Assessment of the seismic performance and sustainability of the Kath-Kuni building style in the Indian Himalaya

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Assessment of the seismic performance and sustainability of the Kath-Kuni building style in the Indian Himalaya

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Cover picture
Kath-Kuni building in Vashisht, Kullu (source: own picture)
Preface

This document is written as conclusion of the Master Structural Engineering at Delft University of Technology. It all started when I was traveling through the Indian Himalaya in September 2016. Their vernacular ‘Kath-Kuni’ structures which I observed there, were inspiring in terms of their architecture, wood carvings, but most of all because of the stories from local people about their seeming resistance to earthquakes. I never thought I would get the opportunity to research something that interests me so much. Hence, I realise how lucky I was to meet Dr. Sanjay Chikermane from the Indian Institute of Technology, with whom I share a great passion for these traditional buildings. For the approximate period of one year (over the course of 2017 and 2018), I worked on this thesis together with Sanjay in India. He is the first person I would like to thank tremendously for all the support he has given me during, and outside this project. Without him as a daily supervisor and good friend, this thesis would not have been possible. He invested a lot in this thesis, which was based on a tremendous faith he has in me to make this work successful. He always highlighted the importance of creative thinking and under his guidance I grew as an engineer. Thank you Sanjay, for always being patient with me. I am very happy to be able to continue and extend the research on vernacular architecture in the Indian Himalaya under the project name ‘Resilient Himalayan Homes’, together with Sanjay at the Indian Institute of Technology Roorkee.

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Last, I would like to address a few words to my family and friends, since they were there for me through the whole process in all its ups and downs, even though I was often in India. Thank you for the help and encouragement you gave me during this Master’s thesis and just for always being there. I would like to especially thank my parents, Camille, Prakhar, Shweta, Arvind and Tessa for giving me so much positive energy.

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Summary

Increasing disasters, due to urbanism in combination with the high probability of earthquakes in the Western Indian Himalayas, are impacting the well-being of local communities. The vernacular Kath-Kuni (also called: ‘cator-and-cribbage’ or ‘timber laced masonry’) architecture is a centuries-old building method spread over the Himalayas and the Karakoram mountain range in India, Pakistan and parts of Afghanistan and China. This building typology’s efficacy was proved by its ability to withstand significant seismic events in the region and is, hence, well-known to be an earthquake resistant building style.

Kath-Kuni houses are characterised by double horizontal timber beams in part of the thick load bearing walls, which are connected to one another by timber dovetail connections called ‘maanwi’s’. The vertical space between the timber beams is filled with dry-stone masonry. In the corners, the beams are connected in horizontal direction by timber dowels called ‘kadils’. The Kath-Kuni buildings style has never been conventionally engineered, but is a consequence of constant improvement by local carpenters and the knowledge passed on from generation to generation. Over the years, local communities have become less inclined to building in Kath-kuni style, which has resulted in a loss of building knowledge. To ensure that this valuable knowledge does not get lost completely, the main question asked in this research is: “Which earthquake resistant features and embedded traditional knowledge of Kath-Kuni walls are essential in generating adequate seismic performance of this vernacular architecture?”

To this end, field research, out-of-plane and in-plane analysis of walls, laboratory testing to characterise the walls ductility, analysis of the building’s box-action and a sustainability study comparing Kath-Kuni with concrete buildings have been carried out.

From field research it could be concluded that there is a great variation in the construction detailing among Kath-Kuni buildings, enhanced by the diminishing traditional building skills and adaptations influenced by “modern” building techniques. Nevertheless, many common characteristics are also observed, like the cuboid configuration and the type of connections. Another important feature of this building style is the presence of flexible floor diaphragms, which is normally disadvantageous in terms of seismic capacity of construction methods like masonry, especially for the walls loaded in out-of-plane direction. Rigid floor diaphragms normally contribute to the box-action of the building, hence it is investigated what box-action generating features are present in Kath-Kuni construction. The different storeys in a traditional house are built with different construction techniques, however in this thesis only the Kath-Kuni part of the building is investigated. One typical well-maintained and preserved house, located in Old-Jubbal (Shimla district) in Himachal Pradesh, is taken as case study building, where all further calculations are based on.
From literature and the Indian seismic design code IS 1893 (Part 1):2002, peak ground acceleration demand values are obtained. To validate the empirically proven seismic performance of the Kath-Kuni building style, the buildings' capacity is evaluated in terms of maximum allowable peak ground acceleration and compared to the demand values. Probabilistic values are ranging between 0.075 g and 0.732 g, with a return period of 2475 years and they are ranging between 0.0039 g and 0.289 g with a return period of 475 years. Slightly lower values are used in IS 1893 (Part 1) for the design of structures, namely a peak ground acceleration of 0.54 g for a maximum considered earthquake and 0.27 g for a design basis earthquake.²

