Lintels are the timber beams supporting the brick wall portion above the window. The lintel is represented as a shell, in order to be compatible with the masonry shells (in element type and geometry). The lintel is fixed at coincident nodes with the masonry, which is referred to as ‘merged’ with the masonry. It is assumed that the tearing of the lintel and masonry is represented in the masonry material, as it inherently represents internal mortar joints.

- **Nails in diaphragms**
  The masonry is modelled with shell elements, smearing the properties of bricks and mortar over the mesh material. One could argue that it would be consistent to model the diaphragm at the same scale, smearing the properties of the planks and nails over a set of shell elements. However, as it is found that the configuration of planks and nails can have effects on the diaphragm behaviour, it is chosen to model the nailed connections separately with hysteretic beams, and the planks with elastic beam. This method is found in literature (Arup, 2011; Judd & Fonseca, 2007), and is calibrated practice at Arup.

- **Wall-Anchors**
  The failure of the anchor connection is seen in literature to be dominated by shearing of nailed connections. The structural behaviour of the nailed anchor is represented by nonlinear discrete beams, with a prescribed force-displacement representing the shearing of its nailed connections.
• Linear elements
Timber materials used for the diaphragm and the upgrades are modelled as linear elements, as it is expected that the masonry will be governing in failure. It is acknowledged that one should check if the linear elements do not take unreasonable portions of force when checking ultimate model capacity, as they are not able to yield or break.

8.3 Masonry buildings without box-behaviour

• Calculation method
For the simulation of seismic behaviour, four main methods are available. Linear methods are: equivalent lateral force and modal analysis, and nonlinear methods are: push-over and nonlinear time-history. The lateral force is appropriate as an initial estimation of the loads and weight distribution throughout the height of a structure. The modal analysis is a useful tool to evaluate the dynamic behaviour of the model. However, for the final analysis of low-strength masonry walls, methods are needed which are able to capture the highly nonlinear material behaviour. The push-over test can deal with nonlinearity and damage propagation. The push-over test in TREMURI makes SDOF assumption, which is suitable for buildings with rigid floor diaphragms, but not for buildings without box-action. Furthermore, the assessment is only in-plane, although out-of-plane failures are of important influence for buildings without box-behaviour. As the method is quasi-static, it can’t take into account the duration effect while Nepal has long duration earthquakes with many cycles.

Therefore nonlinear time history analysis is chosen as final analysis method, as it can consider all the mentioned key factors and is the most accurate method to model the seismic behaviour of masonry. The finite element software applied is LS-DYNA, specialized in nonlinear dynamic FEM-analysis by means of explicit time integration. The explicit solution method makes the program suitable to simulate seismic behavior up to collapse of the structure with relative ease.

• Loading
In LS-DYNA, motions can be applied to a structure in the form of accelerations, velocity and displacement, in the three main directions at once.

In chapter 9, the loading protocol is followed from the shaking table from literature. This loading protocol is a series of cyclic signals with varying frequency and increased acceleration levels, applied only in one direction.

For the simulation of retrofits in chapter 10, actual seismic records are applied, taken from El Centro earthquake in 1940. The motion is applied only in one direction, to increase the clarity in the interpretation of the results. Actual earthquake motions are chosen in order to take into account the duration effects and frequency spectrum. It is acknowledged that the use of time history records would prefer the use of multiple records, to capture a full range of possible seismic occurrences.
Chapter 9 Verification numerical model to shake table test

9 VERIFICATION NUMERICAL MODEL TO SHAKE TABLE TEST

In this section a numerical model is compared to an experimental test from literature. There are limited experimental tests done on brick masonry bound by mud-mortar representative for developing countries. The experimental campaign of Sakhiparan (2016) is representative due to the application of a very weak mortar mix. In the testing campaign, single storey, box-shaped masonry models of quarter scale, are subjected to cyclic accelerations on a shake table. Three models are tested with different roof conditions. Two out of three experimental tests are compared for this research. First a model without roof and secondly a model with a timber roof fixed to the four walls with bolts.

The experimental test has also been used as experimental verification for the use of AEM modelling for low-strength masonry by Guragain (2012). Although the loading sequence was shortened to 22-43 loading cycles, and the roof structure were strongly simplified, the crack patterns obtained by the AEM method were in good agreement with the experimental results.

![Figure 9.1 Experimental box-shaped masonry models](Source: Sathiparan, 2016)

9.1 Objectives of comparison

The comparison is done to gain insight in the capacities and limitations of the chosen numerical modelling strategies to reproduce the actual experimental behaviour of low-strength masonry structures without box-action. The performance objective is not to exactly match forces and displacements, but to match overall failure mechanisms and crack patterns.

The first model has no roof, only four masonry walls with openings and lintels. This set-up minimizes the amount of parameters in the comparison to evaluate the masonry modelling strategy. The model without roof has no box-action and is susceptible to out-of-plane modes. Capturing this behaviour is important for the overall research goal.

The second model with roof is compared to provide an evaluation of the modelling of connections and diaphragm. Comparison of first and second numerical model should show deviating failure patterns, associated with structures with and without box-action. The experiment shows that the second model experiences less failure in the out-of-plane loaded walls and more in the in-plane walls, as the diaphragm can distribute the forces to the vertical resisting elements proportional to the lateral stiffness.
There aren’t many shake table tests continued until collapse, since a collapsing model can cause damage to the lab and instruments. For this experimental campaign the (scaled) models were loaded until complete collapse. This test allows the opportunity to compare if the same mechanisms lead to total collapse, and to compare at which stage collapse occurs.

9.2 Test set-up

9.2.1 Geometry of the test model

The basic geometry of both models is a quarter scaled, single storey room. The overall dimensions are 933 mm x 933 mm x 720 mm and the single leaf wall thickness is 50 mm. A single door opening is placed on the east wall of 243 mm x 485 mm, and a single window of 325 mm x 720 mm on the opposite west wall. Timber lintels are placed above the openings, supported by a length of 30-50 mm in the masonry.

The first model contains only the four masonry walls and no roof. The second model has a timber pitched roof which is connected to the walls on all four sides. The roof is constructed with wooden trusses. Two inclined plywood sheets with the dimensions 1033 mm x 600 mm and a thickness of 10 mm are attached with 25 mm long nails to the trusses at an interval of 150 mm. The diaphragm-to-wall connection is made by a 10 mm diameter bolt at 160 mm spacing. A 5 mm layer of mortar is provided between the top of the wall and the wall-plate of the roof diaphragm.

Figure 9.2 Geometry of the box-model(Source: own picture, based on dimensions of Sathiparan, 2016)
9.2.2 Material properties

The models are built with brick units of 75 mm x 50 mm x 35 mm and a mortar joint size of 5 mm. The mortar mix applied is cement : sand : lime, in the ratios 1 : 2.8 : 8.5. This weak mortar mix is representative for low-strength mortar in developing countries (Sathiparan, 2016). The bricks are Japanese made adobe units, which are generally stronger than those in developing countries. The impact on the results is expected to be limited, since the capacity of the model is governed by the mortar bond strengths. Several materials tests were conducted to obtain the mechanical properties of the masonry. The compression stress-strain curve data were kindly provided by Sathiparan.

![Figure 9.3 Dimensions and set-up of the masonry components for the mechanical tests (in mm) (Sathiparan, 2016)](image)

![Figure 9.4 Compression stress-strain curve of three unburned brick specimens(Source: Sathiparan)](image)

<table>
<thead>
<tr>
<th>Material properties</th>
<th>Without roof</th>
<th>With roof</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression strength [MPa]</td>
<td>4.40</td>
<td>4.28</td>
</tr>
<tr>
<td>Shear strength [MPa]</td>
<td>0.0064</td>
<td>0.0057</td>
</tr>
<tr>
<td>Bond strength [MPa]</td>
<td>0.0045</td>
<td>0.0046</td>
</tr>
<tr>
<td>Diagonal shear strength [MPa]</td>
<td>0.042</td>
<td>0.041</td>
</tr>
</tbody>
</table>

Table 9.1 Averaged test values for the mechanical properties of the masonry components (Source: Sathiparan, 2016)
9.2.3 Loading protocol and measurement

The models are subjected to cyclic sinusoidal loading signals (with varying frequency in the range of 2 to 35 Hz and an increasing input amplitude ranging from 0.05g to 0.8 g. The number of cycles is kept constant over the runs. Due to the dynamic interaction between the shaking table and the models, the actual acceleration imposed by the shake table deviated from the intended (Table 9.2). For the model without roof the amplitude was circa 30% higher than the loading input in the computer, where the second model with roof is subjected to accelerations of approximately 8% higher shaking intensity (Sathiparan, 2016). The incremental loading sequence does not give insight at which acceleration level the model fails, but it does gradually demonstrate the damage propagation.

The models are loaded in their weak direction: in-plane for the walls with openings and out-of-plane for the side walls. The response of the box models is measured by means of accelerometers and laser displacement measuring instruments. The displacements are measured at roof level (L1, L2 and L3) and at the base (L5).

<table>
<thead>
<tr>
<th>Amplitude</th>
<th>Sweep</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05 g</td>
<td>03, 04, 05, 06, 07, 08, 09, 10</td>
</tr>
<tr>
<td>0.1 g</td>
<td>11, 12, 13, 14, 15, 16, 17, 18</td>
</tr>
<tr>
<td>0.2 g</td>
<td>19, 20, 21, 22, 23, 24, 25</td>
</tr>
<tr>
<td>0.4 g</td>
<td>26, 29, 32, 35, 38, 41, 44</td>
</tr>
<tr>
<td>0.6 g</td>
<td>27, 30, 33, 36, 39, 42, 45</td>
</tr>
<tr>
<td>0.8 g</td>
<td>28, 31, 34, 37, 40, 43</td>
</tr>
</tbody>
</table>

Table 9.2 Shaking input motion applied to the shaking table, (Source: Sathiparan, 2016)

Figure 9.5 (a) Positions of instrumentation (b) Sinusoidal cyclic loading series (Source: Sathiparan, 2016)

Figure 9.6 Sequence of cyclic signals and the difference in input motion (blue) and actual shaking (orange) at the shaking table for model 1 (Source: data retrieved from Sathiparan)
9.3 Numerical assumptions

9.3.1 Analysis method

The analysis method applied is the *Non-Linear Time History Analysis*, with a fixed base numerical model. The analysis software is LS-DYNA. Besides gravity loads, cyclic accelerations are imposed at the fixed base constraints in the weak horizontal direction.

9.3.2 General assumptions

The following general assumptions have been made in the analysis:

- Geometry of the model and construction details are based on the drawings provided by Sathiparan (2016).
- Damping – 2% small strain material damping is included for all materials. Further damping in the structure is derived from yielding element hysteresis.
- Damping – an additional 5% stiffness weighted damping is included for the masonry material to damp out high frequency numerical noise without affecting the response of the building.

9.3.3 Element types

For the model without roof the following elements are applied:

- Unreinforced masonry walls – Shell elements (4-noded elements, fully integrated, 5 integration points through thickness).
- Timber lintels – shells elements (4-noded elements, 1 integration point through thickness)\(^\text{12}\)

For the model with roof, additional elements are applied:

- Timber lintels – shell elements (4-noded elements, fully integrated, 5 integration points through thickness)
- Timber purlins – Beam elements (2-noded elements, 2 integration points along length)
- Timber board – Shell elements (4-noded elements, reduced integration, 3 integration points through thickness).
- Nails – Beam elements (2-noded, 2 integration points along their length, 4 integration points in section).
- Anchors – Discrete elements (2-noded), having multiple (up to 6DOF) independent, inelastic springs.

\(^{12}\) The lintel is modelled as a shell element with one mesh element in height. Ideally the lintels are modelled fully integrated to capture accurately the out-of-plane bending behavior, but this option was not applied here. But, because the lintels are in the plane of loading, this limitation is assumed to have little influence on the model.
9.3.4 Material properties

Suitable material properties are required as input for the main materials: masonry, timber and steel. The assumptions on material properties for the baseline model are given in Table 9.3 and Table 9.4. In the sensitivity studies (Section 9.5), some material property values are changed for consecutive models.

- Unreinforced masonry

The main masonry properties are taken from component tests on the scaled brick masonry wallets specifically for the masonry used in this shake table test (Figure 9.3).

<table>
<thead>
<tr>
<th>Masonry material property</th>
<th>Symbol</th>
<th>Model without Test values</th>
<th>Model with roof Test values</th>
<th>Assumed value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Masonry density [kg/m3]</td>
<td>( \gamma_m )</td>
<td>2000</td>
<td>2000</td>
<td>-</td>
</tr>
<tr>
<td>Masonry compressive strength [MPa]</td>
<td>( f_m )</td>
<td>4.40</td>
<td>4.33</td>
<td></td>
</tr>
<tr>
<td>Masonry Young’s modulus [MPa]</td>
<td>( E^* )</td>
<td>1100</td>
<td>1100</td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>( \nu )</td>
<td>-</td>
<td>-</td>
<td>0.2</td>
</tr>
<tr>
<td>Compressive stress strain curve</td>
<td>( f_m - \varepsilon )</td>
<td>averaged curve from tests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diagonal shear strength [MPa]</td>
<td></td>
<td>0.042</td>
<td>0.041</td>
<td></td>
</tr>
<tr>
<td>Shear bond strength [MPa]</td>
<td>( f_{vo} )</td>
<td>0.0064</td>
<td>0.0065</td>
<td>-</td>
</tr>
<tr>
<td>Flexural bond strength [MPa]</td>
<td>( f_w )</td>
<td>0.0045</td>
<td>0.0052</td>
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</tr>
<tr>
<td>Masonry shear friction coefficient [-]</td>
<td>( \mu )</td>
<td>-</td>
<td>-</td>
<td>0.75</td>
</tr>
<tr>
<td>Masonry shear modulus [MPa]</td>
<td>( G )</td>
<td>-</td>
<td>-</td>
<td>( \frac{E}{2(1+\nu)} )**</td>
</tr>
</tbody>
</table>

* The Young’s modulus is derived at the first 30% of the maximum capacity of an averaged compression stress-strain curve.
** Stand formula for isotropic materials is set as a default relationship in the material model

In the material model, the shear-modulus is calculated as default from the value input for the Young’s modulus with the standard formula for isotropic materials. Since this formula is actually not applicable to the anisotropic masonry material, sensitivity studies will be performed to optimize the input of this parameter. Furthermore, some input needed for LS-Dyna was not provided in the tests done for the campaign. These values were taken according to current practice at Arup.

- Timber and steel

No information was available on the specific timber or steel properties used in the shake table tests. In consultation with timber researcher Geert Ravenshort a strength class of C24 is assumed for all timber elements of the experimental model. The timber properties are taken from EN 338:2016. The density of pine plywood lies between 500-650 kg/m³*. The Young’s modulus of the plywood is given an averaged 6000 MPa**. Plywood exists of several layers which are placed alternatively perpendicular and parallel to the grain (Metsa Wood, n.d.).

<table>
<thead>
<tr>
<th>Material</th>
<th>Type</th>
<th>Mass Density (kg/m3)</th>
<th>Young’s modulus (MPa)</th>
<th>Poisson’s ratio ( \nu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber</td>
<td>Elastic</td>
<td>420</td>
<td>11000</td>
<td>0.3</td>
</tr>
<tr>
<td>Plywood</td>
<td>Elastic</td>
<td>600*</td>
<td>6000**</td>
<td>0.3</td>
</tr>
<tr>
<td>Steel</td>
<td>Elastic</td>
<td>7950</td>
<td>200000</td>
<td>0.3</td>
</tr>
</tbody>
</table>

** (Metsa Wood, n.d.)

Table 9.3 Material properties numerical model for unreinforced masonry

Table 9.4 Material properties for Timber, Plywood and steel
9.3.5 Description of the analysis model

In the following section the modelling methods are outlined per model component. For the model without roof these are: the unreinforced masonry walls and the lintel. The model with roof has also a wall-plate, roof truss beams and a roof sheet connected with bolts and nails to the trusses.

- **Unreinforced masonry walls**
  A masonry shell material model, newly developed by Arup, is used for the modelling of the masonry walls. The square shaped shell elements are given the same height as the bricks (40 mm). For a description of the material model and an explanation of which failure modes are captured and collapse criteria are considered, refer to APPENDIX D.

- **Lintel**
  The lintel is modelled as an elastic shell element, not as a 1D beam element. The four-noded timber shell elements enhance the compatibility with the masonry shells; in this way the lintel loads are transferred via two nodes in height to the masonry, replicating the true height of the lintel beam, instead of creating force concentrations on one node. Failure of the joint between lintel and masonry is represented in the masonry shells above and beneath the lintel, because mortar joints are implicitly defined in the smeared crack model.

- **Roof structure**
  The roof is supported by three timber trusses that span from side wall to side wall. The connections between the members of the truss beams are represented by nodal rigid bodies, which are constraints that maintain nodes at the same location.

  The plywood timber sheet of the roof is nailed to the trusses. The timber roof sheet is modelled with elastic shell elements. The nonlinear behaviour of the diaphragm is concentrated in the nailed connections. The nails in the experimental model are 25 mm. The thickness is estimated at 2 mm. The nails are modelled explicitly as non-linear beam elements, using LS-DYNA material model MAT_HYSTERETIC_BEAM. Nodal rigid bodies are used to connect the two nail-heads to a single shell node of the plywood.

  A load-displacement curve is defined for the non-linear behaviour in shear, representing yielding of the nail and crushing of surrounding timber material. An effective yield point is calculated with NEN-
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EN 1995, from which a shear force-displacement backbone curve is generated according to the nail model described by Foschi (1974), see Figure 9.9. The curve does not plateau or degrade, so for high values of strain the input is not accurate. The hysteretic beam is given a peak-oriented hysteresis shape (Figure 9.9b), whereby a pinching factor is included to produce pinching behaviour for shear hysteresis. The nails are defined to remain elastic under axial and bending behaviour, therefore the element is not able to represent pull-out failure of the nails. No ultimate strength of the nails or cyclic degradation was included in the modelling method. The method is current practice at Arup and is calibrated to tests on timber diaphragms with planks in literature.

The horizontal members of the truss are nailed to the wall plate which lies on top of the masonry walls. The nailed wall-plate to beam connection has been explicitly modelled, applying the same methodology as the nail-modelling of the roof-board to the trusses.

The wall-plate is placed on the top layer of bricks with a layer of mortar in between. Additionally the wall plate is fixed with bolts to the masonry. Meshing the wall-plate to the masonry walls is believed to demonstrate unrealistic tying behaviour to the masonry. Instead, the wall-plate is connected to the masonry with zero-length, 2-noded discrete elements. The discrete element provides mortar cohesion, vertical bearing support to the wall-plate and frictional sliding resistance in horizontal directions. For

- **Anchor – wall-plate**

  The wall-plate is placed on the top layer of bricks with a layer of mortar in between. Additionally the wall plate is fixed with bolts to the masonry. Meshing the wall-plate to the masonry walls is believed to demonstrate unrealistic tying behaviour to the masonry. Instead, the wall-plate is connected to the masonry with zero-length, 2-noded discrete elements. The discrete element provides mortar cohesion, vertical bearing support to the wall-plate and frictional sliding resistance in horizontal directions. For

---

13 IHARD is the option for the isotropic hardening of the hysteresis curve. For options 2.0-4.0 the abscissa represent the peak value of the plastic deformation. Option 4.0 produces peak-oriented hysteresis, as shown in Figure 9.9b. Refer to MAT_HYSTERETIC_BEAM keyword manual (14-1-2016).
the direction perpendicular to the wall a maximum displacement limit is defined of half the wall width, at which the discrete element will fail (representing the beam falling of the wall).

The wall plate is fixed with bolts to the masonry. At the location of the bolts, a parallel nonlinear discrete beam is created, resisting vertical tensile (uplift) and lateral shear forces. The bolts are given similar nonlinear shear force-displacement curve as for the nails in the roof (using NEN-EN 1995 and Foschi’s nail model). For the bolts the force-displacement is input by means of a force-slip curve. An unloading option, providing a quadratic unloading from peak displacement value to a permanent offset, provides the hysteresis shape. It should be mentioned that the method used to generate the force-slip curve for wall-plate to masonry bolts is actually meant for timber-timber connections. The ductility of the connection may therefore be overestimated.

![Force-slip curve for roof-to-wall bolts](image1.png)

![Build-up of the roof structure with bolts and nails](image2.png)

Figure 9.10 Connection roof to masonry wall (a) Force-slip curve for roof-to-wall bolts (b) Build-up of the roof structure with bolts and nails (Source: Sathiparan, 2016)

### 9.3.6 Loading protocol

The loading sequence for the numerical model is a reproduction of the test protocol of Sathiparan (2016), as described in paragraph 9.2.3. Sinusoidal loading curves are generated with varying frequencies in the range of 2 to 35 Hz and increasing input amplitude (as reported in Table 9.2). For model 1 (without roof) the amplitude was 30% higher than the loading input in the computer, where the second model (with roof) is subjected to approximately 8% higher shaking intensity due to dynamic interaction of the shake table (Sathiparan, 2016). The sinus functions are scaled to match the higher peak accelerations. For model 1 peak accelerations were read from the data of the recorded base acceleration received from Sathiparan and for model 2 all motions were generally scaled 8% higher.
9.4 Comparison own numerical model to experiment

The results of the numerical model and experimental test are compared on the basis of the developed crack patterns (damage propagation), failure modes and the force-displacement behaviour.

9.4.1 Comparison of crack patterns

The total loading sequence consists of a series of 43 cyclic signals. The state of the experimental model is presented by Sathiparan (2016) after run 25, run 34 and run 43. The numerical model experiences earlier collapse and a more rapid propagation of damage. During comparison for some stages an earlier stage of the model is shown than for the experimental. The numerical model is shown after stages 25, 34, 39 and halfway 41. After run 39, extensive damage occurs to the numerical models, and halfway run 41 the numerical model collapsed.

### How to read the MAT-SHELL Masonry plots?

The material model MAT_SHELL_MASONRY offers several output variables for plotting the results. Individual output variables can be checked in order to understand which failure mode has caused certain damage. The maximum crack opening or sliding displacement provides the most complete representation of the crack formation in the masonry. This output variable is generally used for the post-processing of results, and is used here to present the analysis results.

Alongside the damage plots, a contour bar is given. The contour colour indicates the value of maximum crack opening or sliding displacement (in mm) which has occurred during the whole analysis. Grey shells have not seen any cracking yet. Cracking becomes more severe moving from blue, to green, yellow, orange, red and magenta.

A linear concentration of coloured mesh elements indicates a crack in the masonry. Sometimes no clear crack line can be discerned, instead an area of cracking. A possible explanation is that the mortar strength is very weak, causing some sliding and cracking to occur already at low levels of acceleration.

Failure modes represented are:

- Bed joint opening
- In-plane sliding
- Out-of-plane sliding
- Diagonal modes

Figure 9.11 Masonry failure modes captured by MAT_SHELL_MASONRY for the executable of 25th of May 2016 (Source: Anup)
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Figure 9.12 Crack patterns of experimental model without roof after run 25 (Source: Sathiparan 2016)

Figure 9.13 Maximum predicted crack opening in masonry after run 25/43

Figure 9.14 Crack patterns of experimental model without roof after run 34 (Source: Sathiparan 2016)

Figure 9.15 Maximum predicted crack opening in masonry after run 34/43
After run 25

<table>
<thead>
<tr>
<th>ID</th>
<th>Damage experimental model</th>
<th>Damage in numerical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Vertical crack due to out-of-plane bending perpendicular to bed joints</td>
<td>Minor vertical crack</td>
</tr>
<tr>
<td>A2</td>
<td>Minor vertical cracks at the corner</td>
<td>Minor cracks at all corners</td>
</tr>
<tr>
<td>A3</td>
<td>Debonding of lintel and masonry</td>
<td>[-]</td>
</tr>
<tr>
<td>A4</td>
<td>Debonding of lintel and masonry</td>
<td>Minor debonding under the lintels at the supports</td>
</tr>
<tr>
<td>A5</td>
<td>Diagonal cracks at the bottom corners of the windows</td>
<td>[-]</td>
</tr>
</tbody>
</table>

The diagonal cracks propagating downwards from the window corners are not reproduced at this stage in the numerical model. However, as can be seen from the latest damage stage of the experimental model (Figure 9.19) these cracks remain minor and do not trigger failure.

![Figure 9.16 Sketch of crack patterns numerical model after run 25/43](image)

After run 34

<table>
<thead>
<tr>
<th>ID</th>
<th>Damage experimental model</th>
<th>Damage in numerical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1, B5</td>
<td>Diagonal cracks in the north and south walls, due out-of-plane actions.</td>
<td>Vertical/ horizontal cracks in north and south walls</td>
</tr>
<tr>
<td>B2,4</td>
<td>Vertical cracks at the corners</td>
<td>Vertical cracks at the corners</td>
</tr>
<tr>
<td>B3,6</td>
<td>Debonding of lintel and masonry and vertical cracks from the corners of the door and window openings towards the top of the wall</td>
<td>Debonding of lintel and masonry and vertical cracks from the corners of the door and window openings towards the top of the wall</td>
</tr>
<tr>
<td>B7</td>
<td>Diagonal cracks at the bottom corners of the windows</td>
<td>Horizontal cracks at the bottom of the window</td>
</tr>
</tbody>
</table>

The diagonal cracks in the north and south wall of the experiment are of a more vertical/horizontal nature in the numerical model (B1,B4) than in the experimental. Damage in the spandrels and debonding of the lintel is well captured.

![Figure 9.17 Sketch of crack patterns numerical model after run 34/43](image)
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Figure 9.18 Maximum predicted crack opening in masonry after run 39

Figure 9.19 Crack patterns of experimental model 1 after run 43 (Source: Sathiparan 2016)

Figure 9.20 Maximum predicted crack opening in masonry halfway run 41.
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After run 39 numerical

<table>
<thead>
<tr>
<th>ID</th>
<th>Damage experimental model</th>
<th>Damage in numerical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>*no comparison to experimental model at this stage of analysis</td>
<td>Severe debonding between lintel and masonry. Several shell elements deleted. Vertical cracks in the spandrel</td>
</tr>
<tr>
<td>C2</td>
<td>Vertical crack in the spandrel above door</td>
<td></td>
</tr>
<tr>
<td>C3</td>
<td>Large horizontal crack in the pier next to the top of the window</td>
<td></td>
</tr>
<tr>
<td>C4</td>
<td>Envelope-like cracking pattern</td>
<td></td>
</tr>
<tr>
<td>C5</td>
<td>Large horizontal crack in the pier next to the bottom of the window</td>
<td></td>
</tr>
</tbody>
</table>

The east and west wall are oriented in-plane with respect to the applied motion. The cracks in the masonry are located where the in-plane section changes; at the top and bottom of the wall openings.

![Figure 9.21 Sketch of crack patterns numerical model after run 39/43](image)

After run 41 numerical, run 43 experimental

<table>
<thead>
<tr>
<th>ID</th>
<th>Damage experimental model</th>
<th>Damage in numerical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>Separation of the spandrel above the window</td>
<td>Separation of the spandrel above the window</td>
</tr>
<tr>
<td>D2</td>
<td>Large vertical cracks in the spandrel above the door, on the verge of falling</td>
<td>Separation of the spandrel above the door</td>
</tr>
<tr>
<td>D3</td>
<td>An envelope-like crack pattern, Out-of-plane toppling of part of the north wall</td>
<td>Envelope like crack patterns. Large areas of cracking. No out-of-plane toppling of wall.</td>
</tr>
<tr>
<td>D4</td>
<td>On the south wall the masonry pocket has toppled out of the wall</td>
<td>Envelope like crack patterns. Large areas of cracking. No out-of-plane toppling of wall.</td>
</tr>
</tbody>
</table>

![Figure 9.22 Sketch of crack patterns numerical model halfway run 41/43](image)
Separation of the spandrels is captured by the numerical model, which is the main mechanism leading to total collapse. Tears corresponding to out-of-plane actions are captured, but out-of-plane toppling of the walls is not reproduced.

Figure 9.23 Out-of-plane failure (a) Free standing wall (b) tied at top (c) tied to sides (d) tied to top and sides (Source: Enggpedia) (e) experimental model (Source: Sathiparan, 2016)

Collapse

1. Separation of the adjacent walls casus them to act as freestanding cantilevers, only supported at the bottom
2. The freestanding walls are very vulnerable and collapse out-of-plane overturning
3. 
4. Total collapse of model

Figure 9.24 Sequence showing collapse of the numerical model at respectively a), end of run 41 b) and c) at the beginning of run 42 then d) during run 43.
9.4.2 Force-displacement behaviour

In this section the force-displacement response of the numerical and experimental model are compared. The base shear is defined as the maximum lateral force occurring at the base of a structure. The base shear coefficient is the ratio between the maximum base shear resistance and the weight of the model.

The roof level deformation is defined as the displacement at roof level minus the displacement measured at the base of the shake table. For the experimental model, there were several lasers applied to measure the storey drift and diaphragm drift. The laser at the top corner (L1) is assumed to be used for the measurements of the roof level displacement (9.25). Therefore in the numerical model, the top node at the corner is taken as the location to measure the displacements at the roof.

The maximum roof displacement is taken per run and plotted against the maximum occurring base shear coefficient of that run. It must be noted that these peaks may not occur at the same time, and the coupled values therefore do not express an actual physical relationship. The values of the experimental model are shown up till the point that parts start separating from the model (generally after run 37/43). The model experiences crack propagation at the corners, resulting in element deletion. From the moment of significant damage the measurements at the top corner node are unreliable.

Figure 9.25 Measurement of roof displacement (Source: Sathiparan, 2016)
The base shear force generated in the numerical model is depicted by the orange line in Figure 9.30. From run 25 and onwards it can be seen that while the input acceleration is increased, the base shear does not increase accordingly anymore. From this point on significant plastic deformations are occurring in the numerical model. The numerical model does not reach the same levels of base shear as the experimental model. Apart from an outlier which lies around 0.73, the main data points reach a coefficient of 0.5 which is approximately 60% of the base shear coefficient reached in the experimental model (0.81).

The black line shows the weight of the model. The staggered drop of the weight line around run 41 indicates that masonry parts are separating from the model. At this point serious damage arises, leading to collapse. Collapse occurs earlier for the numerical than for the experimental model. The numerical model collapses halfway the 41st run, whereas the experimental model reaches 43 runs. The moment of collapse does lie approximately in the same range of time.

The walls are very flexible since they are not constrained at the top. The walls are mostly excited in the out-of-plane modes. Strongly magnified displacement plots of model states in the elastic range (Figure 9.26) demonstrate the outwards bulging of the out-of-plane walls in the opposite of direction of motion. The in-plane walls take an S-shape. From the force-displacement graph (Figure 9.28) it can be seen that initially the corner point experiences less displacement (stiffer) than the experimental model. It seems as if the numerical model behaves too stiff in the elastic range. However, as plastic deformation starts to occur, the corners bulge outwards while keeping angular distance between the perpendicular walls. This phenomenon results in large deviations with respect to the experimental model in the force-displacement behaviour in the nonlinear range (Figure 9.28). Although in early stages the outwards bulging of corners is only seen if the model displacements are significantly magnified, in later stages this behaviour is visible without magnification (Figure 9.27). The numerical behaviour of the corners seem unrealistic. A potential explanation is that the interlocking of the perpendicular walls with this particular masonry material model might produce corners which behave too stiff. This is referred to as the ‘flange’ effect.

![Figure 9.26 Magnified out-of-plane displacements of the model in the elastic range (a) run 11 t=65.3s (b) t=65.4s (c) run 19 t=130.5s (d) t=130.6s](image)

![Figure 9.27 Outward displacement of corners](image)
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Figure 9.28 Shear resistance with deformation at roof level measured at one corner node

Figure 9.29 Acceleration at base of total loading sequence

Figure 9.30 Base shear of the model (orange) and the total model weight (black) (model mass * 9.81)
9.5 Sensitivity studies

The masonry material model requires many input parameters for its material properties. In this section several sensitivity studies are done to establish the effect of changing certain parameters on the analysis results. Deviations between experimental and numerical model give rise to these studies:

- The lateral stiffness of the model is too high in the elastic range, as demonstrated by the force-displacement graph (Figure 9.28). The corners experience too much displacement in the plastic range. There are two input values associated with elastic stiffness: the young's modulus and the shear modulus.
- The maximum base shear in the numerical model only reaches 60% of the maximum lateral force in the experimental model. The mortar bond strength is governing for the strength of the walls ((Sathiparan, 2016). A sensitivity study is performed in which the parameters associated to mortar bond are increased by 200%.
- The input parameter for diagonal tension strength is not directly the diagonal shear strength of the masonry. The input is based on calibrations for Dutch masonry. A percentage of 25-75% of the mortar shear cohesion is advised. However, as the shear bond strength of low-strength masonry is exceptionally low, taking a percentage value leads to unrealistic low values. In the baseline model a value is chosen in the range of the actual measured diagonal tensile strength.
- The friction coefficient is initially set to a value of 0.75. Later on I discovered that average friction values for low-strength masonry lie around 0.6 ((Kiyono & Kalantari, 2004; Parajuli & Kiyono, 2015; Sathiparan & Meguro, 2015) The effect of changing the friction parameters is evaluated.
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9.5.1 Lateral stiffness

The parameters associated with the elastic stiffness of the structure are the young’s modulus and the shear modulus. The input value for the young’s-modulus is derived from the compressive stress-strain curve from the component tests. A standard equation for isotropic materials is set as default value for the masonry material, relating shear-modulus to the young’s modulus:

\[ G = \frac{E}{2(1 + \nu)} = 0.4E = 40\%E \]

Masonry is an anisotropic material, so this relationship between the shear and young’s modulus does not apply. Tomažević (1999) found that masonry shear moduli derived from experiments varied from 6 to 25% of the young’s modulus. This finding suggests that the parameter for the shear modulus needs to be decreased in the model. However, for stability of the numerical model it is more robust to alter the young’s modulus, which then calculates to a lower shear-modulus via the default relationship.

The base shear coefficient is plot versus the deformation at roof level in Figure 9.37. A close-up of the elastic range (Figure 9.37) shows that the numerical model with a young’s modulus of 30 percent of the initial value shows more resemblance in elastic lateral stiffness to the experimental model. This adaptation leads to a shear modulus which is approximately 10% of the young’s modulus instead of 40%. The ratio of 10% lies within the range (6-25%) found through experimental tests by Tomažević (1999).

The base shear coefficient versus roof level displacement graph also produces more realistic deformation values in the plastic range for the reduced young’s modulus. It can be seen that the bulging out of the corner effect is lower. The 30%E model experiences a slower development of cracks. A different collapse sequence is seen the spandrel above the door is separated first from the model.

\[ G = \frac{\tau_{xy}}{\gamma_{xy}} = \frac{F l}{A \Delta x} \]

Figure 9.31 Simplified diagram of shear deformation (Source: adopted from Wikipedia).

Figure 9.32 Plot of resultant displacement after run 39/43 with Young’s-modulus reduced to 30%
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Figure 9.33 Crack patterns of experimental model without roof (Source: Sathiparan, 2016)

Young’s modulus 100% (G=0.4E)

Figure 9.34 Maximum predicted crack opening in masonry of baseline model

Young’s modulus reduced to 30% (G=0.1E)

Figure 9.35 Maximum predicted crack opening in masonry of the model with reduced Youngs modulus
Figure 9.36 Base shear coefficient vs roof level deformation close-up on the elastic range comparing the experiment and numerical model with 100% and 30% Young's modulus.

Figure 9.37 Base shear coefficient vs roof level deformation experiment and numerical model with 100% and 30% Young's modulus.
9.5.2 **Mortar bond strength**

The maximum base shear in the numerical model only reaches 60% of the maximum lateral force in the experimental model. Therefore a sensitivity study is run with mortar bond values increased to 200%. For the increased mortar strength model the maximum base shear is in better agreement with the test results, as there are multiple data points in the range of 0.7-0.8.

![Graph showing base shear coefficient vs roof level displacement](image)

**Figure 9.38** Base shear coefficient vs roof level displacement of the experimental model and mortar strength values of 100% of the test values and 200%.
For the increased mortar strength model the damage is initially concentrated more to the in-plane walls. The most important in-plane vertical cracks above the spandrels are also reproduced by this model. An additional failure mode arises for increased mortar strength model. A horizontal crack is produced at the bottom of the pier next to the door. This sliding mode at the base is not seen in the experimental model.

![Figure 9.39 Crack patterns of experimental model without roof (Source: Sathiparan, 2016)](image)

Figure 9.39 Crack patterns of experimental model without roof (Source: Sathiparan, 2016)

![Mortar bond strength 100%](image)

After run 34/43  After run 39/43  Halfway run 41/43

Figure 9.40 Maximum predicted crack opening in masonry of baseline model

![Mortar bond strength 200%](image)

After run 34/43  After run 39/43  Halfway run 41/43

Figure 9.41 Maximum predicted crack opening in masonry of the model with mortar bond strength values of 200%
9.5.3 Diagonal tension strength

Diagonal tensile failure is considered as a combination of bed joint sliding and head joint opening. Failure occurs when the direct tensile stress exceeds a summation of the diagonal tensile strength and a frictional term relative to the compressive stress on the associated bed joints. For the diagonal shear parameter an appropriate input range is defined, calibrated to component tests. The value is defined as a percentage of the mortar shear strength. However, since the shear strength for low-strength masonry mortar is so minimal, this might not be a good assumption for this situation. For the baseline model a value is chosen which approximates the actual diagonal shear capacity (40 kPa), derived from a test specific diagonal shear test. A sensitivity study is done here to show what the model does if a percentage of the mortar shear strength (<5kPa) is taken as input for the parameter.

The model with the lower diagonal shear value displays deviating failure mechanisms with respect to the experimental test. The damage is initially mostly concentrated in the out-of-plane walls, whereas the tests show large cracks at the corners of the openings of the in-plane walls. The model with reduced diagonal tensile strength notably experiences collapse at an earlier stage (38 versus 41).

\[ \text{Diagonal tensile strength parameter - 40 kPa} \]

![Image](image1.png)

Figure 9.42 Maximum predicted crack opening in masonry of baseline model

\[ \text{Diagonal tensile strength parameter <5kPa} \]

![Image](image2.png)

Figure 9.43 Maximum predicted crack opening in masonry of the model with decreased diagonal tensile strength
9.6 Model with roof

A comparison is made of the crack patterns for the numerical and experimental model with roof.

After run 25

<table>
<thead>
<tr>
<th>ID</th>
<th>Damage experimental model</th>
<th>Damage in numerical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>[-]</td>
<td>Minor damage at the horizontal top layer of the masonry, where the roof is connected.</td>
</tr>
<tr>
<td>A2</td>
<td>Diagonal cracks at the corners of the window,</td>
<td>Very minor diagonal cracks at the corners of the windows</td>
</tr>
<tr>
<td></td>
<td>propagating towards the bottom and top of the roof</td>
<td></td>
</tr>
</tbody>
</table>

Already at this stage it is clear less cracks are developed in the out-of-plane walls (north and south) compared to the model without roof.

After run 34

<table>
<thead>
<tr>
<th>ID</th>
<th>Damage experimental model</th>
<th>Damage in numerical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1, B4</td>
<td>Horizontal flexural cracks, where the roof is fixed to the masonry wall</td>
<td>Horizontal cracks at the fixation of roof to the wall.</td>
</tr>
<tr>
<td>B2</td>
<td>Diagonal cracks above the corners of the door</td>
<td>Diagonal cracks above the corners of door opening</td>
</tr>
<tr>
<td></td>
<td>propagating towards the top.</td>
<td></td>
</tr>
<tr>
<td>B3</td>
<td>Horizontal cracks in the pier at bottom of the door</td>
<td>The horizontal cracks at the bottom of the wall with the door</td>
</tr>
<tr>
<td></td>
<td>opening.</td>
<td></td>
</tr>
<tr>
<td>B5, B6</td>
<td>Diagonal cracks at the top and bottom corners of the window become clearer and extend towards the outer corners.</td>
<td>Cracks at top and bottom of the corners of the window; the cracks are more vertical or horizontal in nature.</td>
</tr>
</tbody>
</table>

It is clear that less damage is occurring in the out-of-plane walls (north and south) than in the model without a roof, as now the diaphragm can transfer the loads to the in-plane walls.
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Figure 9.46 Crack patterns of experimental model with roof after run 25/45 (Source: Sathiparan, 2016)

Figure 9.47 Maximum predicted crack opening in masonry after run 25/45

Figure 9.48 Crack patterns of experimental model with roof after run 34/45 (Source: Sathiparan, 2016)

Figure 9.49 Maximum predicted crack opening in masonry after run 34/45
Figure 9.50 Crack patterns of experimental model with roof after run 43/45 (Source: Sathiparan, 2016)

Figure 9.51 Maximum predicted crack opening in masonry after run 39/45

Figure 9.52 Crack patterns of experimental model 1 after run 45/45 (Source: Sathiparan, 2016)

Figure 9.53 Maximum predicted crack opening in masonry halfway run 41/45
After run 39

<table>
<thead>
<tr>
<th>ID</th>
<th>Damage experimental model</th>
<th>Damage in numerical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1-C5</td>
<td>The cracks became wider with each following run, and critical in run 43.</td>
<td>The cracks become wider</td>
</tr>
<tr>
<td>C4</td>
<td>Weaking of the junction between perpendicular walls</td>
<td>Weakening of the connection between perpendicular walls is also observed, in the form of vertical cracks</td>
</tr>
</tbody>
</table>

![Figure 9.54 Sketch of crack patterns numerical model after run 39/45](image)

After experimental run 45 / numerical run 41

<table>
<thead>
<tr>
<th>ID</th>
<th>Damage experimental model</th>
<th>Damage in numerical model</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1,D3</td>
<td>Some horizontal portions of the top layer of masonry of the out-of-plane walls (north and south) are separated from the model. Bricks have fallen out at the point of this connection</td>
<td>Large horizontal area of cracking at the line of connection with the roof</td>
</tr>
<tr>
<td>D2</td>
<td>A top portion of the wall above the door separated from the model.</td>
<td>The spandrel hasn’t fallen out yet, but there are very large cracks surrounding the spandrel.</td>
</tr>
<tr>
<td>D4</td>
<td>The spandrel above the window toppled outwards.</td>
<td>The wall above the window has separated from the model, due to the large in-plane cracks propagating from the corners of the window.</td>
</tr>
</tbody>
</table>

![Figure 9.55 Sketch of crack patterns numerical model halfway run 41/45](image)

It is clear that the bolted connection was not governing, as the bolts are still fixed to the bricks, although the parts beneath have fallen out. The model has earlier failure than the model in the experiment. But it is clear that less failure is occurring in the side-walls and portions of the in-plane walls are toppling out, as is the case in the experiment.
Model collapse

1. At the end of run 42 both upper portion of the wall above the door- and window openings topple out, causing the roof to be supported only by the out-of-plane walls.
2. Eventually the model collapse by bulging of the out-of-plane walls, as is observed in the experimental model.
3. Causing total collapse of the model

Figure 9.56 Sequence showing collapse of the numerical model at respectively a) beginning of run 43 \( t=255.5 \)s and then b) \( t=260.0 \)s, c) \( t=260.2 \)s and d) 264.2s at the end of run 43.
9.7 Discussion

The main objectives of the comparison of the own numerical model to the experimental model were:

1. To compare the damage propagation and mechanisms leading to collapse.
2. To evaluate if out-of-plane failure modes are captured.
3. To evaluate the displacement behaviour of the model.
4. Confirm different failure patterns for the model without box-action and with box-action.

Conclusions are made on the basis on set-up, modelling methods and obtained results.

Test set-up and assumptions

- **Geometry**
  
  Since the model is scaled, the shell and beam elements are very small (and therefore stiff) in order to obtain accurate results and a clear display of crack patterns. For explicit analysis, the time step of the calculation is governed by the Courant condition (refer to paragraph 5.2.2). The short timber beam elements were governing, causing it to become very small. Although small models seem time-efficient, this scaled model had relatively long computational duration (8-10 hours) due to the small time-step. It was necessary to add some mass to the timber elements in order heighten the time step. This has possibly influenced the modelling results.

- **Loading protocol**
  
  The loading protocol with cyclic sinusoidal signals was simple and clear as input for the numerical analysis. The varying frequencies ensured that the model was loaded over a broad frequency spectrum. The increasing acceleration values made it possible to gradually evaluate the damage propagation. However, the duration of the loading protocol was quite long: 43 runs of cyclic signals, taking over 250 seconds. The long duration was also a cause of the relatively long computation time for the small model.

  The long test takes the experimental models far into the nonlinear range. As nonlinear damage propagation is inherently stochastic, preferably a larger sample of tests is needed to make a statistically significant comparison. Discrepancies between numerical model and experimental model are expected.

- **Material properties**
  
  Specific tests on the mechanical properties of wallets built with the masonry used for the test houses, including data on masonry compression strength, shear- and tensile bond strength. Not all values needed as input for the material model were available through tests. It is acknowledged that several input parameters could be optimized to obtain better results. Preferably calibration is done first for wall sub assemblages, and after on the structure. The head joint and bed joints are assumed the same mortar bond strength. The energy release rates of the head- and bed joints are taken as a percentage of the mortar bond strengths, according to best practice at Arup. There was no test input data on the Poisson’s ratio or the shear modulus.

Results model without roof

- The main failure mechanisms in the numerical model are shear, and diagonal modes. Since the mortar shear strengths are very low, these are credible failure mechanisms. Diagonal modes are a combination of bed-joint sliding and head joint opening.

- The overall propagation of damage shows satisfying resemblance to the experimental model with respect to reproducing the most severe cracks. At the first comparison stage (Run 25) the experimental model has several minor diagonal in-plane cracks which are not reproduced by the
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The initial diagonal cracks turn out to be small at the end (Figure 9.19). Both the numerical and experimental model experience debonding of the masonry and the lintel by means of bed joint sliding. Horizontal bed-joint sliding is also seen for both numerical and experimental at the bottom corners of the windows. Vertical cracks above the corner of the wall openings are replicated by the numerical model, as well as some damage in the spandrels. Similar to the experimental model, eventually the spandrel above the window topples out. The out-of-plane walls experience envelope-like two way bending cracks. Although cracks are roughly reproduced by the numerical model, out-of-plane toppling of the walls is not captured by the numerical model.

- **Mechanisms leading to collapse**
  The numerical and experimental models experience the same mechanism leading to collapse. The collapse of the spandrels causes the out-of-plane walls to lose their support, forcing them to behave as free standing cantilevers, which then rapidly collapse in the direction of the out-of-plane motion.

- **Moment of collapse**
  The models are excited in a sequence of 43 cyclic signals. Severe damage starts occurring in the numerical model after run 39/43. Somewhere halfway run 41/43 the spandrels above window opening fall down. The stage of collapse of the numerical model lies in a similar range as that of the experiment. The models fail during run 41, where the experimental model fails at run 43.