To assess the seismic behaviour of the full building, the linear static ‘equivalent lateral force method’ is used. The in-plane loaded walls are evaluated for the global mode of vibration of the building, hence these walls are governing for the total stiffness of the building. However, the walls will be prone to vibrations in out-of-plane direction and because rigid floor diaphragms are lacking, other box-action generating features are required to prevent out-of-plane wall failure. Non-linear behaviour (ductility) of the structure cannot directly be taken into account in the equivalent lateral force method. However, a behaviour factor $R$ can be applied to allow designing the structures for resistance to seismic forces smaller than those corresponding to a linear elastic response. The behaviour factor $R$ and the time period $T$ of the structure are input parameters obtained from IS 1893 (Part 1), valid for other more conventional construction typologies and their validity for Kath-Kuni structures is therefore questionable and is investigated in this research.

The out-of-plane loaded wall can be assessed by focussing on an individual layer, because the stiffness of the wall is dominated by the stiffness of the timber beams spanning in horizontal direction. Hence, the lateral force is transferred directly to the in-plane loaded walls. The maximum peak ground acceleration an out-of-plane loaded wall can resist is determined by making use of a modelling range, which accounts for the uncertainty in the connectivity and stiffness of the corner connections and the maanwi’s in between the beams. The time period $T$ is derived for the bounds of the stiffness range and since the ductility of walls in this direction is uncertain, a behaviour factor $R$ is conservatively assumed to be 1.0. Nevertheless, the linear out-of-plane wall model gives a maximum allowable lower bound peak ground acceleration of 0.53 g and the upper bound peak ground acceleration of 13.53 g. The lower bound value is higher than the design demand peak ground acceleration value obtained from IS 1893 for a design basis earthquake and similar to the design demand peak ground acceleration value of a maximum considered earthquake. The values are also exceeding the peak ground acceleration obtained from literature with a return period of 475 years. The analysis shows that the wall’s absolute lower bound capacity is insufficient to withstand an earthquake with a return period of 2475 years. However, this lower bound calculation is conservative since pinned supports (connection with the in-plane loaded wall) are assumed. Moreover, a behaviour factor larger than 1.0, reflecting non-linear behaviour, is not considered. Therefore, it can be concluded that an out-of-plane loaded wall is not critical for the seismic capacity of the total building.

To investigate the lateral force capacity of the full building, a seven layered in-plane wall model is designed. It is based on the virtual work approach, where several internal rotation failure mechanisms are evaluated. The kadil connections, maanwi connections, beam rotations and friction are the contributing elements to the total lateral force capacity of a layer. This resulted in expected failure in the top layer with a maximum force capacity of 59 kN for a wall of 2995 mm length, 2110 mm height and 460 mm thickness. Upscaling of the Kath-Kuni wall results in the peak ground acceleration capacity of the full building, where a behaviour factor $R$ of 2.5 and a

² Taking into account a partial safety factor of 1.5.
time period $T$ of 0.38 s are used. This leads to a maximum allowable peak ground acceleration of 0.06 g. This peak ground acceleration is not in accordance to the empirically proven seismic performance of a Kath-Kuni building. Therefore, a quasi-static in-plane wall experiment is performed to investigate the validity of the standard behaviour factor $R$ and the time period $T$ values obtained from IS 1893 (Part 1).