- **There is a deviation between the relationship base shear coefficient vs roof displacement of the experimental model and the numerical. The maximum base shear in the numerical is only 60% of experimental model. Since the numerical model withstands less base shear, it is regarded that the obtained result is conservative. Possible explanations for these deviations are:**
  - Inherent differences between numerical model and experimental model
  - The calibration of the masonry model is done for walls representative for Groningen
  - Uncertain input parameters for material properties which were not tested: fracture energy of head joints and bed joints, the shear modulus, the Poisson’s ratio, the diagonal tension strength
  - Difference in the method which is used to retrieve the base shear.

- **Generally for shaking table tests, the base shear is calculated by taking the acceleration response time histories at multiple positions on the model, and multiplying those values by their tributary mass. The base shear force in the numerical model is read by outputting the forces in the shell elements at the base of the model in the direction of the applied shaking. Inherently the base shear force in the numerical model is read more precisely, as the internal forces are calculated at each node of the meshed structure, whereas for the numerical model the developed shear forces are calculated by means of a limited amount of accelerometers.**

  Preferably one would take the average of multiple nodes for measuring displacement in the numerical model, to prevent capturing possible local numerical deviations associated with selecting only one node on the masonry.

- **Model displacement**
  The walls are mainly excited in out-of-plane modes. The structure does not develop a global response; it doesn’t behave as an SDOF structure. It is questioned if the story drift (roof level displacement) is representative for the global behaviour of the model without roof.
The out-of-plane walls bulge outwards in the middle and the corners are pushed outwards as residual displacement. Therefore the displacement of the numerical corner point diverges from the experimental data.

- **Evaluation use of model for next steps**
  More calibration of input values is needed if one attempts to evaluate the maximum resistance of a structure, or its actual capacity. This was not possible in the scope of this research. However, the objective of this research is to investigate enhanced performance, comparing the increase in capacity of alternative models. So therefore simulating the actual strength is not considered the highest priority. Furthermore, it is questioned how representative any specific model strength would be, as there is so much variety in the material properties and wall construction of low-strength masonry structures.

**Sensitivity studies**

- **Lateral stiffness**
  The shear modulus is initially set to a default relationship with the measured Young’s modulus. The relationship set is $G = E/(2(1 + \nu))$ which is approximately 40% $E$, but this relationship is not applicable for anisotropic materials such as masonry. The numerical model with Young’s-modulus of 30 percent of the tested value ($E = \sim 0.3 \text{GPa} \Rightarrow G = 10\%E$) shows more resemblance to the experimental model in terms of initial lateral stiffness. The newly found value corresponds with the E-modulus (0.27GPa) used by the Japanese research team (Furukawa et al., 2012), and comes closer to the approximate value of 0.5GPa suggested by Nienhuys (2003). It is chosen to proceed with the better matching input for the Young’s modulus.

  Besides parameters for the masonry, the modelling method of the timber lintel could cause an increase of the lateral stiffness of the model. The lintel is meshed into the walls. Failure of the interface between masonry and timber lintel beam is represented in the masonry shells above or beneath, in which the horizontal mortar joint is inherently defined within the smeared crack model. A limitation to this method is that the lintel might overestimate the bond between the timber lintel and the masonry, and create some tying effect of the elastically modelled lintel which overestimates the in-plane stiffness of the model. The model is mainly excited in out-of-plane modes, thus the influence of the potential in-plane stiffness increase may be limited. More research is needed to evaluate this effect. Furthermore, crack patterns show debonding of lintel and masonry, abolishing the tying effect.

- **Mortar bond strength**
  The baseline model showed lower maximum base shear than the experimental model, therefore a sensitivity study is done in which the mortar bond strength is increased by 200%. For this model two measured base shear coefficient values of the numerical model reach $\sim 0.8$, approaching the experimental results better. However, the model experiences less damage and deviating failure modes in the out-of-plane walls. It is chosen not to proceed with the altered mortar bond strength parameters.

- **Diagonal tensile strength**
  For the diagonal tensile failure, (combination of bed joint sliding and head joint opening) a calibrated parameter is set as input for MAT_SHELL_MASONRY. A value in the range of the shear strength of the bed joints is suggested. However, for low-strength masonry this value is extremely low, therefore using this low input causes early collapse and crumbling of the out-of-plane walls of the model. An input of $\sim 40$ kPa (in the range of the measured diagonal shear strength of an associated component test) yields better results, but preferably more calibration is done on this subject.
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Results model with roof

- **Main failure mechanisms**
  Main mechanisms are bed joint sliding and diagonal modes, similarly to the model without roof. However, the failure mechanisms are now mostly shifted to the in-plane walls and the location of the wall-to-roof connections.

- **Damage propagation**
  The numerical model demonstrates similarity to the experimental model with respect to reproducing the most important cracks. Initially small cracks are propagating diagonally from the corners of the window opening, accompanied by horizontal cracks at the location of the connections. Cracks in the numerical model are defined, but initially very small. The in-plane cracks around the wall openings grow larger, and ultimately cause the collapse of the spandrels. It is clear that both in the numerical and experimental model the damage is concentrated more in the in-plane walls, and on the top layer of masonry which is connected to the roof. This result was expected, as more shear force is now distributed to the in-plane elements via the roof diaphragm.

- **Mechanisms leading to collapse**
  Both numerical and experimental experience the same collapse mechanism: as the spandrels above the window openings separate from the model, the roof is only supported by the out-of-plane walls. These out-of-plane walls then experience horizontal bending due to the out-of-plane motion, ultimately leading to total collapse of the structure.

- **Moment of collapse**
  The time of collapse lies in the reasonably similar range as the experiments, reaching a state of extensive damage at run 41, and experiencing full collapse in run 43 with respect to run 45 for the experimental model with a roof.

- There has been no validation of the stiffness of the roof. There were no laser measurements done on the roof to evaluate its displacement behaviour. It is acknowledged that this can have a significant effect on the behaviour of the building. The modelling method of the nailed connections has been calibrated by Arup to experimental tests, although this calibration was done for timber diaphragms with planks and not with plywood sheets.

**Evaluation of model strategy for next steps**

Base shear and roof displacement aren’t perfectly captured by the numerical model. More research is needed to calibrate the modelling of the low-strength masonry and the connections. However, it is judged that the chosen strategies are sufficiently suitable to use for the purpose of the next steps of this research. The goal is to compare damage propagation of unstrengthened and strengthened models; to simulate the effect of retrofit measures on the global seismic response of a structure. The goal is not to capture as closely possible the actual capacity of the masonry.
Chapter 9 Verification numerical model to shake table test
10 SIMULATION OF RETROFIT EFFECTS

In this chapter, the effects of connection- and diaphragm retrofit are simulated by means of numerical analyses. For clarity, a simple structure is chosen which is representative of one bay of a vernacular house (Figure 10.1). A full house will have different dynamic behaviour and additional failure mechanisms, but a simplified model allows a clearer view on the effects of the modifications on the outcome of the analyses.

The geometry of this full scale numerical model is comparable to the scaled box-model of the prior chapter. However, several adjustments are made to the geometry in order to better approach the Nepalese traditional building style (refer to paragraph 10.2.1).

(a) Sketch of the numerical model
(b) Section of Newari house

Figure 10.1 Box-model for numerical analysis (Source: b: Korn, 2007)

10.1.1 Analysis methods

A series of numerical analyses is performed in this section. The first step of analysis compares two opposite extreme cases:

a) The unstrengthened model, with flexible diaphragm and friction masonry joint-pocket connections.
b) The fully fixed model, with a stiff diaphragm\(^{14}\) and fully fixed connections.

A gravity analysis is conducted on the unstrengthened model. Secondly, a modal analysis is performed, identifying the modal shapes and eigenfrequencies of the unstrengthened and fully fixed model. Then the opposite models are studied by means of nonlinear time history analyses. Accelerations are prescribed at the fixed base constraints, in one horizontal direction per analysis.

The next step is to implement the retrofit strategies proposed in paragraph 7.2 on the unstrengthened model. The effect of the retrofits is simulated via nonlinear time history analyses. Four retrofit combinations are studied:

1. Connections upgraded with plate anchors – diaphragm braced with timber diagonal planks
2. Connections upgraded with plate anchors – diaphragm stiffened by parallel timber planks
3. Horizontal seismic bandage at floor level – diaphragm stiffened by parallel timber planks
4. Horizontal seismic bandage at floor level – diaphragm stiffened by timber planks + reinforced concrete overlay

\(^{14}\) The diaphragm is given the same floor weight as the unstrengthened model for this comparison.
The seismic behaviour of the unstrengthened model will be compared to the performance of the retrofitted models, on basis of crack patterns, force- and displacement behaviour.

Figure 10.2 Numerical models – extreme cases

(a) Unstrengthened (b) Stiff floor and fully fixed connections

Figure 10.3 Numerical models of retrofit alternatives

1) Timber diagonal bracing and plate anchors
2) Timber planks and plate anchors
3) Timber planks and horizontal bandage
4) Concrete floor overlay and horizontal bandage
10.2 Description of analysis models

10.2.1 Unstrengthened model

Geometry
The main aspects of the geometry are:

- The floor-joists span from front to back façade.
- The timber joists have a section of 100 by 100 mm and are closely spaced at 300 mm.
- The model is rectangular; 4.5 m by 3.0 meter.
- The model has one door and a window opening in the front façade, and two windows in the back façade. The windows have lintels above and beneath the wall opening.
- The wall thickness is 350 mm, which is typical for low-rise (one to 2 storey) masonry buildings.
- The joists are supported in the masonry, without being tied by any shear locks or anchors.

![Figure 10.4 Numerical model of unstrengthened structure](image)

Masonry walls and timber lintels
The masonry walls are modelled as shell-elements with the newly developed LS_DYNA material model MAT_SHELL_MASONRY (Refer to APPENDIX D). The lintels are represented by elastic shell elements, which have coincident nodes with the masonry. Debonding of lintel and masonry is represented by failure in the masonry shells above or beneath the lintels.

Connections
The connection between perpendicular masonry walls is often weak and simply butt-jointed. At each corner, for one vertical row of shell elements (blue strip), the mortar bond of the head joints is weakened by 50% to approach the same effect.

In the direction perpendicular to the spanning of the joists, no structural connection is modelled. The masonry joist-pocket connections are represented by discrete beams. The discrete beam provides frictional sliding resistance in horizontal directions and some cohesion of mortar. The frictional force is defined by means of the friction coefficient $\mu$ and the vertical force which is read in the node. The discrete beam only transfers the overburden of the joist. The cohesion between the top of the joist and the masonry is expected to dissolve quickly when cracking occurs. After that, the vertical load transfer will drop quickly. The friction beam has a very high initial stiffness. Once the force exceeds the frictional resistance the beam will start to slide. The beam is protrudes through the whole wall. A maximum displacement limit is defined in the pull-out direction, at which the discrete element will fail.
Chapter 10 Simulation of retrofit effects

(Figure 10.5b). This represents the floor joist falling out of its pocket in the wall. In the push through direction the joist is allowed to slide. In-plane, a very small displacement limit is defined, because the beam is assumed to lie snug in its pocket. After reaching this limit the connection will become very stiff (Figure 10.5a).

**Component test friction beam**

A numerical component test is conducted to study the behaviour of the numerical friction beams which are used for the masonry joist-pocket connection. A cyclic displacement is prescribed (Figure 10.6a) to an isolated friction beam connection. A rectangular hysteresis loop is produced (Figure 10.6b) Close-ups of the curve show that the force initially ramps up with a high stiffness and then starts to slide at a certain friction force. Close-up of the curve at unloading show that the force ramps down with same stiffness as the direction of the force (and therefore its sign) changes.
Floor

There are no nailed connections between the planks and the floor joists. Therefore, only the floor joists are modelled. The joists are modelled with elastic beam elements. Planks and mud are considered superimposed dead load. 25% of the live load is taken as extra seismic weight. The extra weight is represented by modifying the density of the timber floor joists:

\[
γ_{mod} = G \cdot \left( \frac{1}{h_p} \right) \cdot \left( \frac{\text{spacing}}{w_p} \right) = 258,30 \cdot \left( \frac{1}{0,1} \right) \cdot \left( \frac{0,3}{0,1} \right) = 7750 \, kg/m^3
\]

<table>
<thead>
<tr>
<th>Elements</th>
<th>Density [kg/m3]</th>
<th>Width [m]</th>
<th>Height [m]</th>
<th>Spacing [m]</th>
<th>Unit mass [kg/m2]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor joists</td>
<td>700</td>
<td>0,1</td>
<td>0,1</td>
<td>0,3</td>
<td>23,33</td>
</tr>
<tr>
<td>Planks</td>
<td>700</td>
<td>0,02</td>
<td></td>
<td></td>
<td>14</td>
</tr>
<tr>
<td>Mud</td>
<td>1700</td>
<td>0,1</td>
<td></td>
<td></td>
<td>170</td>
</tr>
<tr>
<td>Seismic live load</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>50(^\dagger)</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>258,30</strong></td>
</tr>
</tbody>
</table>

Table 10.1 Unit mass of the floor components

10.2.2 Model with stiff floor and full connections

The fully fixed model represents the opposite extreme case with respect to the unstrengthened model. The connections are fully fixed, the diaphragm is stiff and the perpendicular walls are interlocking; simulating full box-action. The modelling method of the masonry walls and lintels is kept similar to the unstrengthened model.

Connections

The slab shell nodes are coincident to the wall mesh nodes (merged), this means that a full connection is assumed.

Diaphragm

The stiff diaphragm is modelled as a reinforced concrete slab of 150 mm height. The slab is modelled elastic, as the masonry is expected to be governing.

The weight of the slab is kept similar to the unstrengthened model, in able to purely identify the effect of the box-action. Therefore the density of the slab is modified such that it equals the slab weight of the unstrengthened model containing timber joists, planks and mud. The total weight of the floor slab of the unstrengthened model is:

\[
A_{floor} = b \cdot (l - 0,3) = 3 \cdot (4,5 - 0,3) = 12,6 \, m^2
\]

\[
W_{timber \, floor} = A \cdot w = 12,6 \, m^2 \cdot 258,30 \, kg/m^2 = 3250 \, kg
\]

The modified density of the concrete slab:

\[
γ_{mod} = \frac{W_{timber \, floor}}{L \cdot B \cdot h} = \frac{3250}{3 \cdot 4,5 \cdot 0,15} \approx 1600 \, kg/m^3
\]

\(^\dagger\) 25% of 2 kN/m2 for residential buildings, according to NBC 105 : 1994
10.2.3 Wall-anchors

The wall-tie is a steel strip fixed to the floor joist with several nails and secured by an anchor plate on the outside of the wall. For both pull-out and push-through forces nailed connections are assumed to be governing. The total effect of three nails is modelled with a discrete beam, which has been given a prescribed shear force-displacement backbone curve. The curve is multiplied by 3 to replicate 3 nails. For a description of the nail modelling, refer to paragraph 9.3.5. The in-plane restraint is governed by the floor joists lying snug in their masonry pocket. This restraint is defined in the discrete beam formulation.

Where the walls are parallel to the floor joists, the strips are placed diagonally on the joists to provide some in-plane (shear) resistance. To represent this geometry, the actual steel strip is modelled, as are the nails connecting the strip to the floor joists.

The discrete beam is attached to multiple masonry nodes by means of a nodal rigid body, to replicate the load spread by the plate anchor perimeter. In order to ensure that adjacent cone failures do not interact, a minimum spacing is required of:

\[ s_{\text{min}} = d + 2h_{\text{ef}} = 200 + 2 \times 350 = 900 \text{ mm} \]

In which: \( h_{\text{ef}} = t_{\text{wall}} = 350 \text{ mm} \)

\[ d = \text{plate dimensions} \]
\[ t_{\text{wall}} = \text{wall thickness} \]

![Figure 10.9 Spacing of anchors: minimum to prevent cone interaction (Source: Arup, 2015a)](http://www.strongtie.de/products/detail/windrispenband/458)

The assumed dimensions are as follows:

<table>
<thead>
<tr>
<th>Anchorplate</th>
<th>Assumed dimensions</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor plate</td>
<td>200x200 mm</td>
<td>175 x 175mm (Moreira et al., 2014)</td>
</tr>
<tr>
<td>Spacing</td>
<td>900 mm</td>
<td>( s_{\text{min}} = d + 2h_{\text{ef}} ) (Arup, 2014)</td>
</tr>
<tr>
<td>Steel strip(^{17})</td>
<td>80* 5 mm</td>
<td>(Bothara &amp; Brzev, 2011)</td>
</tr>
<tr>
<td>Nails</td>
<td>3 nails x 3 mm diam length: 65 mm</td>
<td></td>
</tr>
</tbody>
</table>

\(^{16}\) The nodes are spread only over one layer of shells in height, to limit interfering with the masonry shear failure mode.

\(^{17}\) [http://www.strongtie.de/products/detail/windrispenband/458](http://www.strongtie.de/products/detail/windrispenband/458)
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Figure 10.10 Numerical model with braced diaphragm

Figure 10.11 Modelling of the plate anchor

(a) Line model  (b) Elements modelled with true thickness

Figure 10.12 Modelling of the steel strip, fixed with nails to the floor joists
10.2.4 Timber diagonal bracing

The diagonal timber bracing elements are modelled underneath the timber floor joists. Along the perimeter of the walls, additional planks are placed as diaphragm chords. The bracing planks are modelled with elastic beam elements. The planks are nailed to the joists with several nails per crossing of diagonal-joist.

![Figure 10.13 Timber bracing retrofit top view](image)

<table>
<thead>
<tr>
<th>Bracing member</th>
<th>Assumed dimensions</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brace member</td>
<td>100 x 20 mm</td>
<td>20 mm reference (J. Bothara)</td>
</tr>
<tr>
<td>Spacing nailed connection</td>
<td>300-360 mm per nailed connection</td>
<td></td>
</tr>
<tr>
<td>Nail dimensions</td>
<td>3 or 4 nails per crossing joist 3 mm diam, length: 65 mm</td>
<td></td>
</tr>
</tbody>
</table>
10.2.5 Timber planks

The timber planks are modelled with elastic beam elements. The planks are nailed to the floor joists every 3 joists (which is around 900 mm) with two nails per connection. Nails are modelled with hysteretic beams at their actual locations. Where the planks aren’t nailed, they are supported with a friction beam, providing a vertical support and horizontal friction between the timber elements.

![Figure 10.14 Timber bracing retrofit top view](image)

**Figure 10.14** Timber bracing retrofit top view

![Figure 10.15 Modelling of nailed connections of timber planks](image)

**Figure 10.15** Modelling of nailed connections of timber planks

<table>
<thead>
<tr>
<th>Timber planks</th>
<th>Assumed dimensions</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber plank</td>
<td>100 x 20 mm</td>
<td>20 mm reference J. Bothara</td>
</tr>
<tr>
<td>Nails spacing</td>
<td>2 nails per 900 mm</td>
<td></td>
</tr>
<tr>
<td>Nail dimensions</td>
<td>3 mm diameter length: 65 mm</td>
<td></td>
</tr>
</tbody>
</table>
10.2.6 Concrete floor overlay

The concrete floor overlay is located above a layer of timber floor planks. The overlay is modelled with composite shell elements of 100x100 mm dimension. Because the overlay is relatively thin, it is expected to be vulnerable to cracking. The material model MAT_CONCRETE_EC2 for reinforcement in LS-DYNA is applied in order to provide nonlinear material behaviour and the correct location of reinforcement throughout the height of the slab.

The concrete overlay is connected to the existing timber via shear studs. These studs are modelled similar to the diaphragm nails. In practice, the studs can be placed slanted for better shear connection. For this analysis, the nails are assumed to be straight for simplicity.

![Figure 10.16 Modelling of concrete overlay diaphragm – studs provide shear connection between the timber and concrete](image)

<table>
<thead>
<tr>
<th>Element</th>
<th>Dimensions</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overlay thickness</td>
<td>50 mm</td>
<td>&gt;40 mm (Bothara &amp; Brzev, 2011) typically 50 mm (Baldessari, 2010; Brignola et al., 2009)</td>
</tr>
<tr>
<td>Concrete cover</td>
<td>±35-40 mm</td>
<td></td>
</tr>
<tr>
<td>Reinforcing mesh (two directions)</td>
<td>Φ6 mm, spacing: 200x200 mm</td>
<td>(Baldessari, 2010; Brignola et al., 2009)</td>
</tr>
<tr>
<td>Studs</td>
<td>10 mm diameter</td>
<td>14 mm diameter (Baldessari, 2010)</td>
</tr>
<tr>
<td>Spacing of studs</td>
<td>200 : 300 mm</td>
<td>200-300 cc (Baldessari, 2010)</td>
</tr>
</tbody>
</table>
10.2.7 Bandage

The bandage retrofit consists of a horizontal strip of thin reinforced mortar overlay which is applied at floor height. The mortar can be reinforced cement / sand plaster or micro concrete and the reinforcement standard wire mesh. It is chosen only to model the welded wire mesh, because the out-of-plane resistance of the thin concrete will be limited and modelling it will complicate the model. The wire mesh is modelled by beams elements with the actual dimensions of the rods. The wire mesh is connected to the concrete overlay via wall-ties. The bandage rods are directly connected (meshed-in) to the masonry nodes at the location of the ties (spacing of 600 mm).

![Numerical model of horizontal bandage](image1.png)

![Horizontal bandage retrofit](image2.png)

Figure 10.17 Bandage retrofit method numerical model and reference (Source a: own picture, b: GOM, 1998)

<table>
<thead>
<tr>
<th>Bandage components</th>
<th>Dimensions</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>300 mm</td>
<td>Typically 200-400 mm (Bothara &amp; Brzhev, 2011)</td>
</tr>
<tr>
<td>Overlay thickness</td>
<td>50 mm</td>
<td>Typically 40-50 mm (GOM, 1998)</td>
</tr>
<tr>
<td>Mesh</td>
<td>Φ6 mm, 100x100 mm</td>
<td>(Brignola et al., 2009)</td>
</tr>
<tr>
<td>Ties spacing</td>
<td>600 mm</td>
<td>assumption</td>
</tr>
<tr>
<td>Tie diameter</td>
<td>Φ 10 mm</td>
<td>Typical larger reinforcement bar size (Nienhuys, 2003)</td>
</tr>
</tbody>
</table>

In the model which has nailed timber planks as a diaphragm, the bandage is connected with the same wall plate anchors as mentioned in 10.2.3. In the model which also has a concrete overlay, the bandage is tied to the concrete overlay by placing rebar elements into the wet concrete and folding them around the wire mesh on the outside of the masonry. These rods are represented by a zero-length discrete beam, spaced every 600 mm.

The rebar rod is modelled elastic-perfectly plastic. The force-displacement behaviour and characteristics are outlined in Table 10.3. Loading and unloading follow the same curve. No hysteresis is included here. Analyses show that the forces do not reach the plastic range (10.6.1).
Chapter 10 Simulation of retrofit effects

The shear stiffness of a circular section is defined as (Hoogenboom & Spaan, 2005):

\[
K_{\text{shear}} = \frac{32 \cdot G A}{37 \cdot L}
\]

Figure 10.18 Force-displacement behaviour of the wall-tie

<table>
<thead>
<tr>
<th>Material property</th>
<th>Symbol</th>
<th>Value</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>E-modulus [MPa]</td>
<td>(E)</td>
<td>200000</td>
<td></td>
</tr>
<tr>
<td>Axial stiffness [N/mm]</td>
<td>(K)</td>
<td>44857</td>
<td>(K_{\text{axial}} = \frac{E A}{L})</td>
</tr>
<tr>
<td>Axial length of rod [mm]</td>
<td>(L)</td>
<td>350</td>
<td>Thickness of the masonry wall</td>
</tr>
<tr>
<td>G-modulus steel [MPa]</td>
<td>(G_s)</td>
<td>80000</td>
<td></td>
</tr>
<tr>
<td>Diameter rod [mm]</td>
<td>(D)</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Section area rod [mm(^2)]</td>
<td>(A_{\text{rod}})</td>
<td>78.5</td>
<td>(A = \frac{1}{4} \pi D^2 = 78.5 \text{ mm}^2)</td>
</tr>
<tr>
<td>Free length of rod [mm]</td>
<td>(L)</td>
<td>100</td>
<td>Assumptions, relatively free in first wythe of mud-bound masonry</td>
</tr>
<tr>
<td>Shear stiffness [N/mm]</td>
<td>(K_{\text{shear}})</td>
<td>54313</td>
<td>(K_{\text{shear}} = \frac{32}{37} \cdot \frac{G A_s}{L})</td>
</tr>
<tr>
<td>Shear strength [N]</td>
<td>(F_{y;E:d})</td>
<td>11330</td>
<td>(F_{y;E:d} = A_{\text{rod}} \cdot \frac{f_y}{\sqrt{3}} = 78.5 \cdot \frac{250}{\sqrt{3}})</td>
</tr>
<tr>
<td>Deformation at yield shear strength [mm]</td>
<td>(\Delta x)</td>
<td>0.209</td>
<td>(\Delta x = \frac{F_{y;E:d}}{K_{\text{shear}}})</td>
</tr>
</tbody>
</table>

Table 10.3 Force-displacement characteristics of the wall-tie rod

Figure 10.19 Modelling of the wall-tie rod by means of a zero-length discrete beam
10.3 Numerical assumptions

10.3.1 Element types

Unstrengthened model:
- Unreinforced masonry walls – shell elements (4-noded elements, fully integrated, 5 integration points through thickness).
- Timber lintels – shells elements (4-noded elements, fully integrated, 5 integration points through thickness)
- Timber purlins – beam elements (2-noded elements, 2 integration points along length)
- Discrete (friction) beams – beam elements (2-noded elements, 2 integration points along length)

The fully fixed model, with a stiff diaphragm and fixed connections:
- Unreinforced masonry walls – shell elements (4-noded elements, fully integrated, 3 integration points through thickness).
- Timber lintels – shells elements (4-noded elements, fully integrated, 5 integration points through thickness)
- Concrete slab – shell elements (4-noded elements, 2 integration points through thickness\(^\text{18}\))
- Discrete (friction) beams – discrete elements (2-noded) comprising multiple independent, inelastic springs (up to 6DOF)

Retrofitted models:
- All elements from unstrengthened model
- Wall anchors – discrete elements (2-noded), having multiple (up to 6DOF) independent, inelastic springs.
- Steel strips – beam elements (2-noded, 1 integration points along its length).
- Nails – beam elements (2-noded, 2 integration points along their length, 4 integration points in section).
- Nodal rigid bodies – a selection of defined nodes which behave as a rigid body.
- Timber bracing – beam elements (2-noded elements, 2 integration points along its length).
- Timber planks – beam elements (2-noded elements, 2 integration points along its length).
- Steel rods – beam elements (2-noded, 1 integration points along its length).
- Reinforced concrete overlay – shell elements (4-noded elements, fully integrated, 7 integration points through thickness).
- Studs – beam elements (2-noded, 2 integration points along their length, 4 integration points in section).

10.3.2 Gravity loads and seismic mass

First a static (gravity) analysis is done. A percentage of 25% of the live load is applied as seismic weight to the floor slabs. The resulting stresses are set as initial stresses for the dynamic analyses.

Gravity loads included in the model are:
- Dead Load (DL): as calculated from material unit weight
- Superimposed Dead Load (SDL):
  - Unstrengthened: SDL Floors (mud+planks) = 1,84 kN/m²
- Live Load (LL):
  - All models LL Floors = 25% of 2 kN/m² = 0,5 kN/m²

All gravity loads, including the live loads are represented in the numerical model as mass. The total mass represents the seismic mass of the building.

\(^{18}\) 3 integration points through stiffness is preferred to actually capture out-of-plane bending.
### 10.3.3 Material properties

Suitable material properties are needed as input for the numerical model. The main materials are: unreinforced low-strength masonry, reinforced concrete, timber, plywood, steel.

**Unreinforced low-strength masonry**

The basic material properties for masonry are derived from the experimental masonry component testing campaign by Parajuli (2012). The material properties provided in the test were: density, compressive strength, Young’s modulus and Poisson’s ratio. Complementary values needed as input for the numerical model are derived from the lab testing of Sathiparan (2016), and the field tests of Parajuli & Kiyono (2015) and Kiyono & Kalantari (2004).

The following assumptions are made with respect to the input material properties, based on lessons learned from the numerical verification in chapter 9:

- In the shake table tests, no compression failures were seen such as toe-crushing. Furthermore, damage observation reports (such as Bothara et al., 2004) note that these kind of failure modes are rare for low-strength Nepalese brick masonry buildings. The compressive modes are unlikely for small buildings with no slender piers. Therefore the compressive stress strain curve is adapted in such a way that after the peak compressive resistance, the curve is extended as perfectly plastic.

- The young’s modulus is set to 300 MPa. This value is recognized to provide a more resembling lateral stiffness of the numerical model to experimental tests (paragraph 9.5.1). The objective of lowering the young’s modulus is to decrease the shear modulus. The value of 300 MPa resembles the value used by Furukawa et al. (2012). In their research, 270 MPa was chosen as Young’s modulus input for the numerical analysis of a traditional mud-bound masonry Newari structure.

- For the parameter associated with diagonal tension, a value of 40 kPa is adopted. This input has led to better results in terms of damage patterns than using the input calibrated for Dutch masonry (refer to Paragraph 9.5.3). It must be noted that this parameter could be optimized by further calibration.

<table>
<thead>
<tr>
<th>Material property</th>
<th>Symbol</th>
<th>Parajuli tests</th>
<th>Assumed value</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density [kg/m³]</td>
<td>𝛾_𝑚</td>
<td>1800</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Compressive strength [MPa]</td>
<td>𝑓_𝑚</td>
<td>1,8</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Young’s modulus [MPa]</td>
<td>𝐸</td>
<td>800</td>
<td>300</td>
<td>Based on conclusions shaking table test comparison</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>𝜈</td>
<td>0.25</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Compressive stress strain curve</td>
<td>𝑓_𝑚 − 𝜀</td>
<td>-</td>
<td>Curve scaled to 1.8 MPa</td>
<td>From Sathiparan (2016)</td>
</tr>
<tr>
<td>Diagonal tension parameter [MPa]</td>
<td></td>
<td>-</td>
<td>0,04</td>
<td>From Sathiparan (2016)</td>
</tr>
<tr>
<td>Shear bond strength [MPa]</td>
<td>𝑓_𝑣₀</td>
<td>-</td>
<td>0,005</td>
<td>From Sathiparan (2016), Parajuli &amp; Kiyono (2015)</td>
</tr>
<tr>
<td>Shear friction coefficient</td>
<td>𝜇</td>
<td>-</td>
<td>0,6</td>
<td></td>
</tr>
<tr>
<td>Shear modulus</td>
<td>𝐺</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*The G-modulus is calculated as default from the value input in E with the standard formula for isotropic materials. As this does not applicable to the anisotropic masonry material, sensitivity studies were performed to optimize this input by altering the Young’s modulus.

Table 10.4 Material properties assumed for Nepalese traditional brick masonry
Reinforced concrete

For the thin concrete overlay, the LS-DYNA material model MAT_CONCRETE_EC2 material is applied. The model can represent plain concrete, reinforcing steel or a smeared combination of both. Shell elements are defined with different layers through thickness, allowing the user to define smeared properties per direction for either plain concrete or reinforcement layers. As the general lay-out of the reinforcement is known, with this material model the correct positioning of reinforcement bars through thickness can be modelled. The reinforcement is given uniaxial material properties, resisting stress in axial direction of the reinforcement bars of that layer. The material model is able to capture concrete cracking in tension and crushing in compression, yielding of reinforcement, hardening and failure.

As defined in the building codes, the concrete must have a minimum crushing strength of 15 N/mm² at 28 days for a 150 mm cube (NBC 205: 1994). This minimum required strength is used as a reference for the concrete strength material property. For the reinforcement bars, the codes specify 3 types of reinforcement steel (Mild bars Fe250, High-strength deformed bars Fe415, Fe550). The mild steel bars are chosen as reference strength class.

<table>
<thead>
<tr>
<th>Material properties concrete</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength class</td>
<td></td>
<td>C12/C15</td>
</tr>
<tr>
<td>Density [kg/m³]</td>
<td>γc</td>
<td>2400</td>
</tr>
<tr>
<td>Compressive strength [MPa]</td>
<td>fc</td>
<td>20</td>
</tr>
<tr>
<td>Characteristic cube strength [MPa]</td>
<td>fck,cube</td>
<td>15</td>
</tr>
<tr>
<td>Mean tensile strength [MPa]</td>
<td>flm</td>
<td>1,6</td>
</tr>
<tr>
<td>Young’s modulus [MPa]</td>
<td>E</td>
<td>27000</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>ν</td>
<td>0,2</td>
</tr>
</tbody>
</table>

Table 10.5 Material properties assumed for non-linear concrete material

<table>
<thead>
<tr>
<th>Material properties reinforcing steel</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel grade</td>
<td></td>
<td>Fe250°</td>
</tr>
<tr>
<td>Young’s Modulus of Reinforcement [GPa]</td>
<td>E</td>
<td>200</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>ν</td>
<td>0,3</td>
</tr>
<tr>
<td>Yield stress [MPa]</td>
<td>fy,dl</td>
<td>250</td>
</tr>
</tbody>
</table>

Table 10.6 Material properties assumed for reinforcement steel

Figure 10.20 Stress-strain behaviour for reinforced concrete materials (LSTC, 2014)

° This value is taken from the Nepal National Building Code NBC 201: 1994 – Mandatory rules of thumb reinforced concrete buildings with masonry infill
Other materials

Properties of other materials are taken as:

<table>
<thead>
<tr>
<th>Material</th>
<th>Type</th>
<th>Mass Density (kg/m³)</th>
<th>E-modulus (MPa)</th>
<th>Poisson’s ratio ν</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber (existing)</td>
<td>Elastic</td>
<td>700</td>
<td>10000</td>
<td>0,3</td>
</tr>
<tr>
<td>Timber (new)</td>
<td>Elastic</td>
<td>420</td>
<td>11000</td>
<td>0,3</td>
</tr>
<tr>
<td>Plywood</td>
<td>Elastic</td>
<td>420</td>
<td>5500</td>
<td>0,3</td>
</tr>
<tr>
<td>Steel</td>
<td>Elastic</td>
<td>7950</td>
<td>200000</td>
<td>0,3</td>
</tr>
<tr>
<td>Reinforced concrete (simple)</td>
<td>Elastic, isotropic</td>
<td>2400</td>
<td>27000</td>
<td>0,2</td>
</tr>
</tbody>
</table>

Table 10.7 Material properties of other materials used in the analysis models
10.4 Ground motions

The numerical models are loaded in one horizontal direction at a time. Earthquake motions are simulated by means of prescribed accelerations which are applied at a fixed base. First the model is excited in horizontal X-direction. In a successive analysis the model is excited in horizontal Y-direction.

The seismic hazard of Nepal is reviewed in APPENDIX E in order to define appropriate ground motions for the numerical analyses. Several seismic hazard aspects influence the choice of ground motions:

- Frequency content
- Magnitude
- PGA
- Duration
- Soil characteristics

The surface ground motions of the well-known records of the El Centro 1940 earthquake, retrieved from the PEER Ground motions database\textsuperscript{20}, are chosen as input for the numerical analyses. The El Centro records display approximate similarity to the Nepalese seismic code spectrum (Figure 10.21) at the fundamental periods associated with low-rise masonry structures. The duration of the El Centro motions are in the same range (40-50 seconds) as the range which is calculated probabilistically for Kathmandu earthquakes (Parajuli et al., 2012, Appendix E2). The layer of sediments in the Kathmandu valley lies around 550-650m, corresponding to a shear wave velocity of 250 m/s at 30 m. The value is comparable to the vs30 =213m/s of the El Centro motions.

The magnitude and PGA of the El Centro motions are respectively 6,9 Mw and 0,28g. One of the latest seismic hazard studies for the Kathmandu Valley (Parajuli et al., 2012) shows a maximum earthquake acceleration of 0,5g for 10% exceedance in 50 years for soft soil in the Kathmandu region. This probability of exceedance corresponds to a return period of ~475 years, which is indicated by FEMA (1997) as standard performance objective of ‘Life Safety’ for normal buildings. The El Centro signal with highest PGA is chosen for analysis and scaled to match a peak acceleration of 0,5g. The time history motion is cropped to 33 seconds, as the acceleration has dropped below 10% of the PGA.

![Figure 10.21 Comparison of normalized response spectra for the NBC Design spectrum 105:1994 for soft soil and the earthquake records of Gorkha (2015), El Centro (1940), Kobe (1995) (Data from PEER Ground motion database)](http://ngawest2.berkeley.edu/)

\textsuperscript{20} Database at: http://ngawest2.berkeley.edu/
10.5 Analysis of unstrengthened and fully fixed model

In this section the seismic behaviour of two opposite extreme cases are compared; the unstrengthened and the model with stiff diaphragm and fixed connections. The scheme below gives an overview of the different analyses which are conducted:

<table>
<thead>
<tr>
<th>Numerical models</th>
<th>Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Par 10.4.1</td>
<td></td>
</tr>
<tr>
<td>Unstrengthened</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Gravity</td>
</tr>
<tr>
<td>Par 10.4.2</td>
<td></td>
</tr>
<tr>
<td>Unstrengthened</td>
<td></td>
</tr>
<tr>
<td>Stiff diaphragm</td>
<td>Modal analysis</td>
</tr>
<tr>
<td>and full connections</td>
<td>Ground motions</td>
</tr>
<tr>
<td>Par 10.4.3</td>
<td></td>
</tr>
<tr>
<td>Unstrengthened</td>
<td></td>
</tr>
<tr>
<td>Comparison two extreme cases</td>
<td>Original El Centro ground motions</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Unstrengthened</td>
<td></td>
</tr>
<tr>
<td>Stiff diaphragm</td>
<td></td>
</tr>
<tr>
<td>and full connections</td>
<td>Scaled El Centro ground motions</td>
</tr>
<tr>
<td>Par 10.4.4</td>
<td></td>
</tr>
<tr>
<td>Unstrengthened</td>
<td></td>
</tr>
<tr>
<td>Merged connections</td>
<td>Nonlinear time history analysis</td>
</tr>
<tr>
<td>Lintel modelling method</td>
<td></td>
</tr>
<tr>
<td>Par 10.4.5</td>
<td></td>
</tr>
<tr>
<td>Lintel merged</td>
<td></td>
</tr>
<tr>
<td>Lintel friction connections</td>
<td></td>
</tr>
</tbody>
</table>

Figure 10.22 Overview of initial analysis models
10.5.1 Static analysis (gravity)

A gravity analysis is performed on the unstrengthened model. Dead loads, superimposed and live loads are taken into account. The distribution of vertical stresses in the masonry is calculated with LS-DYNA. The vertical stress calculated by hand at the base of the structure is in good correspondence with the numerical results:

\[ \sigma_z = -10 \cdot \gamma_m \cdot h_{wall} = -18000 \cdot 2.5 = 45 \cdot 10^3 \text{ N/m}^2. \]

![Figure 10.23 Stress in global Z direction](image)

The line load on the timber floor joists is:

<table>
<thead>
<tr>
<th>Element</th>
<th>Width [m]</th>
<th>Height [m]</th>
<th>Spacing [m]</th>
<th>Weight [kg/m³]</th>
<th>Area load [kN/m²]</th>
<th>Line load [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams</td>
<td>0.1</td>
<td>0.1</td>
<td>0.3</td>
<td>7</td>
<td>0.23</td>
<td>0.07</td>
</tr>
<tr>
<td>Planks</td>
<td>0.02</td>
<td></td>
<td></td>
<td>7</td>
<td>0.14</td>
<td>0.042</td>
</tr>
<tr>
<td>Mud</td>
<td>0.1</td>
<td></td>
<td></td>
<td>17</td>
<td>1.70</td>
<td>0.51</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>2.07</strong></td>
<td><strong>0.622</strong></td>
<td></td>
</tr>
</tbody>
</table>

**Table 10.8** Dead load, superimposed and live loads for timber floor joists

The application of load combinations provides the following line load values:

\[ q = q_{per} + q_{var} = 0.622 + 0.60 = 1.22 \text{ kN/m [SLS]} \]
\[ q = 1.2 \cdot q_{per} + 1.5 \cdot q_{var} = 1.2 \cdot 0.622 + 1.5 \cdot 0.60 = 1.65 \text{ kN/m [ULS]} \]
\[ q = 1.35 \cdot q_{per} = 1.35 \cdot 0.622 = 0.84 \text{ kN/m [ULS]} \]

No load factors are applied within LS-DYNA. The maximum bending moment in the timber joists is calculated with LS-DYNA in SLS as \( 1.37 \cdot 10^6 \text{ Nmm} \). A hand calculation yields the same value:

\[ M_d = \frac{1}{8} q l^2 = \frac{1}{8} \cdot 1.22 \cdot 3000^2 = 1.37 \cdot 10^6 \text{ Nmm [SLS]} \]

![Figure 10.24 Bending moment under gravity loads](image)
The maximum bending moment in ULS is:

\[ M_d = \frac{1}{8}ql^2 = \frac{1}{8} \cdot 1.65 \cdot 3000^2 = 1.86 \cdot 10^6 \text{Nmm} \] [ULS]

\[ W_e = \frac{1}{6}bh^2 = \frac{1}{6} \cdot 100 \cdot 100^2 = 1.67 \cdot 10^5 \text{mm}^3 \]

\[ \frac{M_d}{W_e} = \frac{1.86 \cdot 10^6}{1.67 \cdot 10^5} = 11.1 \text{ N/mm}^2 < f_{m,0:d} = 25 - 35 \text{N/mm}^2 \rightarrow \text{the check satisfies} \]

The maximum vertical displacement at midpoint of the floor joists (sag) in the beams is calculated by LS-Dyna as 15.6 mm. A hand calculation yields:

\[ w = \frac{5}{384} \frac{ql^4}{EI} = \frac{5}{384} \frac{1.22 \cdot 3000^4}{10000 \cdot 8.33 \cdot 10^6} = 15.4 \text{ mm} \]

\[ w_{\text{max}} = 0.004L = 0.004 \cdot 3000 = 12 \text{ mm} \]

\[ \frac{w}{w_{\text{max}}} = \frac{15.6}{12} = 1.3 \rightarrow \text{The check shows that the sag at mid span is too high.} \]

In prior calculation the floor joist is modelled as pined-pinned. Whereas in reality, the masonry pocket connection will be partially clamped if the joist lies snug in the pocket. This will reduce the vertical displacement at midpoint of the beam. For beams which are fully clamped at the ends the sag is:

\[ w = \frac{1}{384} \frac{ql^4}{EI} = \frac{1}{384} \frac{1.22 \cdot 3000^4}{10000 \cdot 8.33 \cdot 10^6} = 3.1 \text{ mm} \]

\[ \frac{w}{w_{\text{max}}} = \frac{3.1}{12} = 0.26 \]

The excessive vertical displacement of the floor joists can be decreased by removing the mud cover on the floor, for example by replacing the mud by planks or sheets. By removing only the layer of mud, the line load decreases from 1.22 to 0.71 N/mm. De sag decreases from 15.6 to 8.9 mm.

\[ w = \frac{5}{384} \frac{ql^4}{EI} = \frac{5}{384} \frac{0.71 \cdot 3000^4}{10000 \cdot 8.33 \cdot 10^6} = 8.9 \text{ mm} \]

\[ \frac{w}{w_{\text{max}}} = \frac{8.9}{12} = 0.75 \rightarrow \text{The check satisfies} \]
10.5.2 Identification of modal parameters

The eigenfrequencies and mode shapes can be computed via an eigenvalue analysis in LS-DYNA. For the purpose of this analysis, the nonlinear materials are converted to linear, and the friction connections are converted to fixed. The modal analysis has been done for the unstrengthened model and the model with stiff diaphragm and fixed connections.

Mode shapes of the unstrengthened model

The unstrengthened model is very flexible, due to the lack of fixed connections and stiff diaphragm and the relatively flexible masonry. The model is excited in many modal shapes (38+). The masonry is excited mainly in primary (out-of-plane) modes.

The first 14 modes (around 3.0 Hz) consist of vertical bending of individual or pairs of floor joist. These modes do not activate a significant part of the structure’s mass. The first modes engaging significant parts of the mass (>50%) are

- Out-of-plane bending of the longitudinal wall (6.7 Hz)
- Out-of-plane bending of the transversal wall (9.9 Hz).

The next modes are several horizontal bending configurations of the beams, with eigenfrequencies ranging from 10.7 until 12.2 Hz. Mode 30 and higher show several different configurations of out-of-plane bending of the masonry walls, resembling S and W-like curves.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Vibration type</th>
<th>Period (s)</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-14</td>
<td>Vertical bending of individual, or pairs of beams</td>
<td>0.33</td>
<td>3.0</td>
</tr>
<tr>
<td>15</td>
<td>Out-of-plane (vertical) bending of longitudinal walls</td>
<td>0.14</td>
<td>6.7</td>
</tr>
<tr>
<td>16</td>
<td>Out-of-plane (vertical) bending of transversal walls</td>
<td>0.10</td>
<td>9.9</td>
</tr>
<tr>
<td>17</td>
<td>Upward bending of the floor joists</td>
<td>0.09</td>
<td>10.7</td>
</tr>
<tr>
<td>18-29</td>
<td>Horizontal bending of the floor joists</td>
<td>0.09-0.08</td>
<td>11.3-12.2</td>
</tr>
<tr>
<td>30</td>
<td>Out-of-plane bending longitudinal walls in S-shape, transversal walls</td>
<td>0.08</td>
<td>12.9</td>
</tr>
<tr>
<td>31</td>
<td>Out-of-plane bending longitudinal walls in S-shape, transversal walls</td>
<td>0.08</td>
<td>13.0</td>
</tr>
<tr>
<td>32</td>
<td>Out-of-plane bending of transversal walls – both inward</td>
<td>0.07</td>
<td>13.6</td>
</tr>
<tr>
<td>33</td>
<td>Out-of-plane bending of longitudinal walls and transversal walls, S-shape</td>
<td>0.06</td>
<td>15.8</td>
</tr>
<tr>
<td>34</td>
<td>Out-of-plane bending of longitudinal walls</td>
<td>0.05</td>
<td>18.2</td>
</tr>
<tr>
<td>35</td>
<td>Out-of-plane bending of transversal walls – W-shape</td>
<td>0.05</td>
<td>21.1</td>
</tr>
<tr>
<td>36</td>
<td>Out-of-plane bending of longitudinal walls – S-shape</td>
<td>0.04</td>
<td>23.5</td>
</tr>
<tr>
<td>37</td>
<td>Out-of-plane horizontal bending of longitudinal walls</td>
<td>0.04</td>
<td>24.4</td>
</tr>
<tr>
<td>38+</td>
<td>Several modes (minimal) local bending of masonry</td>
<td>0.04</td>
<td>25.7</td>
</tr>
<tr>
<td>...</td>
<td>...</td>
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</table>

Table 10.9 Identification of modal parameters of the unstrengthened numerical model
Figure 10.26 Modal shapes of the unstrengthened model as calculated with LS-DYNA.
Modal shapes for the model with stiff floor and fixed connections:

The model with a stiff diaphragm and full connections develops a more 3D response to the vibrations. The first mode is in-plane longitudinal bending, in-plane to the walls with openings. The second is in-plane transversal bending, in-plane to the closed side wall. The third mode is torsion. The building is asymmetrical since the front façade has one door and one window, whereas the back façade has two windows. This configuration results in a front façade with lower lateral stiffness.

![Modal shapes](image)

Figure 10.27 Modal shapes of the model with stiff floor as calculated with LS-DYNA

<table>
<thead>
<tr>
<th>Mode</th>
<th>Vibration type</th>
<th>Period (s)</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>In-plane longitudinal bending</td>
<td>0.079</td>
<td>12.7</td>
</tr>
<tr>
<td>2</td>
<td>In-plane transversal bending</td>
<td>0.074</td>
<td>13.5</td>
</tr>
<tr>
<td>3</td>
<td>Torsion of the masonry walls</td>
<td>0.053</td>
<td>18.8</td>
</tr>
<tr>
<td>4+</td>
<td>Several modes (minimal) local bending of masonry</td>
<td>0.04</td>
<td>25.7</td>
</tr>
</tbody>
</table>

Table 10.10 Modal parameters of the model with stiff floor and fixed connections
10.5.3 Dynamic analyses

Unstrengthened model - PGA 0.28g

First, the unstrengthened model is excited by the original El Centro motion. The magnified displacement plots at the time of the first major peak (Figure 10.28), demonstrate that the main modes of vibration are bending of the out-of-plane loaded masonry walls.

Overall damage plots show that at this level of ground motion, the numerical model only experiences very small cracks of ≤1 mm. The façade wall is 1.5 times as long (4.5 m) as the transversal wall (3.0 m). The longer unsupported façade walls cause the structure to be weakest when shaking in transverse direction. In Y-direction the model experiences larger and more cracks and 10x larger displacements.

Figure 10.29 Maximum predicted gap and sliding displacement of the whole analysis run

Figure 10.30 Maximum absolute displacement of the whole analysis run in the principal direction of movement
**Unstrengthened model vs stiff floor and full connections - 0.5g**

The seismic performance of the *unstrengthened model* is compared to a model with a stiff diaphragm and fully fixed connections.

The *unstrengthened model* does not collapse at this level of ground motion, but it does experience large crack openings (>10mm) and high levels of out-of-plane displacement of the walls (>150mm). The U-shaped displacement gradient on the out-of-plane walls show that the walls are moving independently. The models experience large vertical cracks at the junctions of perpendicular walls. Large areas of cracking are concentrated in the out-of-plane walls. The out-of-plane swaying of the walls causes a plastic hinge to occur at the bottom and large vertical cracks in the outer piers.

The model with stiff diaphragm and fixed connections sees the damage shift from out-of-plane to mostly in-plane for both loading cases. A clear transition has occurred from excitation in primary modes to secondary (in-plane) response. This is illustrated clearly by the plots of the absolute maximum displacement of the analysis. The plots show a parallel rainbow gradient over the height of the masonry walls, indicating the structure is moving as a whole.

The masonry experiences a clearly defined damage concentration at the fixation to the slab. Since the slab is fully fixed (merged) to the masonry, a very stiff connection is created. An abrupt change in stiffness (between the stiff slab and the relatively flexible masonry) causes the damage to be concentrated at this section. The in-plane loaded transversal walls do not display the typical X-shaped in-plane cracks over the full height of the wall. The displacement is concentrated at the upper half of the wall under the slab. The global response is an S-shaped sideways displacement. The masonry in the middle strip is relatively flexible compared to the fixed base lower half, and the stiff slab at the upper half. This strip in the middle experiences overall the most strain and several diagonal cracks (bed joint sliding).
**Unstrengthened model**

Shaken in X-direction

**Figure 10.33** Maximum predicted gap and sliding displacement

**Figure 10.34** Maximum absolute displacement in the principal direction of movement

**Stiff floor and fixed connections**

Shaken in X-direction

**Figure 10.35** Maximum predicted gap and sliding displacement

**Figure 10.36** Maximum absolute displacement in the principal direction of movement
Why is there less displacement in-plane of wall openings?

The fixed model which is shaken in-plane of the wall openings experiences lower absolute maximum displacement than the model shaken in-plane to the closed wall (11.4 mm to 62.8 mm). The façade walls have multiple openings and would therefore generally be expected to behave more flexible. One would also expect in-plane to the façade to be the weakest direction; the in-plane model however is undergoes to far less damage.

The modelling method of the lintels was initially suggested as possible explanation. The elastic lintel could have a tying effect on the masonry and enhance the in-plane stiffness in the façade. To check this hypothesis, a variation study (paragraph 10.5.5) is performed in which the lintels are connected with friction connections to the masonry. This study indicates that the lintel is in this case not the cause.
The other hypothesis considers the severe cracks in the transversal walls. It is important to note that the displacement plots (Figure 10.36) show the absolute maximum displacement over the whole analysis run. As expected, the black line in Figure 10.37 shows that the model shaken in X-direction is initially more flexible. At moment ‘A’ (t=3.6) the model has slightly more displacement (0.75mm) than the model shaken in Y-direction (0.55mm).

After the first large acceleration peak (Figure 10.38), the model shaken in Y-direction experiences its first large cracks. The solid wall has less deformation capacity (it is ‘force-controlled’). The cracked walls are softened too such extent that the model loaded in Y-direction instantly comes to have the largest displacements.

![Figure 10.39](image)

Figure 10.39 Absolute maximum displacement at respectively t=3.6 and t=3.7s, deformed shape magnified 100x

These results demonstrate the importance of nonlinear material models for masonry. The crack propagation in the masonry has in this case significantly influenced the degradation of strength and stiffness of the model, and by this its overall dynamic behaviour during the analysis.
Figure 10.40 In-plane crack propagation at t=3.6, t=3.7, t=4.0, and t=4.2, magnified 100x
**Base shear, roof displacement and diaphragm displacement**

In the following section the unstrengthened and tied model are compared on the basis of their maximum base shear coefficient, roof displacement and diaphragm displacement. The unstrengthened model excited by the original El Centro is also shown in the analyses. The force-deflection curves are given in APPENDIX F.

A comparison of the base shear coefficient shows that there is no significant difference in the maximum developed base shear between the unstrengthened and the tied model. Shaken in X-direction base shear coefficient goes from 0.44 to 0.45, shaken in Y-direction the base shear coefficient goes up from 0.37 to 0.43.
In X-direction both unstrengthened and fixed model experience a story displacement of ~10 mm. For the shaking case in Y-direction, the story displacement almost doubled from 26.3 mm to 45.6 mm when adding stiff diaphragm and fixed connections. This indicates that modifications result in the global structural response.

The diaphragm displacements decrease drastically moving from unstrengthened to fixed model:
10.5.4 Variation studies connections

Two variation studies are performed on the wall-to-floor connections:

The first study (a) evaluates the necessity of modelling the floor joists with friction connections. For this case, the joist ends are modelled coincident (merged) with the masonry. Results (Figure 10.43, Figure 10.44) show that the damage is highly concentrated at the line of connections, especially when shaken Y-direction. This does not seem realistic, as in the unstrengthened case the beams should be allowed to slide in the masonry pocket. This study seems to confirm the need to apply friction beam connections.

The next study (b) evaluates the effects of connecting all the walls to the beams. This case basically examines an extreme retrofit on the connections, without stiffening the diaphragm. The unstrengthened model is adjusted by adding rigid connections in the form of nodal rigid bodies from the floor joists to the masonry walls. For the beams running parallel to the wall, elongated nodal rigid bodies are created, which are constraints that maintain the nodes at the same location.

Also for this case the damage is concentrated at the level of the fixed connections. The nodal rigid bodies connecting the last beams to the transversal walls basically create a very stiff strip along the wall. This locally higher stiffness generates stress peaks in the longitudinal walls at the fixation point of the last joist.
Chapter 10 Simulation of retrofit effects

**Beam ends merged instead of friction connection**

Shaken in X-direction

<table>
<thead>
<tr>
<th>Maximum crack width (Max all pts)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
</tr>
<tr>
<td>mm</td>
</tr>
</tbody>
</table>

Figure 10.43 Maximum predicted gap and sliding displacement

Shaken in Y-direction

<table>
<thead>
<tr>
<th>Maximum crack width (Max all pts)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
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<tr>
<td>mm</td>
</tr>
</tbody>
</table>

Figure 10.44 Maximum absolute displacement in the principal direction of movement

**Walls connected to the floor joists at all sides**

Shaken in X-direction

<table>
<thead>
<tr>
<th>Maximum crack width (Max all pts)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
</tr>
<tr>
<td>mm</td>
</tr>
</tbody>
</table>

Shaken in Y-direction

<table>
<thead>
<tr>
<th>Maximum crack width (Max all pts)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
</tr>
<tr>
<td>mm</td>
</tr>
</tbody>
</table>

Figure 10.45 Maximum predicted gap and sliding displacement

Figure 10.46 Maximum absolute displacement in the principal direction of movement
10.5.5 Variation studies lintel modelling

In the prior models, the lintel nodes are modelled coincident (merged) with the masonry nodes. This method might overestimate the shear connection between the lintel and masonry, allowing the elastic lintel to work as a tie for the masonry. In this study the lintel is modelled as a beam. The beam is connected to the masonry with discrete beams representing friction (and some cohesion). Refer to paragraph 10.2.1 for more elaboration on the discrete friction beams. The end nodes at the lintel support are merged, to provide resistance to the beam sliding (endlessly) in-plane. This part is believed to rapidly tear.

![Figure 10.47 Model with the lintel modelled with friction-beams](image)

Besides an extra horizontal crack in the middle façade pier at the level of the door-opening, this study shows comparable crack patterns. Both absolute displacements at the end of the run, as the displacements at 3.7 seconds are very similar. From this analysis it is concluded that this modelling method does not provide significant difference in this case.

The lintel modelled ‘merged’ with the masonry

![The lintel modelled with friction beams](image)

The lintel modelled with friction beams

![Figure 10.48 Comparison of model displacements at 3.7s](image)
Chapter 10 Simulation of retrofit effects

**Stiff floor and full connections – lintels merged to masonry**

Shaken in X-direction

![Diagram](image)

**Figure 10.49 Maximum predicted gap and sliding displacement**

![Diagram](image)

Shaken in Y-direction

**Figure 10.50 Maximum absolute displacement in the principal direction of movement**

**Stiff floor and full connections – lintels with friction connections**

Shaken in X-direction

![Diagram](image)

**Figure 10.51 Maximum predicted gap and sliding displacement**

![Diagram](image)

Shaken in Y-direction

**Figure 10.52 Maximum absolute displacement in the principal direction of movement**
10.6 Simulation of retrofit measures

In this section, the retrofit proposals from subsection 7.2 are implemented into the unstrengthened model. For both retrofit proposals two diaphragm alternatives are tested. The following scheme gives an overview of the performed analyses:

<table>
<thead>
<tr>
<th>Numerical models</th>
<th>Analysis</th>
<th>Ground motions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate anchors + diagonal bracing</td>
<td>Plate anchors + timber planks</td>
<td>Scaled El Centro ground motions</td>
</tr>
<tr>
<td>Bandage + timber planks</td>
<td>Bandage + concrete overlay</td>
<td>PGA = 0.5g</td>
</tr>
</tbody>
</table>

Figure 10.53 Overview of retrofit analysis models

10.6.1 Dynamic analyses

Four retrofit alternatives are analysed. Results show that the damage and displacement of the differently retrofitted models are quite similar. The plots of the maximum gap- and sliding displacement show that the implementation of the retrofit does not necessarily lower the amount of damage in the masonry. However, for all the retrofitted models the damage is shifted from the out-of-plane toward the in-plane walls.

This kind of damage is less critical and generally more favourable than out-of-plane modes. In-plane failure modes are seen as less brittle modes (refer to paragraph 3.3), which experience less rapid failure than the out-of-plane modes. Out-of-plane modes result in sudden losses of equilibrium of portions of the masonry wall.

Figure 10.54 Primary modes (out-of-plane) versus secondary modes (in-plane) (Source: Touliatos, 1996)
Chapter 10 Simulation of retrofit effects

**Bracing + anchors**

Shaken in X-direction

Maximum crack width

<table>
<thead>
<tr>
<th>Max (all pts)</th>
<th>0.00</th>
<th>1.00</th>
<th>2.00</th>
<th>3.00</th>
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</table>

Shaken in Y-direction

Maximum crack width

<table>
<thead>
<tr>
<th>Max (all pts)</th>
<th>0.00</th>
<th>1.00</th>
<th>2.00</th>
<th>3.00</th>
<th>4.00</th>
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</tbody>
</table>

**Figure 10.55** Maximum predicted gap and sliding displacement

**Figure 10.56** Maximum absolute displacement in the principal direction of movement

**Planks + anchors**

Shaken in X-direction

Maximum crack width

<table>
<thead>
<tr>
<th>Max (all pts)</th>
<th>0.00</th>
<th>1.00</th>
<th>2.00</th>
<th>3.00</th>
<th>4.00</th>
<th>5.00</th>
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Shaken in Y-direction

Maximum crack width

<table>
<thead>
<tr>
<th>Max (all pts)</th>
<th>0.00</th>
<th>1.00</th>
<th>2.00</th>
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</table>

**Figure 10.57** Maximum predicted gap and sliding displacement

**Figure 10.58** Maximum absolute displacement in the principal direction of movement
Chapter 10 Simulation of retrofit effects

**Planks + bandage**

Shaken in X-direction

![Planks + bandage X-direction](image1)

Maximum crack width

<table>
<thead>
<tr>
<th>Max all pts</th>
<th>0.0</th>
<th>1.0</th>
<th>2.0</th>
<th>3.0</th>
<th>4.0</th>
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<th>6.0</th>
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</tbody>
</table>

Shaken in Y-direction

![Planks + bandage Y-direction](image2)

Maximum crack width

<table>
<thead>
<tr>
<th>Max all pts</th>
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<th>1.0</th>
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<th>3.0</th>
<th>4.0</th>
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Figure 10.59 Maximum predicted gap and sliding displacement

Figure 10.60 Maximum absolute displacement in the principal direction of movement

**Concrete overlay + bandage**

Shaken in X-direction

![Concrete overlay + bandage X-direction](image3)

Maximum crack width

<table>
<thead>
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<th>Max all pts</th>
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<th>2.0</th>
<th>3.0</th>
<th>4.0</th>
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<th>6.0</th>
<th>7.0</th>
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</table>

Shaken in Y-direction

![Concrete overlay + bandage Y-direction](image4)

Maximum crack width

<table>
<thead>
<tr>
<th>Max all pts</th>
<th>0.0</th>
<th>1.0</th>
<th>2.0</th>
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<th>4.0</th>
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</table>

Figure 10.61 Maximum predicted gap and sliding displacement

Figure 10.62 Maximum absolute displacement in the principal direction of movement
**Damage concentration in side wall**

When shaken in Y-direction all the models experience high damage concentration in a horizontal strip on the side wall. In reality the strip does not have equal gap and sliding displacement. Due to the limit on the contour bar the whole strip turns magenta. Changing the levels of the contours from 0-10 mm to higher thresholds of 10-20 delivers a pattern of diagonal cracks at the height of the strip. These are the same type of cracks which were experienced by the fixed model, as explained in paragraph 10.5.3.

**Bracing + anchors – examination of damage area**

*Shaken in Y-direction*

![Figure 10.63 Maximum predicted gap and sliding displacement](image)

![Figure 10.64 Maximum predicted gap and sliding displacement adjusted contour thresholds](image)
Concentration of stiffness

In the models with anchors, the steel strips are placed diagonally with respect to the transverse walls. This configuration is intended to provide in-plane (shear) resistance to the connection. This configuration basically creates a truss adjacent to the wall. The ‘truss’ configuration creates a concentration of stiffness, and a sudden shift of stiffness in the façade. This causes damage concentration in the numerical models. The situation is comparable to the results in paragraph 10.5.4., where the stiffness concentration as caused by the nodal rigid bodies.

![Diagram of concentration of stiffness](image)

Effect of cracking on dynamic behaviour

In the model with planks and anchors, the absolute maximum deformation plot shows torsional displacement. After the first large acceleration peak at t=3.7s, the façade endures a horizontal tear occurring at t=3.7, causing the front façade to become extra weak with respect to the back façade. This enhanced asymmetry causes the torsional displacement. This event confirms the necessity of nonlinear material models. The propagation of cracks influence the stiffness (distribution) during the analysis and in this way the dynamic behaviour.

![Diagram of effect of cracking](image)
10.6.2 Comparison of base shear coefficient, diaphragm / storey drift

In the following section, the retrofitted models are compared based on base shear coefficient storey drifts and diaphragm drifts. Force-deflection curves are given in APPENDIX F.

**Base shear coefficient**

The base shear coefficient is defined as the lateral force measured at the base of the model, divided by the model weight. The base shear coefficient has not been (substantially) increased by the implementation of the retrofit measures. When shaken in X-direction, the base shear coefficient maintains ~0.45, where in Y-direction the value fluctuates around 0.4. This result can be corresponded to the fact that the damage is not necessarily lowered by the retrofits, but shifted towards the in-plane walls.
**Model displacement behaviour**

Secondly the displacements of the model are evaluated. Storey drift and diaphragm drift are defined as follows:

\[
\text{Storey disp in X-dir} = \text{disp} - \text{disp X} \\
\text{Diaphragm disp in X-dir} = \text{disp} \bullet - \text{disp} \cdot \\
\text{Storey disp in Y-dir} = \text{disp} - \text{disp Y} \\
\text{Diaphragm disp in Y-dir} = \text{disp} \bullet - \text{disp} \cdot
\]

Figure 10.67 Definition of diaphragm drift and storey drift (Source: Sathiparan, 2016)

For the measurements, it is chosen not to average nodes from opposite walls, as these walls of the flexible model can sometimes move in opposite directions. Furthermore it is chosen not to pick points at the corners, as it is seen in the prior chapter that the corners have a tendency to bulge outwards and demonstrate behaviour which is local and unrepresentative as a measurement for the global model. The measurement points are shown below.

Figure 10.68 Indication of nodes for measured displacements

**Note on drift limits**

The models see relatively high diaphragm or storey displacements: drifts of up to 4%. The wall thickness is relatively large with respect to the building height. This might explain why there is no out-of-plane toppling of walls, despite large drifts. Further research is needed to evaluate if the wall is actually capable of keeping stable, or if the collapse prevention is perhaps caused by an instant opposite thrust of the acceleration of the El Centro ground motions.
Storey displacement

It can be seen that the implementation of the retrofit measures causes the storey displacement to increase, since the model generates a more 3D (in-plane) response. In the Y-direction this increase in storey displacement is the largest, since the model is softened by the many in-plane cracks in the side walls.

Diaphragm displacement

The diaphragm displacement is drastically decreased after implementing the retrofit measures. For the unstrengthened model the diaphragm displacement in Y-direction exceeds $\frac{1}{3} \cdot t \approx 117\text{mm}$, for the retrofitted models, diaphragm displacement is decreased to values lower than 13 mm.

For a quick understanding of the diaphragm limit displacement, $\Delta_{D,X,max}$ or $\Delta_{D,Y,max} \leq \frac{1}{3} \cdot t$ is proposed by Arup (2014), based on preventing a horizontal out-of-plane bending mechanism. In this relationship, $t$ is the thickness of the considered wall under consideration.
PART IV: IMPLEMENTATION PROPOSAL

In the previous part, numerical analyses were done on simplified one-storey box models. However, the research is intended for 3 to 4 storied brick masonry buildings, as described in chapter 2. A strategy and approach are outlined for the integration of the retrofits into the architecture and construction of the full-sized traditional brick houses.

Chapter 11: Retrofit approach
11 RETROFIT APPROACH

11.1 Linking box to building

The analyses in chapter 9 and 10 are done on a one-storey box-model, whereas the typical vernacular houses are two stories in remote areas and 3 to 4 stories in urban areas. The box and building will have different response to earthquake motions as their periods are different.

However, it is believed that the basic conclusions regarding tying the building together will also hold for the full size building. In fact, a house with multiple stories will undergo larger out-of-plane displacements, causing higher risk of out-of-plane toppling of walls. For these buildings it is then even more important that the walls are tied to the floors. Where for the box-model the walls did not topple over, possibly due to the relatively thick wall for the limited height, the full size building gable walls at roof level could be more vulnerable.

Results of box – conclusions for full building

The main conclusions from chapter 10 are that the unstrengthened models are highly flexible and are excited in the primary modes (out-of-plane). The retrofits generate a more 3D response, however, the damage is rather replaced than reduced. The low-strength masonry is very flexible and weak, therefore the cracks occur in the in-plane walls at the fixation of the stiffened diaphragm. Tying the wall at floor levels causes concentrated stresses at these locations.

- Therefore more fixation points are suggested over the height of the wall, to spread the load more evenly over the in-plane wall.
- In traditional construction, since the building could not be made strong enough, it was chosen to make the building flexible (dissipating energy, and attracting less loads). Timber pegs, allowing some deformation, provide this flexibility to the building.
- It is suggested that more deformation capacity is required (in the method of modelling or the design) in the upgraded retrofit connections, as is seen in the traditional techniques. This is expected to lower the damage concentration at floor levels.
- If the masonry quality of the wall is too very low, (for example if there is very little coherency between wall wythes), it is judged that these retrofits are not sufficient.

Figure 11.1 Typical traditional brick masonry building, Bungamati (Source: own picture)
Box-action for thick walls

Newer houses in Holland have very slender walls. For lateral loads such as wind, the walls are basically supported at the top by concrete floors. Box-action is needed even to remain stable under wind loads. The Nepalese walls however, are more like thick cantilevering walls with floor joists lying on top. Most weight is concentrated in the load bearing masonry walls (refer to Appendix B2). The houses are generally low-rise, with low storey heights. The walls provide some of their own stability to resist overturning, due to their own thickness.

Figure 11.2 Comparing the structural system of masonry walls of narrow walls, to the thick Nepalese walls.
### 11.2 Overall upgrade approach

An overall upgrade approach is proposed, based on the analyses conclusions. Upgrading the connections and diaphragms is only sensible if the to-be-connected building elements are of sufficient capacity. Therefore firstly the initial condition of the building must be examined by experienced professionals. This evaluation can yield the following outcome strategies:

- If the connections of the house are lacking, but the masonry walls have sufficient interlocking and the timber elements of seismic provisions are not severely deteriorated by moisture, consider retrofit.
- However, if not only the connections, but also the basic building materials (masonry and timber elements) are in very bad state, retrofit might not be feasible. In this case intensive interventions are needed to hold the walls together. This would lead to high costs and a severe infringement of the identity of the building. Advice would be to rebuild the structure.

![Retrofit approach scheme](image)

**Figure 11.3 Retrofit approach scheme**

11.3 Plate anchor + diagonal timber bracing

The diaphragm is stiffened with timber diagonal bracing or timber planks. The anchors are executed with steel strips, on the exterior, timber slats are nailed to the strip as anchor plats.

- The steel strips themselves are applied as anchors, no welding is needed.
- To create a better spread of the load, additional timber vertical posts / strongbacks between stories are suggested. These strongbacks provide the opportunity to create additional wall anchors, and spread the transfer of load over the height of the wall.
- For the top floors, the end-columns of the Dalan frames can be used as vertical posts for the additional anchors.
- The connections can be made less stiff if some tolerance is applied between the exterior anchors and the masonry wall. This slight distance between the timber surface and masonry, also contributes to preventing deterioration due to moisture.
- If the steel strips are nailed horizontally to the diaphragm, the timber exterior slats are aligned with the horizontal bed joints. However, if the strip is rotated, the anchor slats are oriented vertically, the slats trespass multiple horizontal masonry bed-joints, providing higher anchorage capacity. However, the rotated steel strip will have little in-plane resistance.
- With a simple operation of placing a horizontal timber beam, the roof beams can be turned into an A-truss. This will provide the roof diaphragm with more in-place stiffness. However, one should take caution not to place the transversal beam to low, which would interfere with the passage of the space.

Figure 11.4 Overview of retrofit measures plate anchor and diagonal bracing (Adapted from: Danish architects, 1968)
(a) Steel strips fixed under the bracing  
(b) A vertical plate anchor is preferred, perpendicular to the head joints

Figure 11.5 Timber slats nailed on both sides to the steel strips (Source: own image)

(a) Fixation of the gable wall  
(b) Vertical posts along masonry walls

Figure 11.7 Vertical posts provide extra anchorage to the walls
11.4 Horizontal bandage + concrete overlay option

For the implementation of the horizontal bandage and concrete floor overlay.

- Some traditional houses have brick ridge decoration, or a plastered ridge bands on the façade at floor levels. These exterior decorative bands can be used to integrate the bandage into the appearance of the façade.
- For a better spread of the loads, several vertical mesh strips can be placed on the interior, spanning from floor level to floor level, providing possibility of anchoring along the height of the wall.
- Typically the staircase is placed adjacent to the side walls. This means that the aperture in the diaphragm lies besides the wall, making it impossible to tie the diaphragm to the wall here. The advantage of the bandage option is that the wall is tied by a seismic belt on the exterior.
- Concrete beam at roof level. For the wall-to-roof connection a concrete band can be placed on top of the wall supporting the main roof beams. This area is hard to excess from the exterior to place a horizontal bandage. A wall-to-roof connection can be laid into the in-situ concrete ring beam. The seismic band/ diaphragm chord will contribute to holding the walls together.
11.5 Retrofit of multiple houses (in array)

Houses in dense areas are typically built in array, or around a courtyard. Several suggestions are done with respect to upscaling or collaboration of retrofit:

- **Multiple retrofit at once**
  As in the dense cities multiple houses are in need of retrofit, it could be profitable to retrofit multiple houses at once, as the tools and workers are on location, as well as the bamboo scaffolding system.

- **Fixation party walls – mutual stability**
  The side-walls can be fixed as both wall-exteriors are indoor; the timber features can be placed on both sides, without being affected by the moisture.

- **Ongoing bandage for row-housing**
  As the Newari building style incorporated horizontal alignment of the storey heights of adjacent houses, this could facilitate a multi-retrofit approach for row-housing, using the bandage system.

![Figure 11.9 Fixation of strongbacks on multiple sides of unsupported party walls](image1)

![Figure 11.10 Ongoing bandage applied to multiple houses in array (Source: own pictures)](image2)
11.6 Demount and rebuild

The advantages of demounting and rebuilding are as follows:

- A new and integral resilient structure can be built, in contrast to patch working with retrofit interventions. The interventions risk interfering with the architecture of the house.
- The vernacular houses are quite old, some are over 200 years. The houses are generally not equipped with basic modern bathrooms or lighting. Furthermore, as people grow taller, the typical storey heights of 1.8 to 2.4 m are very low. For new construction higher storey levels and modern services can be implemented.

There are basically two options for rebuilding: traditional or modern construction style. These terms refer respectively to building with brick masonry and timber provisions or reinforced concrete frames with masonry infill.

Rebuilding in traditional style

Several experts are convinced that the traditional way of building is sufficiently earthquake resilient\(^1\), when limiting the building height and incorporating appropriate seismic provisions. However, the seismic performance of vernacular houses is seriously affected by bad maintenance and deterioration of its timber features. During the devastating earthquake of 1934, more than 200,000 structures collapsed (Marahatta, n.d.). Due to hasty rebuilding and omission of essential seismic provisions (such as shear locks) after the earthquake in 1934, widespread damage was caused in the Gorkha earthquake of 2015. It is observed that well-maintained and well-constructed temples and palaces, including seismic timber bands, have survived large earthquakes. Maintenance and treatment of timber elements are therefore essential for this building method.

The advantages of building in traditional style are:

- Rebuilding cultural heritage and restoring traditional knowledge
- Creating employment for local wood-workers
- Materials are mostly re-usable, since the bricks are bound by mud-mortar and timber elements are fixed with wedged connections.

The most important seismic provisions are:

- Regular plans and configurations
- Maximum building height of three stories
- Horizontal timber bands and timber framing of windows elements
- Wedged connections allowing some deformation: shear locks, wedges, pinned columns

The model houses of the Namuna Ghaun project, by Rabindra Puri\(^2\) are good examples on how to properly rebuild in traditional style (Figure 11.11).

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\(^1\) Prem Nath Maskey, Sudarsan Raj Tiwari
There are several additional provisions or modifications to the traditional building style which can be incorporated to improve the resiliency, as suggested by Nienhuys (2003);

- Replace the layer of mud by composite concrete floor or water-proof timer plywood, which reduces the seismic mass.
- Increase the coherency of wall wythes with through-stones and through wall-ties.
- Use all fired bricks throughout the whole wall (to reduce delamination risk between outer brick façade and inner adobe wall).
- Apply perforated or hollow bricks to reduce the weight of the walls.
- Better fixation method of the roof tiles.

Rebuilding in modern style

Reinforced concrete frames with masonry infill are referred to as ‘modern’ construction style. The advantages of rebuilding in this way are:

- Reinforced concrete is the main building method used for reconstruction
- People generally have more faith in the earthquake resistance of this modern construction method.
- If concrete framing is applied, less bricks (material) are required, as the masonry is not intended to load-bearing.
- Higher buildings can be made, with higher storey heights.

Main threats:

- Adequate (seismic) concrete detailing is essential to its structural performance. Ill-executed concrete details can result in highly vulnerable buildings.
- It is very hard to check afterwards if detailing is executed correctly, as the reinforcement is covered.
- If the concrete structure is damaged, it is very hard to re-use materials. In remote areas, it will also be very hard to demolish the building and dispose the building materials.
12 CONCLUSIONS AND RECOMMENDATIONS

The conclusion section outlines the main findings, followed by recommendations for further research.

12.1 Conclusions

The traditional brick masonry houses in the Kathmandu valley are vulnerable to seismic excitations, due to the low-strength masonry and the lack of sufficient connections. However, their inherent design and symmetrical configuration does show abidance to seismic principles. Tying the building elements together could enhance the global seismic performance.

The first objective of the research was to explore suitable retrofit measures, considering the specific circumstances of Nepal. The second aim was to study the effect of the retrofit measures on the seismic performance of a typical low-strength masonry structure.

Connection and diaphragm upgrading

The floors of the vernacular houses exist of timber joists, planks and mud, without any nailed connections. Only improving wall-to-floor connections would effectively mean tying the wall to one single beam. Therefore the retrofit strategy should be a combination of both improving the connections as stiffening and strengthening of the diaphragm.

For the evaluation of suitable retrofit measures for the Nepalse situation, six design- and construction criteria are drawn up: structural efficiency, impact on architecture, durability, constructability, cost-effectiveness and material availability. Comparing the criteria to those used for retrofit in the Netherlands (Arup, 2015a), it is noted that the criteria “impact during construction” (discomfort) and “duration” are switched to “cost-effectiveness” and “material availability” for Nepal.

A list of existing retrofit measures for connections and diaphragms is evaluated by giving them scores per criterion: unfavourable, neutral or favourable. A distinction is made between solutions for urban and remote areas, due to difference with respect to availability of materials, equipment and accessibility. Two combinations of measures with the best overall scores are proposed as suitable retrofit strategies:

- **Retrofit strategy remote areas**: Timber planks are nailed to the joists as diagonal floor bracing. Wall ties are made with simple steel strips, Timber slats are nailed to the steel strips on the outside of the wall, to serve as plate anchors.
- **Retrofit strategy urban areas**: A horizontal bandage is placed at the height of the floors to tie the walls together. The bandage is combined with a thin reinforced concrete floor overlay. Reinforcement rods laid into the wet in-situ concrete serve as wall-ties, connecting the diaphragm to the exterior bandage.

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23 Remote areas are defined here as settlements in the mountains with very low accessibility.
Numerical analyses

Numerical analyses are conducted to study the effect of the proposed retrofit measures on the seismic performance of a low-strength masonry structure. A smeared crack material model which is newly developed by Arup is used to model the masonry walls. Non-linear time history analyses are conducted, making use of LS-DYNA software.

Comparison numerical model to (scaled) shake table test from literature
A simple numerical model is compared to the results of an experimental shake table testing campaign on a scaled low-strength masonry structure by Sathiparan (2016). Two models are compared: one box model without a roof and one box-model with a timber roof diaphragm. The goal is to evaluate the capacity of the chosen numerical modelling strategies to capture the seismic behaviour of low-strength masonry.

The smeared crack approach with shell elements for masonry walls was initially chosen because of its proclaimed computational and modelling speed. However, the scaled model has several small (and stiff) elements. In the explicit analysis, the time step is governed by the Courant condition. Therefore the scaled simple model still requires 10 hours of calculation time while making use of 20 Central Processing Units.

Damage propagation
The numerical models reproduce the main cracks which are seen in the experimental models. For the roofless model these are: debonding of lintel and masonry, vertical cracks at the corners of the lintels, weakening of the connection between perpendicular walls and out-of-plane modes in the side walls. In both numerical and experimental model the spandrels of the in-plane walls topple out-of-the wall, leaving the side walls as two out-of-plane cantilevers. This situation rapidly leads to collapse. In the experimental model also a wall pocket is toppled out-of-plane; this is not captured by the numerical model.

The model with roof has alternative damage patterns; there is damage concentration at the line of fixation with the roof diaphragm, and there are diagonal cracks propagating from the window corners to the outer edge of the façade. Again the spandrels topple out- causing the roof to be carried by two out-of-plane walls. For both the numerical and experimental model the structure with roof experiences less out-of-plane damage and it collapses at a later stage than the roofless model.

Time of collapse
The explicit analysis with LS-DYNA is well capable of modelling masonry structure until collapse. The main collapse mechanisms are reproduced. The numerical collapse does not occur simultaneously, but it does lie within reasonably similar range as the numerical model. For the model without roof the model reaches 41 out of 43 cyclic runs, the model with roof collapses at run 43/45.

Force displacement behaviour
In the numerical analysis the maximum base shear coefficient of the experimental test is not reached: apart from an outlier which lies around 0,73, the main data points reach a coefficient of 0,5 which is approximately 60% of the base shear coefficient reached in the experimental model (0,81). The roof displacement data shows that the numerical model behaves initially too stiff. Later on, the corners of the numerical model see too much displacement. In magnified displacement plots the masonry model shows an extreme local (numerical?) bulging out of the corners.

24 Refer to subsection 5.2.2, paragraph non-linear time history analysis for a description of explicit analysis and the Courant condition.
Sensitivity studies
Several sensitivity studies are done in which the masonry material properties are changed. For calibration of the numerical model, wall component tests would be preferable, but these were not available. The studies show that changing the Young’s modulus values to \( \sim 0.3 \text{GPa} \) provide a better matching lateral displacement. Via the Young’s modulus, actually the Shear modulus is decreased, which is generally lower for anisotropic masonry than the initial default relationship for isotropic materials.

The mortar bond strength values are increased with 200%. The experimental values better approach the numerical in base shear coefficient, but the crack patterns become less similar. Therefore this alteration is dismissed. For the diagonal shear value, a higher value (40 kPa) than calibrated for the Netherlands retrieves better results. The recommended value for the masonry model is a percentage of the shear bond strength; taking a percentage of low-strength masonry bond would provide an unrealistically low value.

Main conclusion of the comparisons is that the model can capture the overall damage propagation quite well, but it can’t perfectly simulate the actual strength and stiffness. This performance is sufficient for comparing alternative retrofit models in this research, but not for estimating an exact model capacity.

Simulation of retrofit measures on a (full scale) box-model
A simple one storey box-model with comparable dimensions and characteristics to one bay of a vernacular house is modelled to simulate the effect of the proposed retrofit measures. The fixed-base models are loaded sequentially in X- and in Y-direction with El Centro motions which are scaled to 0.5g. Firstly the unstrengthened situation is compared to a model with a fully connected, stiff diaphragm. Secondly the retrofit methods proposed in PART II are modelled into the case study.

Comparing unstrengthened versus stiff diaphragm and fixed connections
A gravity analysis shows that the floor joists experience too much deflection (15.6 mm > 0.004L) under gravity loads. The excessive mass of the layer of clay can be removed and replaced by a lighter alternative floor topping.

An eigenvalue analysis is performed on the unstrengthened model and the model with stiff floor and fixed connections. The analysis is converted to implicit and the materials to elastic. The unstrengthened model is excited in a large amount of modal shapes (>38). The first modes engaging >50% of mass are out-of-plane bending of the longitudinal and transversal wall. Modal analysis of the model with stiff diaphragm and fixed connections shows the model generates a 3D response, with only 3 main modes of vibration. In-plane longitudinal bending, in-plane transversal bending, and torsion (as the façade openings are not symmetrical).

The two opposite extremes are then compared in dynamic analyses. The unstrengthened model is mainly excited in out-of-plane modes. The displacement plots show that the walls bulge out-of-plane independently of the rest of the structure. This movement causes large vertical cracks at the corners and at the corners of wall openings. The model with stiff diaphragm and roof develops a 3D in-plane building response. The displacement plots show a parallel vertical gradient over the height, indicating global response. At the line of wall-to-floor connections the strengthened model sees localized damage. This indicates the risk in making too stiff connections for flexible structures. Variation studies are done to evaluate the necessity of using friction beam connections. Damage plots show a clear concentration of stresses where the stiff connections are placed.
The model seems less affected by the earthquake (in terms of crack patterns as storey displacement) when excited in X-direction, although in this direction the façade has wall-openings. A side study is performed to evaluate if the potential cause could be the lintel modelling method, acting as an in-plane tie stiffening the masonry. An alternative modelling method with friction beams between masonry and lintel beam does not show significant difference in lateral displacement behaviour or crack patterns.

Comparison of the model displacement throughout the whole analysis shows that the occurrence of cracks has a large influence on the dynamic behaviour of the model. The large diagonal cracks in the solid wall cause the building to behave more flexible in this direction after first cracking. The influence of model stiffness degradation confirms the necessity of nonlinear analyses.

**Simulation of retrofit measures**

Four retrofit alternatives are modelled. Results in terms of damage plots and displacements are quite similar. Outcome of the numerical analyses shows that damage is not necessarily reduced, but changed to in-plane (bed joint sliding). The amount of damage does not seem to be significantly decreased, but relocated. It is suggested that if the masonry is of very low quality of vulnerable to delamination, the retrofit of connections and diaphragms only might not be sufficient.

The maximum base shear coefficients show that the strength of the models is barely increased, staying around 0,45 when shaken in X- and 0,4 when shaken in Y-direction. The storey displacements increase for the retrofitted models. Shaken in X-direction the increase is approximately 2x, (11,7 to ~20mm) and in Y-direction 3,5x, from 26,3 to values around 90mm. This is expected as a more 3D response is generated. The diaphragm displacement drastically decreases as the diaphragm is stiffened, from 97,5 in X and 120,2 in Y-direction to values around 10 mm.

For the retrofitted models which now portray a more global response, severe diagonal cracks are observed in the in-plane transversal walls. A concentrated area of cracking is observed halfway the wall.

**Retrofit approach**

A retrofit strategy is proposed, in which an extensive check of the initial house condition is incorporated. If masonry walls are of decent quality, but only the connections are lacking, the retrofit can be feasible. If not, one may consider rebuilding the houses either in traditional or modern construction style.

The vernacular houses can have adequate seismic performance up to 3 stories, if the important seismic provisions are included (such as horizontal seismic bands, lintel framing, wooden pegs as shear locks and symmetrical configurations). It is essential that the timber elements are not deteriorated by moisture. Concrete frame structures offer the opportunity to build higher houses and higher storey levels. Adequate concrete detailing is essential for earthquake resistance. Brick masonry façades can still provide a traditional look for the concrete frame buildings.

When chosen for retrofit, it is suggested that the load can be better distributed over the wall than only at floor level. This can be done by placing vertical timber strongbacks against the walls, or vertical mesh strips for the bandage option.

The retrofits should be thoughtfully integrated into the exterior of the façades. The timber anchor plate can be carved to match the existing decorated timber features such as lintels and doors. The horizontal bandage can be designed as a decorative ridge at storey level, which is also not uncommon for these traditional buildings.
Vernacular houses in urban areas are mostly built in dense clusters: in array, or courtyard configuration. Implementing several retrofits at once has practical benefits such as re-use of bamboo scaffolding, connecting interior party walls and making an ongoing horizontal bandage. Last is possible due to perfect horizontal alignment of storeys.

12.2 Recommendations for further research

People have been building with low-strength masonry for ages in highly seismic areas. Although the traditional masonry structures are widespread and widely inhabited, the knowledge on how these structures behave during earthquakes is far from sufficient. The experimental and numerical research on improving existing low-strength masonry houses is very limited. Especially when compared to the knowledge on high-tech structures such as steel and concrete. This is aggravating, as many houses and their inhabitants are at great risk.

The low-tech masonry structures require the most state of the art technology for seismic assessment. The seismic behavior of low-strength masonry is very complicated. Experimental component tests of mud-bound brick masonry walls, on the mortar bond strengths and simple structures will help us improve and validate our numerical modelling techniques. Using MAT_SHELL_MASONRY smeared crack model for low-strength masonry allows relatively quick modelling and computational speed. This research has shown that the main crack patterns and failure mechanisms can roughly be reproduced. By means of calibration to component tests the material model could be further enhanced. However, the mesh model can’t capture delamination of wall wythes. Since many thick masonry walls are expected to have little interlocking between the leaves, this is a serious shortcoming. Explicit modelling of bricks by means of AEM could provide a better solution.

It would be useful is there were more accessible tools for everyday engineers to better assess the seismic performance of masonry. It is a fact that only a very small group of people has access to high computational power to be able to perform the long-duration nonlinear time history analyses.

For masonry buildings with box-action simpler methods are available: TREMURI offers an equivalent frame modelling approach, using macro-elements to model the masonry. TREMURI provides the opportunity to perform nonlinear static and dynamic analyses of complete buildings while offering a relatively easy understanding of the analysis results. Such accessible methods should be developed for masonry without box-action, methods which are able to capture out-of-plane and MDOF behaviour.

Overall, more research is needed on masonry buildings without box-behaviour. Most seismic calculation methods are developed for SDOF behaviour. For these flexible structures it is not straightforward how to define a characteristic node to evaluate the force-displacement behaviour, since the walls behave independently out-of-plane. Alternative methods should be developed to evaluate the response of these buildings.

This research has shown that retrofits on connections and diaphragms contribute to shifting the damage from out-of-plane to in-plane cracks. More experimental research is needed on the seismic/cyclic behaviour of connections and diaphragms. This data serves as invaluable input for the nonlinear modelling of connections. Better understanding will help us better simulate the benefits of ductile connections and better approach the actual building behaviour. It is acknowledged that fully stiff diaphragms and connections are not optimal or feasible, therefore it would be useful to evaluate how much deformation capacity and ductility is desired in the connections. Whereas this research has mostly focussed on the masonry material, a next research could focus more on what is actually happening in the connections during the analysis.
In the traditional brick masonry houses, timber features are essential seismic provisions. However, timber is vulnerable to deterioration by moisture. As hardwood is becoming more and more scarce in Nepal. Research is needed on simple techniques to treat softwood and bamboo and enhance the durability to make the materials feasible for retrofit.

For this research a simplified one-storey structure is analysed. It is recommended to evaluate the effect of upgrading connections and stiffening of diaphragms on a full-sized 2-4 storey vernacular house in order to capture the effects associated with full buildings, such as gable wall failure.

By optimizing and verifying our modelling techniques we will learn more about the seismic behaviour of vernacular structures and hopefully contribute to making the world a safer place. Brick by brick.

Figure 12.1 Traditional brick masonry houses in Bungamati, Nepal (Source: own picture)
Arup. (2015a). Modelling masonry walls with MAT_SHELL_MASONRY.
Bonpace, C., & Sestini, V. (2003). Traditional materials and construction technologies used in the Kathmandu Valley. UNESCO.


Maffei, J., Bazzurro, P., Marrow, J., & Goretti, A. (n.d.). Recent Italian Earthquakes: Examination of Structural Vulnerability, Damage and Post-Earthquake Practises - Case Studies and Comparisons to U.S. Practice. Oakland, CA.


### APPENDIX A PERCEIVED EARTHQUAKE BEHAVIOR

#### A1 Seismic deficiencies and resilient features
For seismic retrofit it is important to assess the seismic performance of the building as a whole. An overview is made of the seismic deficiencies and earthquake resilient features per building aspect. The table is based on texts of D’Ayala and Bajracharya, and Maskey (2014)

<table>
<thead>
<tr>
<th>Aspect/element</th>
<th>Description</th>
<th>(Seismic) Deficiency</th>
<th>(Seismic) Resiliency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction</td>
<td>Maintenance</td>
<td>Generally old buildings. Over the years poor maintenance has caused deterioration: rotting of timber and decay of mortar</td>
<td>Masonry is quite durable building material not vulnerable to corrosion.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arrangement of buildings</td>
<td>Stand alone</td>
<td>Little redundancy.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Array/joined</td>
<td>Risk of pounding between adjacent houses.</td>
<td>Stabilizing effect of houses in a row? More redundancy</td>
</tr>
<tr>
<td></td>
<td>Around courtyard</td>
<td>Unreinforced re-entrant corners</td>
<td>Stabilizing effect of houses in a row? More redundancy</td>
</tr>
<tr>
<td>Configuration</td>
<td>Plan and elevation</td>
<td>Buildings are sometimes narrow/slender of plan</td>
<td>Buildings are mostly simple and rectangular plan/ elevation – minimizing torsional forces</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plans have a limited length-to-width ratio: mostly L&lt;2B</td>
</tr>
<tr>
<td>Lateral load path</td>
<td></td>
<td>*Refer to connections</td>
<td>Continuous load path for lateral forces to foundation, no cantilevers, stopping columns</td>
</tr>
<tr>
<td>Load distribution</td>
<td></td>
<td>Excessive mass due to topping layers of mud on floors and roof. Excessive live-loads caused by water tanks on the roof.</td>
<td>Decreasing load towards the top: decrease in wall thickness, replacement of wall to timber column frame</td>
</tr>
<tr>
<td>Stories</td>
<td>Amount</td>
<td>Heavy modern additional 4th stories built on top of existing structures</td>
<td>Mostly 3 stories high</td>
</tr>
<tr>
<td></td>
<td>-height</td>
<td>Low storey height 1.8-2.4, max 2.5 m</td>
<td></td>
</tr>
<tr>
<td>Foundation</td>
<td></td>
<td>No-damp proofing layer</td>
<td></td>
</tr>
<tr>
<td>Wall</td>
<td>Thickness</td>
<td>Thick walls generate large out-of-plane forces</td>
<td>Thick walls (45 cm or more) Decrease (minimize force) Shear resistance at the bottom</td>
</tr>
<tr>
<td></td>
<td>Material</td>
<td>(Sun-dried) bricks are brittle and have very low tensile strength. Low quality of mortar, low bonding between mortar and bricks.</td>
<td>The mud-mortar gives the walls some flexibility – and ductility: dissipating energy through frictional sliding</td>
</tr>
<tr>
<td></td>
<td>Coherency</td>
<td>Insufficient bond between outer leaves (wall wythes) and rubble infill can cause delamination of multi-layered wall. Due to difference in brick use and stiffness of the outer shell, differential settlement can cause bulging out of the outer leaf.</td>
<td>Rarely: parallel timber bands embedded in the walls with intermediate connectors to tie the wall leaves together</td>
</tr>
<tr>
<td></td>
<td>Corners</td>
<td>Insufficient connection between cross walls: often simply butt-jointed with no or little interlocking.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dalan frame</td>
<td>The open timber colonnade has less stiffness than upper stories posing the risk of soft-storey mechanism.</td>
<td></td>
</tr>
<tr>
<td>Wall-to-floor</td>
<td>Introduction of load</td>
<td>Plate (beam) embedded in the wall too evenly spread the load of the floor joists in the wall.</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX A Perceived earthquake behavior

<table>
<thead>
<tr>
<th>Connection</th>
<th>Lack of adequate wall-to-floor connection; the floor joist is slid into the wall and out-of-plane connection is purely shear (friction) between joist- and wall. Parallel to the joist there is no connection at all.</th>
<th>Sometimes: wooden pegs pierce through floor joists to anchor the wall to the floor. (inside and/or outside of the wall) However, these connections can be in bad condition: not able to withstand the horizontal forces</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall-to-roof</td>
<td>Lack of adequate wall-to-roof connection; the roof joist simply rests on the wall and out-of-plane connection is purely shear (friction) between joist- and wall. Parallel to the joist there is no connection at all.</td>
<td></td>
</tr>
<tr>
<td>Floor</td>
<td>Diaphragm</td>
<td>Flexible floor diaphragm – not providing box-action</td>
</tr>
<tr>
<td></td>
<td>Topping</td>
<td>Heavy ballast layer of mud – creating greater inertia forces in the building. No structural contribution to the floor span.</td>
</tr>
<tr>
<td>Roof</td>
<td>Diaphragm</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Topping</td>
<td>Heavy ballast layer of mud – creating greater inertia forces in the building. Topping with heavy roof tiles.</td>
</tr>
<tr>
<td></td>
<td>Overhang</td>
<td>Overhanging roof generating extra weight on top.</td>
</tr>
<tr>
<td>Windows</td>
<td>Size / amount</td>
<td>The open timber colonnade has less stiffness than upper stories posing the risk of soft-storey mechanism.</td>
</tr>
<tr>
<td>Boxing</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lattice grid</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### A2 Seismic vulnerability

The vulnerability of traditional brick masonry buildings in Nepal can vary due to factors such as quality of workmanship, quality of materials and amount of decay. A well-built and maintained building is estimated by world-housing.net to have a vulnerability rating of D, medium vulnerability, whereas a poorly built and- or poorly maintained building is estimated as class B, highly vulnerable.

<table>
<thead>
<tr>
<th>Seismic vulnerability class</th>
<th>High vulnerability</th>
<th>Medium vulnerability</th>
<th>Low vulnerability</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
</tr>
</tbody>
</table>

Table A.1 Vulnerability of traditional brick masonry housing Nepal (Source: http://db.world-housing.net/building/99)

It must be noted that these ratings are based on well-educated though subjective assessments. For information on how the vulnerability rating is set-up, consult the seismic vulnerability guidelines document from http://db.world-housing.net/.
APPENDIX B  LATERAL FORCE ANALYSIS

B1  Parameters

Typical building dimensions which are used for the calculation are stated below.

General building dimensions

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building type</td>
<td>House</td>
</tr>
<tr>
<td>No. of stories</td>
<td>4</td>
</tr>
<tr>
<td>Wall</td>
<td>unreinforced masonry, brick in mud-mortar</td>
</tr>
<tr>
<td>Floor/Roof</td>
<td>Timber floor joists – addition of mud</td>
</tr>
<tr>
<td>Building dimensions</td>
<td>10 m x 6,5 m</td>
</tr>
<tr>
<td>Earthquake zone</td>
<td>1 (NBC 105)</td>
</tr>
<tr>
<td>Importance factor</td>
<td>1 (Residential building)</td>
</tr>
</tbody>
</table>

Building drawings
**B2  Seismic weight**

The NBC 102:1994 refers to the Indian Code IS:875 for the unit weight of materials. For the main building materials parameters applied in the report of Disaster risk Management for the historic City of Patan (2012) are used.

<table>
<thead>
<tr>
<th>Element</th>
<th>Density $\rho$ [kg/m³]</th>
<th>Density $\rho$ [kN/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adobe brick wall</td>
<td>1800</td>
<td>18,0</td>
</tr>
<tr>
<td>Wood</td>
<td>700</td>
<td>7,0</td>
</tr>
<tr>
<td>Mud</td>
<td>1700</td>
<td>17,0</td>
</tr>
<tr>
<td>Roof tiles (terra cotta)</td>
<td>0,65</td>
<td></td>
</tr>
</tbody>
</table>

Table B.1 Density of the main materials

The design live loads Nepal National Building code NBC 103 : 1994 Occupancy load (Imposed load) refers to the Indian Standards IS: 875 (Part 2) -1987. The seismic live loads are approximated as a percentage of the design live loads. Up to 3kPa (=kN/m²) a percentage is taken of 25%. For residential buildings:

<table>
<thead>
<tr>
<th>Design Live Load</th>
<th>Uniformly distributed [kN/m³]</th>
<th>For Seismic [kN/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor live load</td>
<td>2,0</td>
<td>0,5</td>
</tr>
<tr>
<td>Roof live load</td>
<td>1,5</td>
<td>0,375</td>
</tr>
</tbody>
</table>

The mass of the ground floor as well as half of the height of the walls at ground floor level are left out of calculation, since it is assumed that these loads are transferred directly to the foundation.

**Calculation of floor loads**

<table>
<thead>
<tr>
<th>Dead load per floor</th>
<th>Dimensions [m]</th>
<th>Spacing [m]</th>
<th>Density $\rho$ [kN/m³]</th>
<th>Unit weight $\rho$ [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor joists</td>
<td>0,1x0,1 m</td>
<td>0,3</td>
<td>7,0</td>
<td>0,23</td>
</tr>
<tr>
<td>Layer of planks</td>
<td>0,02</td>
<td></td>
<td>7,0</td>
<td>0,14</td>
</tr>
<tr>
<td>Layer of mud</td>
<td>0,1</td>
<td></td>
<td>17,0</td>
<td>1,70</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>2,07</strong></td>
</tr>
</tbody>
</table>

**Calculation of roof loads**

<table>
<thead>
<tr>
<th>Dead load per floor</th>
<th>Dimensions [m]</th>
<th>Spacing [m]</th>
<th>Density $\rho$ [kN/m³]</th>
<th>Unit weight $\rho$ [kN/m²]</th>
<th>Slanted roof [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof rafters</td>
<td>0,12x0,12</td>
<td>1,5</td>
<td>7,0</td>
<td>0,067</td>
<td>0,08</td>
</tr>
<tr>
<td>Roof beams</td>
<td>0,12x0,12</td>
<td>1,75</td>
<td>7,0</td>
<td>0,058</td>
<td>0,07</td>
</tr>
<tr>
<td>Purlins</td>
<td>0,07x0,07</td>
<td>0,25</td>
<td>7,0</td>
<td>0,14</td>
<td>0,16</td>
</tr>
<tr>
<td>Planks</td>
<td>0,02</td>
<td></td>
<td>7,0</td>
<td>0,14</td>
<td>0,16</td>
</tr>
<tr>
<td>Mud</td>
<td>0,15</td>
<td></td>
<td>17,0</td>
<td>2,55</td>
<td>2,94</td>
</tr>
<tr>
<td>Tiles</td>
<td></td>
<td></td>
<td></td>
<td>0,65</td>
<td>0,75</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>3,60</strong></td>
<td><strong>4,16</strong></td>
</tr>
</tbody>
</table>
# Wall loads

<table>
<thead>
<tr>
<th>Dead load walls</th>
<th>Dimensions [m]</th>
<th>Density [kN/m(^3)]</th>
<th>Unit weight [kN/m(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Walls internal</td>
<td>0,45</td>
<td>18,0</td>
<td>8,1</td>
</tr>
<tr>
<td>Walls external</td>
<td>0,45</td>
<td>18,0</td>
<td>08,1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wall openings</th>
<th>H [m]</th>
<th>B [m]</th>
<th>A [m(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Door</td>
<td>1,7</td>
<td>0,9</td>
<td>1,53</td>
</tr>
<tr>
<td>Window small</td>
<td>0,8</td>
<td>0,7</td>
<td>0,56</td>
</tr>
<tr>
<td>Window large</td>
<td>1,5</td>
<td>0,9</td>
<td>1,35</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Wall</th>
<th>Dimensions [mxm]</th>
<th>Gross surface [m(^2)]</th>
<th>Net surface [m(^2)]</th>
<th>50% [m(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Façade GF</td>
<td>10x2,2</td>
<td>22</td>
<td>17,82</td>
<td>8,91</td>
</tr>
<tr>
<td>Façade 1-2</td>
<td>10x2,2</td>
<td>22</td>
<td>16,60</td>
<td>8,30</td>
</tr>
<tr>
<td>Façade 3</td>
<td>10x0,6</td>
<td>6</td>
<td>6,00</td>
<td>3,00</td>
</tr>
<tr>
<td>Side wall GF-1</td>
<td>6,5*2,2</td>
<td>14,30</td>
<td>14,30</td>
<td>7,15</td>
</tr>
<tr>
<td>Side wall 2</td>
<td>6,5x2,2</td>
<td>14,30</td>
<td>11,60</td>
<td>5,80</td>
</tr>
<tr>
<td>Gable wall</td>
<td>6,5x0,6+0,5x6,5x2,2</td>
<td>11,05</td>
<td>11,05</td>
<td>5,53</td>
</tr>
<tr>
<td>Spinal wall GF-1</td>
<td>9,1x2,2</td>
<td>20,02</td>
<td>16,96</td>
<td>8,48</td>
</tr>
</tbody>
</table>

## Summary of lumped load calculation

<table>
<thead>
<tr>
<th>Storey</th>
<th>Dead load [kN]</th>
<th>25% of Live Load [kN]</th>
<th>Total Wi [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>448,7</td>
<td></td>
<td>448,7</td>
</tr>
<tr>
<td>3</td>
<td>662,5</td>
<td>27,5</td>
<td>513,9</td>
</tr>
<tr>
<td>2</td>
<td>486,4</td>
<td>27,5</td>
<td>690,0</td>
</tr>
<tr>
<td>1</td>
<td>762,0</td>
<td>27,5</td>
<td>789,5</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>2442,1</td>
</tr>
</tbody>
</table>
Figure B.1 Lumped masses
## LUMPED MASS CALC

<table>
<thead>
<tr>
<th>part</th>
<th>description</th>
<th>Wt. [kN/m²]</th>
<th>area (m²)</th>
<th>Weight [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>First lump</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GF</td>
<td>façades</td>
<td>8,1</td>
<td>17,82</td>
<td>144,34</td>
</tr>
<tr>
<td>1F</td>
<td>façades</td>
<td>8,1</td>
<td>16,6</td>
<td>134,46</td>
</tr>
<tr>
<td>GF</td>
<td>side wall</td>
<td>8,1</td>
<td>14,3</td>
<td>115,83</td>
</tr>
<tr>
<td>1F</td>
<td>side wall</td>
<td>8,1</td>
<td>14,3</td>
<td>115,83</td>
</tr>
<tr>
<td>GF</td>
<td>spinal wall</td>
<td>8,1</td>
<td>8,48</td>
<td>68,69</td>
</tr>
<tr>
<td>1F</td>
<td>spinal wall</td>
<td>8,1</td>
<td>8,48</td>
<td>68,69</td>
</tr>
<tr>
<td>1F</td>
<td>Floor slab</td>
<td>2,07</td>
<td>6,05 x 9,1</td>
<td>105,66</td>
</tr>
<tr>
<td>1F</td>
<td>Live load</td>
<td>0,50</td>
<td>6,05 x 9,1</td>
<td>25,48</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>778,98</strong></td>
</tr>
<tr>
<td><strong>Second lump</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1F</td>
<td>façades</td>
<td>8,1</td>
<td>16,6</td>
<td>134,46</td>
</tr>
<tr>
<td>2F</td>
<td>façades</td>
<td>8,1</td>
<td>16,6</td>
<td>134,46</td>
</tr>
<tr>
<td>1F</td>
<td>side wall</td>
<td>8,1</td>
<td>11,6</td>
<td>93,96</td>
</tr>
<tr>
<td>2F</td>
<td>side wall</td>
<td>8,1</td>
<td>11,6</td>
<td>93,96</td>
</tr>
<tr>
<td>1F</td>
<td>spinal wall</td>
<td>8,1</td>
<td>8,48</td>
<td>68,69</td>
</tr>
<tr>
<td>2F</td>
<td>dalan</td>
<td></td>
<td></td>
<td>0,93</td>
</tr>
<tr>
<td>2F</td>
<td>Floor slab</td>
<td>2,07</td>
<td>6,05 x 9,1</td>
<td>105,66</td>
</tr>
<tr>
<td>2F</td>
<td>Live load</td>
<td>0,50</td>
<td>6,05 x 9,1</td>
<td>25,48</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>679,47</strong></td>
</tr>
<tr>
<td><strong>Third lump</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2F</td>
<td>façades</td>
<td>8,1</td>
<td>16,6</td>
<td>134,46</td>
</tr>
<tr>
<td>3F</td>
<td>façades</td>
<td>8,1</td>
<td>6</td>
<td>48,60</td>
</tr>
<tr>
<td>2F</td>
<td>side wall</td>
<td>8,1</td>
<td>11,05</td>
<td>89,51</td>
</tr>
<tr>
<td>3F</td>
<td>side wall</td>
<td>8,1</td>
<td>11,05</td>
<td>89,51</td>
</tr>
<tr>
<td>2F</td>
<td>dalan</td>
<td></td>
<td></td>
<td>0,93</td>
</tr>
<tr>
<td>3F</td>
<td>floorbeam</td>
<td></td>
<td></td>
<td>3,82</td>
</tr>
<tr>
<td>3F</td>
<td>dalan</td>
<td></td>
<td></td>
<td>0,93</td>
</tr>
<tr>
<td>2F</td>
<td>Floor slab</td>
<td>2,07</td>
<td>6,05 x 9,1</td>
<td>105,66</td>
</tr>
<tr>
<td>3F</td>
<td>Live load</td>
<td>0,50</td>
<td>6,05 x 9,1</td>
<td>25,48</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>503,35</strong></td>
</tr>
<tr>
<td><strong>Fourth lump</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3F</td>
<td>façades</td>
<td>8,1</td>
<td>6</td>
<td>48,60</td>
</tr>
<tr>
<td>3F</td>
<td>side wall</td>
<td>8,1</td>
<td>11,05</td>
<td>89,51</td>
</tr>
<tr>
<td>3F</td>
<td>dalan</td>
<td></td>
<td></td>
<td>9,34</td>
</tr>
<tr>
<td>4R</td>
<td>beam</td>
<td></td>
<td></td>
<td>3,82</td>
</tr>
<tr>
<td>4R</td>
<td>Roof slab</td>
<td>4,16</td>
<td>(10+1)*6</td>
<td>297,39</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>448,65</strong></td>
</tr>
<tr>
<td><strong>Total building load</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>2410,45</strong></td>
</tr>
</tbody>
</table>
B3 Earthquake loads

**Period of vibration:**
For initial purposes the fundamental period of vibration $T_1$ (s) is approximated as:

<table>
<thead>
<tr>
<th>Formula</th>
<th>Direction</th>
<th>Period [s]</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>$0.09 \frac{H}{\sqrt{D'}}$</td>
<td>Depth (6.5 m)</td>
<td>0.33</td>
<td>NBC 105, Eq. 7.4</td>
</tr>
<tr>
<td>$C_t \cdot H^{3/4}$</td>
<td>Length (10.0 m)</td>
<td>0.27</td>
<td>EC 8, Eq. 4.6</td>
</tr>
<tr>
<td>$0.1 \cdot (2/3) \cdot n_{storeys}^*$</td>
<td></td>
<td>0.27</td>
<td></td>
</tr>
</tbody>
</table>

*the factor 2/3 is added, as the typical storey heights are 2 m, instead of the usual dimensions of 3 m.

The parameters extracted from graphs and figures from NBC 105 : 1994.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>Basis Seismic coefficient for seismic coefficient method at plateau</td>
<td>0,08</td>
</tr>
<tr>
<td>Subsoil</td>
<td>Site subsoil category</td>
<td>III</td>
</tr>
<tr>
<td>Z</td>
<td>Seismic zoning factor</td>
<td>1,0</td>
</tr>
<tr>
<td>I</td>
<td>Importance factor</td>
<td>1,0</td>
</tr>
<tr>
<td>K</td>
<td>Structural performance factor</td>
<td>4,0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z</td>
<td>Seismic zone factor</td>
<td>$V = 0.36$</td>
</tr>
<tr>
<td>I</td>
<td>Importance factor</td>
<td>1,0</td>
</tr>
<tr>
<td>R</td>
<td>Response reduction factor</td>
<td>1,5</td>
</tr>
<tr>
<td>$S_a/g$</td>
<td>Average response acceleration coefficient for rock or soil sites</td>
<td>2,5</td>
</tr>
</tbody>
</table>

(IS 1893:2002, Table 2 Clause 6.4.2).
Hence, the horizontal seismic coefficient $C_d$ in both directions is:

$$C_d = CZIK = 0.08 \cdot 1.0 \cdot 1.0 \cdot 4 = 0.32$$

(NBC 105 : 1994, Eq. 8.1)

A reference is made to the Indian Standards (IS 1893:2002).

$$A_R = \frac{Z_{f(Sa/g)}}{2.8} = \frac{0.36 + 1.25}{2.15} = 0.3$$

(IS 1893: 2002 , Eq. 6.4.2)

To be conservative, the value of the NBC 105 : 1994 code is taken as horizontal seismic coefficient. The Total base shear is estimated at:

$$V_b = C_d \cdot W_t = 0.32 \cdot 2442.1 \, kN = 781.5 \, kN \rightarrow \text{which is approximately 30% of the total weight}$$

**Distribution of forces**

A linear distribution of the Base Shear force is adopted, following the NBC Codes. Hence, the horizontal seismic force at each level $i$ shall be taken as:

$$F_i = \frac{V W_i h_i}{\sum W_i h_i}$$

In which:

- $V = \text{total horizontal seismic base shear}$
- $W_i = \text{proportion of } W_t \text{ contributed by level } i$
- $h_i = \text{height to the level designated as } i \text{ from the level of lateral restraint – rigid basement}$

<table>
<thead>
<tr>
<th>Lump</th>
<th>Total Wi [kN]</th>
<th>Hi [m]</th>
<th>Wi*hi (kNm)</th>
<th>Qi [kN]</th>
<th>Storey Shear [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>448.7</td>
<td>9.4</td>
<td>4217.4</td>
<td>266.2</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>513.9</td>
<td>6.6</td>
<td>3391.7</td>
<td>214.1</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>690.0</td>
<td>4.4</td>
<td>3036.9</td>
<td>191.6</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>789.5</td>
<td>2.2</td>
<td>1726.9</td>
<td>109.6</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>718.5</td>
<td>718.5</td>
<td></td>
</tr>
</tbody>
</table>

**Check of Shear stress**

For unreinforced masonry load bearing wall buildings, the shear stress should be limited to 0.1 MPa. The shear stress in Shear walls is given as: $\tau_{wall} = V/A_w$

In y-direction

<table>
<thead>
<tr>
<th>Storey</th>
<th>Storey Shear ($V_b$) [kN]</th>
<th>Area of Shear wall ($A_w$) [m2]</th>
<th>Stress [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>718.5</td>
<td>5.85</td>
<td>0.12</td>
</tr>
</tbody>
</table>

In x-direction

<table>
<thead>
<tr>
<th>Storey</th>
<th>Storey Shear ($V_b$) [kN]</th>
<th>Area of Shear wall ($A_w$) [m2]</th>
<th>Stress [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>718.5</td>
<td>8.6</td>
<td>0.09</td>
</tr>
</tbody>
</table>

Hence, the check is not satisfied in the depth (y) direction.
APPENDIX C  EVALUATION OF RETROFIT OPTIONS

C1  Evaluation of methods
An overview of existing retrofit measures will be evaluated on the basis of 6 design and construction criteria. The measures are classified into the following categories:

1. Wall-to-diaphragm connection
2. Stiffening of diaphragm
3. Connecting perpendicular walls
4. Seismic belts
5. Strengthening of wall
And combined methods

Six design- and construction criteria are defined:

<table>
<thead>
<tr>
<th>Design criteria</th>
<th>Construction criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural efficiency</td>
<td>Impact on architecture</td>
</tr>
<tr>
<td>Impact on architecture</td>
<td>Durability</td>
</tr>
<tr>
<td>Durability</td>
<td>Construct-ability</td>
</tr>
<tr>
<td>Construct-ability</td>
<td>Cost-effectiveness</td>
</tr>
<tr>
<td>Cost-effectiveness</td>
<td>Material availability</td>
</tr>
</tbody>
</table>

An evaluation is made by means of a color grading system, in which green is the most favorable option and red the most unfavorable. For the category concerning material availability, the system is adapted to provide information on the availability of materials with respect to the building location. It is acknowledged that the grading system is inherently subjective.

Design and construction requirements

<table>
<thead>
<tr>
<th>Design and construction requirements</th>
<th>Material availability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Favorable solution</td>
<td>Feasible material in remote areas,</td>
</tr>
<tr>
<td></td>
<td>either local or transportable by foot</td>
</tr>
<tr>
<td>Uncertain or neutral solution</td>
<td>Feasible and known material in urban</td>
</tr>
<tr>
<td></td>
<td>and rural areas</td>
</tr>
<tr>
<td>Unfavorable solution</td>
<td>High-tech material, mostly imported</td>
</tr>
<tr>
<td></td>
<td>and suitable for high-end buildings</td>
</tr>
<tr>
<td></td>
<td>in urban cores</td>
</tr>
</tbody>
</table>

**For the source credits of the pictures on the following pages, refer to the end of this paragraph.**
## APPENDIX C Evaluation of retrofit options

<table>
<thead>
<tr>
<th>Retrofit measure</th>
<th>Evaluation criteria</th>
<th>Comments / issues</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wall-to-floor</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Wall anchors</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Adhesive anchors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Connection to wall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Plate anchors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ductile plate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Decorative plate</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Decorative ridge</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Anchor-to-joist</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Steel strap</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Wall anchors

**Adhesive anchors**
- Chemical bond through epoxy injection, mechanical bond with threaded rod
- Epoxy resin is expensive
- Capacity is dependent on masonry quality
- Closer spacing is possible than for plate anchors

**Plate anchors**
- Larger pull-out capacity than adhesive anchors and larger spread of loads to perimeter of anchor plate
- Larger spacing necessary (to prevent interaction of adjacent cone failures)
- Anchor plates on the exterior of the masonry

**Ductile plate**
- Petalled curved plate
- High-end product
- Energy dissipation through yielding or friction of the anchor plate
- Friction is more effective

**Decorative plate**
- Decorative design
- With consideration of architecture and aesthetics of the façade

**Decorative ridge**
- Plastered decorative ridge at floor level
- Protection against corrosion of wall-ties and fasteners
- Feasible for buildings with a certain status

**Anchor-to-joist**
- Direct connection wall-floor
- If no diaphragm strengthening is needed
- Low seismicity areas

**Steel strap**
- Without anchor plate there is too little anchorage capacity for high-seismicity areas
### APPENDIX C Evaluation of retrofit options

<table>
<thead>
<tr>
<th>Retrofit Option</th>
<th>Selection 1</th>
<th>Selection 2</th>
<th>Selection 3</th>
<th>Selection 4</th>
<th>Selection 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joist anchor</td>
<td>![Joist anchor diagram]</td>
<td>![Joist anchor diagram]</td>
<td>![Joist anchor diagram]</td>
<td>![Joist anchor diagram]</td>
<td>![Joist anchor diagram]</td>
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<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Joist plate</td>
<td>![Joist plate diagram]</td>
<td>![Joist plate diagram]</td>
<td>![Joist plate diagram]</td>
<td>![Joist plate diagram]</td>
<td>![Joist plate diagram]</td>
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<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Joist ties, galvanized</td>
<td>![Joist ties, galvanized diagram]</td>
<td>![Joist ties, galvanized diagram]</td>
<td>![Joist ties, galvanized diagram]</td>
<td>![Joist ties, galvanized diagram]</td>
<td>![Joist ties, galvanized diagram]</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Connector</td>
<td>![Connector diagram]</td>
<td>![Connector diagram]</td>
<td>![Connector diagram]</td>
<td>![Connector diagram]</td>
<td>![Connector diagram]</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Continuous angle section</td>
<td>![Continuous angle section diagram]</td>
<td>![Continuous angle section diagram]</td>
<td>![Continuous angle section diagram]</td>
<td>![Continuous angle section diagram]</td>
<td>![Continuous angle section diagram]</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Individual angles</td>
<td>![Individual angles diagram]</td>
<td>![Individual angles diagram]</td>
<td>![Individual angles diagram]</td>
<td>![Individual angles diagram]</td>
<td>![Individual angles diagram]</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Thin steel plate</td>
<td>![Thin steel plate diagram]</td>
<td>![Thin steel plate diagram]</td>
<td>![Thin steel plate diagram]</td>
<td>![Thin steel plate diagram]</td>
<td>![Thin steel plate diagram]</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Timber blocks</td>
<td>![Timber blocks diagram]</td>
<td>![Timber blocks diagram]</td>
<td>![Timber blocks diagram]</td>
<td>![Timber blocks diagram]</td>
<td>![Timber blocks diagram]</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

- **Joist anchor**
  - Solutions is specific for the anchor parallel to the joist
  - For existing structures: it is hard to drill near the joists

- **Joist plate**
  - Solutions is specific for the anchor parallel to the joist
  - For existing structures: it is hard to drill near the joists
  - Welding required of anchor tie to plate

- **Joist ties, galvanized**
  - In combination with concrete band
  - Galvanized wires resistant to corrosion
  - Feasible for new structures or at roof level

- **Connector**
  - Interface between anchor and diaphragm fixation
  - Eccentricity load transfer

- **Continuous angle section**
  - Stiff connection with respect to masonry and floor
  - Steel sections have little workability on site
  - High costs of steel sections
  - Extra structural contribution as diaphragm chord
  - Spread of the load over a line instead of anchor points

- **Individual angles**
  - Stiff connection with respect to masonry and floor
  - High costs of steel sections
  - Little spread of the load

- **Thin steel plate**
  - Deformation capacity
  - Energy dissipation through deformation of the plate
  - Larger surface to spread the loads, staggered holes

- **Timber blocks**
  - Beneficial workability on-site
  - Must be placed underneath floor joists due to close spacing
# Evaluation of retrofit options

## Diaphragm stiffening/strengthening

<table>
<thead>
<tr>
<th>Evaluation criteria</th>
<th>Comments / issues</th>
</tr>
</thead>
</table>
| Plywood overlay | - Ensures a leveled surface  
- Glued connection provides high increase in stiffness  
- For the roof with water resistant layer  
- No access to plywood in remote areas  
- Extra mass = 13.5 kg/m² |
| Plank overlay | - Rotation of plank direction gains stiffness in other direction  
- Single diagonal planks Extra mass = 10.5 kg/m²  
- Double layer planks Extra mass = 21.5 kg/m² |
| Diagonal straps FRP | - High tensile capacity, high increase in strength and stiffness  
- No contribution to gravity load bearing  
- Extra mass = 10.5 kg/m²  
- Special equipment needed |
| Diagonal straps steel (galvanized) | - High tensile capacity  
- No leveled surface on top  
- No contribution to gravity load-bearing  
- Extra mass = 17.1 kg/m² |
| Diagonal steel truss | - Lighter alternative than a new concrete floor  
- Minimize the section depth of brace, due to low ceiling heights  
- Low workability on site  
- Steel sections expensive |
| Timber bracing | - Minimize the section depth, due to low ceiling heights |
| New RC floor | - Wedged shear lock wall connection detail is hard to construct  
- Alteration of internal traditional architecture  
- High additional mass |
### Concrete overlay
- Nails as dowels (transfer shear force)
- Layer must be at least 40 mm thick
- Thin RC overlay can crack due to flexible diaphragm
- Extra mass= 125 kg/m²

### CGI-sheet RC-overlay
- CGI steel as formwork, reinforcement and composite floor action.
- No reinforcement needed
- Local perception is that CGI can’t be recycled, and is sacrificed

---

<table>
<thead>
<tr>
<th>Retrofit measure</th>
<th>Evaluation criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wall intersections</strong></td>
<td></td>
</tr>
</tbody>
</table>
| Stitching                 | • Steel strip or stone stitches  
  • Easier for new construction |
| Diagonal metal stitching  | • Difficult inclined drilling into masonry |
| Through wall-anchor       |                       |
| Anchored splint overlay   | • Internal and external L-shaped overlay, cross anchors needed  
  • Horizontal bandage is more effective and important than the vertical anchored splint overlay |
## APPENDIX C Evaluation of retrofit options

<table>
<thead>
<tr>
<th>Seismic belt</th>
<th>[Image]</th>
<th>[Image]</th>
<th>[Image]</th>
<th>[Image]</th>
<th>Comments / issues</th>
</tr>
</thead>
</table>
| Containment reinforcement | ![Diagram] | ![Diagram] | ![Diagram] | ![Diagram] | • Executed with locally available reinforcement bars  
• Counteracts separation of perpendicular walls  
• Laborious and challenging to install  
• Little visibility in façade |
| Bands | ![Diagram] | ![Diagram] | ![Diagram] | ![Diagram] | • Brick edging can serve as formwork and cover-up  
• Provides a uniform introduction of floor/roof loads into the wall.  
• A portion of the wall has to be removed  
• More feasible for new construction |
| Timber bands | ![Diagram] | ![Diagram] | ![Diagram] | ![Diagram] | • Provides a uniform introduction of floor/roof loads into the wall.  
• More feasible for new construction |
| Horizontal bandage | ![Diagram] | ![Diagram] | ![Diagram] | ![Diagram] | • Surface cleaning needed  
• Impact on exterior façade  
• No demolition of wall, applied on exterior |
| Splint & bandaging | ![Diagram] | ![Diagram] | ![Diagram] | ![Diagram] | • Surface cleaning needed  
• Large impact on façade exterior  
• No demolition of wall, applied on exterior |
### APPENDIX C Evaluation of retrofit options

<table>
<thead>
<tr>
<th>Retrofit measure</th>
<th>Evaluation criteria</th>
<th>Comments / issues</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strengthening of wall</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Galvanized wires</td>
<td><img src="image1" alt="Image" /></td>
<td>• Connects interior mesh (reinforcement) to outer wall</td>
</tr>
<tr>
<td></td>
<td><img src="image2" alt="Image" /></td>
<td>• Galvanized wire with aluminum plate</td>
</tr>
<tr>
<td></td>
<td><img src="image3" alt="Image" /></td>
<td>• Increases coherency of wall wythes</td>
</tr>
<tr>
<td>Wire mesh</td>
<td><img src="image4" alt="Image" /></td>
<td>• Effective performance</td>
</tr>
<tr>
<td></td>
<td><img src="image5" alt="Image" /></td>
<td>• Simple option, steel reinforcement mesh</td>
</tr>
<tr>
<td></td>
<td><img src="image6" alt="Image" /></td>
<td>• Susceptible to corrosion</td>
</tr>
<tr>
<td></td>
<td><img src="image7" alt="Image" /></td>
<td>• Moderate costs (~$ 4,4 /m²)</td>
</tr>
<tr>
<td></td>
<td><img src="image8" alt="Image" /></td>
<td>• Proven performance</td>
</tr>
<tr>
<td>Polypropylene mesh</td>
<td><img src="image9" alt="Image" /></td>
<td>• Low-cost (~$0,8-1,4 m², by owner</td>
</tr>
<tr>
<td></td>
<td><img src="image10" alt="Image" /></td>
<td>• Durable material</td>
</tr>
<tr>
<td></td>
<td><img src="image11" alt="Image" /></td>
<td>• Widely available material</td>
</tr>
<tr>
<td></td>
<td><img src="image12" alt="Image" /></td>
<td>• Laborious spanning of PP bands</td>
</tr>
<tr>
<td></td>
<td><img src="image13" alt="Image" /></td>
<td>• Research is done on structural capacity</td>
</tr>
<tr>
<td>Bamboo mesh</td>
<td><img src="image14" alt="Image" /></td>
<td>• Low-cost (~$ 2,5 /m²)</td>
</tr>
<tr>
<td></td>
<td><img src="image15" alt="Image" /></td>
<td>• No special workers needed</td>
</tr>
<tr>
<td></td>
<td><img src="image16" alt="Image" /></td>
<td>• Bricks can damage bamboo mesh</td>
</tr>
<tr>
<td></td>
<td><img src="image17" alt="Image" /></td>
<td>• Durability concerns</td>
</tr>
<tr>
<td>Geomesh</td>
<td><img src="image18" alt="Image" /></td>
<td>• Compatible with earthen wall deformation</td>
</tr>
<tr>
<td></td>
<td><img src="image19" alt="Image" /></td>
<td>• Moderate costs. Industrial grid (<del>$ 21/m²), Soft polymer mesh grid (</del>$ 4,5 /m²)</td>
</tr>
<tr>
<td>Plastic carrier mesh</td>
<td><img src="image20" alt="Image" /></td>
<td>• Very low-cost</td>
</tr>
<tr>
<td></td>
<td><img src="image21" alt="Image" /></td>
<td>• Normally sent to landfills</td>
</tr>
<tr>
<td></td>
<td><img src="image22" alt="Image" /></td>
<td>• Little testing is done on performance</td>
</tr>
<tr>
<td>Strongbacks timber</td>
<td><img src="image23" alt="Image" /></td>
<td>• Lack of compatibility with existing structure</td>
</tr>
<tr>
<td></td>
<td><img src="image24" alt="Image" /></td>
<td>• Fixation to the wall?</td>
</tr>
<tr>
<td>Strongbacks steel</td>
<td><img src="image25" alt="Image" /></td>
<td>• Lack of compatibility with existing structure</td>
</tr>
<tr>
<td></td>
<td><img src="image26" alt="Image" /></td>
<td>• Fixation to the wall?</td>
</tr>
</tbody>
</table>

*S *source costs (Sathiparan, 2015)
## Evaluation of retrofit options

### Interior concrete cage

- Lack of compatibility with existing structure
- Additional mass
- Large concrete dimensions

---

The prices m² originate from (Sathiparan et al., 2014)

### Combined solutions

<table>
<thead>
<tr>
<th>Combined solutions</th>
<th>Comments / issues</th>
</tr>
</thead>
</table>
| Anchor fixed under plywood/diaphragm | • Diagonal lay-out of anchors to provide in-plane resistance  
• Perforated steel strip as wall anchor |
| Anchors in wall fold overlay | • Too little anchorage capacity in wall  
• Diaphragm fixation by laying the anchor into the wet concrete  
• Extra mass = 125 kg/m² |
| Anchor inlay into wall | • Too little anchorage capacity in wall  
• Diaphragm fixation by laying the anchor into the wet concrete |
| One sided mesh fixed under plywood | • Pull-off masonry glass fiber (debonding)  
• Cleaning of surface needed  
• Flexible connection  
• Large fixation surface / spread of load  
• Extra mass = 125 kg/m² |
| RC-chord with shear locks | • Wedged connection detail with the wall is hard to construct (tapered shear lock key)  
• Functions as diaphragm chord |
APPENDIX C Evaluation of retrofit options

Pictures source credit


(Arup, 2014)  (Tomažević, 1999)


(Own picture)  (FEMA)  (Dunning thornton consultants )  (CPWD & IBC, 2008)  (Maffei et al., n.d.)
C2 Dismissed proposals

After the evaluation of methods, several proposals were done for retrofit combinations. The following proposals are dismissed due to limitations in construction aspects or feasibility.

Fix beam ends

The idea is to use the protruding beam as wall-anchor and fixing the beam-ends with timber planks or slats as anchors on the exterior of the masonry wall. This proposal faces the following challenges:

- Anchoring along the grain of timber into the beam-ends provides little resistance.
- The beam ends might be affected by moisture. Therefore specialized long bolts would be required to reach decent anchorage capacity in the timber joist.
- A continuous beam or plank on the exterior is susceptible to rotting when it is placed adjacent to the masonry. Masonry will hold the moisture and there is no possibility for ventilation. Timber slats would be a better solution.

Use of corrugated sheet metal

The idea is to apply a thin steel plate as a wall-to-floor connector element. The thin plate will allow some flexibility in the connection. Corrugated steel metal could be used. The CGI-sheet can be used as formwork to create a composite slab with profiled steel sheeting. The proposal faces the following challenges:

- Building materials such as corrugated sheet metal are seen as a material which can be re-used. When used as composite sheet for a composite floor, people consider the corrugated sheet metal ‘sacrificed’, seeming a waste of the material.
- How to fix the steel plate to the wall? Adhesive anchors have insufficient anchorage capacity.
Wall to floor mesh

A glass-fibre mesh applied to the wall and clamped underneath the improve diaphragm (suggested by Nienhuys). The proposal poses the following challenge:

- How to connect the wire mesh to the wall? The mesh is prone to debonding of the masonry.
- The mesh option requires fixation and interlinkage on both sides of the wall for sufficient anchorage.

![Figure C.3 Glass fibre mesh applied on interior wall and fixed underneath the strengthened diaphragm](image)

Alternative diagonal floor bracing

Possible low-cost alternatives for stiffening of the timber floors are using steel diagonal strips (Figure C.4) or multiple galvanized steel wires (Figure C.5).

These perforated steel strips are being used extensively as windbracing of roofs. The low-cost galvanized steel wires can be pre-tensioned by twisting them with a pincer. As the thin wires are stressed and start to deform, they could possibly absorb some of the energy from the earthquake, if the masonry allows enough displacement for the wires to yield.

![Figure C.4 Diagonal steel strips “rispenband” as diaphragm stiffener (Source: ebay.ie and Bauexpertenforum.de)](image)

![Figure C.5 Galvanized diagonal wires as diaphragm stiffener and energy dissipators](image)
APPENDIX D  MASONRY MODELLING

For the masonry modelling, a newly developed masonry shell formulation (MAT_SHELL_MASONRY) is used, which is made for the nonlinear simulation software LS-DYNA. The description of the approach are directly from the appendix on ‘Modelling masonry walls with MAT_SHELL_MASONRY’ by Arup (2015).

D1  Smeared crack approach

This LS-DYNA material model has a smeared crack formulation with crack plane directions pre-defined according to mortar planes (bed joints and head joints).

The strain is decomposed into (a) strain that induces stress in the masonry and (b) strain associated with the cracks (joints) opening, closing or shearing. The opening displacement of cracks is tracked, so that they open and close in a realistic manner under hysteretic loading. Compression can be carried across closed cracks. The shear strength of bonds is the sum of a cohesion component and a friction component. The cohesion and tensile strength of the bonds decay to zero according to a damage law (damage increases linearly with displacement) while the friction coefficient remains constant.

(Arup, 2015a)

D2  Failure modes considered

Tensile failure of the bed joints is defined by a tensile strength (FTH) and an energy release rate (GTH) as illustrated in Figure D.1 and Figure D.2. The same bed joint tensile response governs in-plane (e.g. rocking) and out-of-plane failures.

Figure D.1 Tensile response of bed joints governing in-plane failures  (Arup, 2015a)
Bed joint shear response is illustrated in Figure D.3. The shear strength consists of a friction term (proportional to compressive stress normal to the joint) and a bond strength term. The bond strength decreases with shear displacement according to an energy release rate, which increases with compressive stress normal to the joint.
The material model offers the possibility for vertical cracks through the bricks and the head joints (Figure D.4) but the input data used for almost all GESU models suppresses these. The exceptions are (a) continuous vertical mortar joints, e.g. at junctions between walls, and (b) to model existing damage where the bricks are seen to be cracked.

![Figure D.4 Vertical cracks through the bricks are normally suppressed (Arup, 2015a)](image)

Other modes of deformation considered are illustrated in Figure D.5. These consist of pre-determined combinations of head joint opening with bed joint sliding.

![Figure D.5 Failure modes combinations of head joint opening with bed joint sliding (Arup, 2015a)](image)

Two options are available for the nonlinear response in compression:

- Bilinear elastic-plastic. If only the compressive strength is input, then the behaviour is elastic-perfectly-plastic. This method does not capture the degradation of the material at high strain.
- A user-defined stress-strain curve. The curve typically includes softening/degradation after the point of maximum strength is reached. This can be an important contributing factor in modelling of collapse.

**Modelling of collapse**

Collapse due to out-of-plane bending exceeding the thickness of the wall occurs naturally in the LS-DYNA models because the line of action of the gravity load then falls outside the cross-section leading to increasing deformation.

Other modes of collapse may be initiated by large displacements of the bricks such that they fall out of the structure, or when bricks are crushed. This is modelled by elements being deleted from the calculation when any of these criteria is met:

- In-plane sliding displacement greater than half the brick length
- Out-of-plane sliding displacement greater than half the brick thickness.
- Crushing greater than half the brick height

(Arup, 2015a)
APPENDIX E  SEISMIC HAZARD ASPECTS

E1  Target spectrum NBC

The spectral acceleration coefficient for the horizontal component of seismic action defined as follows, according to NBC:1994.

\[ S_a/g = \begin{cases} \frac{T}{T_B} (\eta \cdot \beta_0 - 1) & 0 \leq T \leq T_B \\ \eta \cdot \beta_0 & T_B \leq T \leq T_C \\ \eta \cdot \beta_0 \cdot \left( \frac{T_C}{T} \right)^{k_1} & T_C \leq T \leq T_D \end{cases} \]

- \( S_a/g \) = the spectral acceleration coefficient
- \( T \) = the vibration period of a linear SDOF system
- \( \eta \) = the damping correction factor with a reference value of \( \eta = 1 \) for 5\% viscous damping
- \( \beta_0 \) = the maximum normalised spectral value assumed constant between \( T_B \) and \( T_C \) (\( \beta_0 = 2.5 \))
- \( k_1 = 1 \) the exponent influencing the shape of the elastic response spectrum

<table>
<thead>
<tr>
<th>Subsoil</th>
<th>( \beta_0 )</th>
<th>( k_1 )</th>
<th>( T_B )</th>
<th>( T_C )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.5</td>
<td>1.0</td>
<td>0.10 s</td>
<td>0.40 s</td>
</tr>
<tr>
<td>B</td>
<td>2.5</td>
<td>1.0</td>
<td>0.10 s</td>
<td>0.50 s</td>
</tr>
<tr>
<td>C</td>
<td>2.5</td>
<td>1.0</td>
<td>0.10 s</td>
<td>1.00 s</td>
</tr>
</tbody>
</table>

\( T_B \) = the lower limit of the period of the constant spectral acceleration branch;
\( T_C \) = the upper limit of the period of the constant spectral acceleration branch;
\( T_D \) = value defining the beginning of the constant displacement response range of the spectrum;

Figure E.1 Normalized design spectrum of Nepalese Building code NBC 105:1994
E2 Seismic hazard aspects – Kathmandu valley

Several aspects associated with the seismic hazard are reviewed in this section:

- Magnitude
- PGA
- Frequency content
- Duration
- Soil characteristics

**Maximum earthquake magnitude**

Large destructive earthquakes tend to occur in the range of 7-9 Mw. The maximum considered earthquake magnitude in Nepal would be M8.2, based on a review of historical earthquake events. However, Parajuli et. al (2012) mention a study (Lave et al., 2005) providing evidence that a big earthquake of M8.8 m is likely to have occurred in medieval times around the same location of the destructive Nepal-Bihar earthquake of 1934. Faults can be seen as a geological evidence of the occurrence of earthquakes. The overview of past earthquake and faults (Figure E.2) shows that the faults are very closely spaced.

**Peak ground acceleration – earthquake density map**

A probabilistic earthquake density map is developed by Parajuli et. al. (2012) on the basis of both earthquake data and fault lines. The contour plot demonstrates PGA values\(^\text{25}\) of around 0.5g near Kathmandu, 0.4g in the western part of Nepal and 0.3 g in the rest of the country. The plot accounts for 5% damping, and a 10 % probability in 50 years (475 years return period).

![Figure E.2 Probabilistic seismic hazard studies](image)

**Frequency content**

The code response spectrum NBC 105:1994 for soft soils is chosen as a reference to pick ground motions for numerical analysis with fitting frequency content. The normalized response spectra of the record motions of El Centro (1994), Kobe (1995) and the recent Gorkha earthquake (2015) are compared to the standard code spectrum for soft soils. The period range 6-9 Hz, or ~0.15 to 0.2 s is common to low-rise masonry structures. (Parajuli & Kiyono, 2015). The El Centro motion lies close to the code spectrum in that period range.

\(^{25}\) PGA values can be expressed in g (1 g = 9.81 m/s²) or in Gal (1 Gal = 0.01 m/s², 1 g = 981 Gal).
APPENDIX E Seismic hazard aspects

Figure E.3 Comparison of normalized response spectra of recent Gorkha earthquake (2015), El Centro (1940) and Kobe (1995)

Figure E.4 Comparison of velocity spectra of recent Ghorka earthquake (2015, El Centro (1940) and Kobe (1995) to the code spectrum.

**Duration**

The average earthquake durations lie between 40-50 seconds of duration. The average total earthquake duration for several probabilities of exceedance were calculated probabilistically by Parajuli et al. (2012).

<table>
<thead>
<tr>
<th>Probability</th>
<th>40% in 50 years</th>
<th>10% in 50 years</th>
<th>5% in 50 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total duration (s)</td>
<td>39.0</td>
<td>50.0</td>
<td>53.0</td>
</tr>
</tbody>
</table>

Table E.1 Probabilistically calculated earthquake durations (Source: Parajuli et al., 2012)

**Soil characteristics**

The valley of Kathmandu is built over a former lake bed. The soft soil can cause amplified vibrations of seismic waves for higher periods. The layer of sediments is around 550-650 m corresponding to a $v_{s30}=250$ m/s.
Figure E.5 Average shear wave velocity at 30 m (Source: Goda et al., 2015)

**E3 El Centro Ground motion records**

The acceleration, displacement time histories and response spectrum are displayed below for the El Centro motions which are used for the numerical analyses. The motions are cut-off at t=33s. [http://www.strongmotioncenter.org/vdc/scripts/event.plx?evt=1098](http://www.strongmotioncenter.org/vdc/scripts/event.plx?evt=1098)

Figure E.6 El Centro Ground motions – acceleration and displacement records, cut-off at 30+ seconds and response spectra (Source: original acceleration records from PEE Ground Motion Database)
APPENDIX F  FORCE DEFLECTION CURVES

The force-deflection curves of the numerical models are placed below. Refer to paragraph 10.6.2 for information on the measured node.

**F1  Unstrengthened model vs. model with stiff diaphragm**

The models shaken in X-direction:

**Unstrengthened**

The models shaken in Y-direction:

**Unstrengthened**
F2 Retrofit models

Models shaken in X-direction:

- Anchors + bracing
- Anchors + planks
- Bandage + planks
- Bandage + concrete overlay
Models shaken in Y-direction:

- **Anchors + bracing**

- **Anchors + planks**

- **Bandage + planks**

- **Bandage + concrete overlay**