A Kath-Kuni wall specimen was built and tested by an in-plane pushover wall test at the laboratory of the Indian Institute of Technology Roorkee. The wall was loaded with an overburden of 0.067 MPa and subsequently a lateral load was applied in displacement controlled increments. The failure of the wall is governed by internal rotation in the layer and showed a high ductility. The behaviour factor $R$ of the tested wall is calculated to be 5.7, which is 2.3 times higher than the value used in the initial assessment, and assumed to be a lower bound value for a full-scale wall. The time period $T$ is established to be a minimum of 0.48 s for the most critical walls, which gives a value outside of the plateau of the elastic response spectrum. A revised maximum allowable peak ground acceleration of 0.19 g is found, which is a factor 3.2 higher than the initial established maximum allowable peak ground acceleration of 0.06 g. This is a significant improvement and proves that the standard values obtained from IS 1893 (Part 1) are not representative for Kath-Kuni structures. Because many uncertainties need further investigation and a lot of lower bound assumptions are taken in this calculation, it is very likely that the behaviour factor $R$ and the time period $T$ may be established to be even higher. Hence, the equivalent lateral force method is used only as an initial seismic evaluation tool; it is recommended in future research to explore non-linear and dynamic analysis methods.

The analytical in-plane loaded wall model used is validated using the experimental results. The analytical model is scaled to correspond with the dimensions of the tested wall and, instead of earlier used design material properties, assumed mean material properties are adopted. The in-plane loaded wall validation model gave a force capacity which complies with the experimental bi-linear maximum force capacity with an accuracy of 10%, suggesting that the analytical in-plane wall model is reasonably able to approximate the actual wall behaviour. The difference in capacity among the various evaluated failure mechanisms does not exceed 10 % of the total wall capacity. The deviation of the material properties of timber is high, which makes the finally activated failure mechanism in the experimental wall highly dependent on local material imperfections and strength reductions.

The main conclusion to the research question starts with the box-action generated by the out-of-plane loaded walls. The horizontal spanning timber beams and the robust connection in the corners are generating box-action by transferring the lateral force from the out-of-plane loaded wall to the in-plane loaded walls. Thus, the seismic capacity of the building is not depending on the rigidity of the floor diaphragms. Secondly, the in-plane loaded walls, which determine the total buildings’ capacity, use a behaviour factor $R$ to take non-linear wall behaviour (ductility) into account. Research to this behaviour factor and the time period of the building indicates that the behaviour factor and time period obtained from standard formulas from IS 1893 (Part 1) are too conservative. Last, the most important earthquake resistant features that contribute to the high ductility of the in-plane loaded walls are the timber connections (kadii dowel connection and maonwi dovetail connection) which are allowing internal rotation in the layer and are acting in parallel and series with one another, the lack of vertical reinforcement which leaves the wall free to deform in vertical direction without damage and the high contribution of friction in between the stone and stone-timber in the wall.

Despite the proven seismic performance of Kath-Kuni structures, this building style seems to be no longer applied and is gradually being replaced with concrete structures, which are sometimes poorly designed. The benefits and disadvantages related to the sustainability of the Kath-Kuni building style are qualitatively investigated by means of literature survey and interviews with local people and categorised under the parameters of performance, service life and environmental impact of the building. Kath-Kuni buildings score high on performance in terms of earthquake resistance and thermal comfort and cultural value, compared to
a concrete building. Furthermore, the service life of a Kath-Kuni building is expected to be at least 200 years. Last, the environmental impact of a Kath-Kuni building is considered to be significantly lower than that of a concrete building, since the main materials are locally obtained and no transportation is needed. The materials are biodegradable and recyclable and the Kath-Kuni design completely circular. During the life time of the structure less emissions are expected, considering the climate comfortability of these buildings. However, the Kath-Kuni performs less because of difficulties in maintenance, lack of modern design comforts and high costs. The disadvantage in maintenance and modern design comforts can be solved relatively easily by performing an architectural redesign. The arguments above led to the conclusion that the Kath-Kuni building style scores well on sustainability. However, high construction costs, due to a scarcity of the main materials (and the restrictions set by the government in obtaining these materials), has rendered building of Kath-Kuni houses economically currently unviable.

Therefore, it is recommended to investigate possibilities in modernising and re-interpreting the Kath-Kuni building style using other sustainable and affordable materials, such as bamboo, stabilized earthen blocks or lime concrete. Even though earthquake resistant construction is also feasible using concrete as main material, it is the vernacular architecture and its embedded traditional knowledge that gives character to the Himalayas, which is beneficial for tourism and livelihood. Moreover, the concrete construction is a big contributor to the total global CO₂ emissions; hence using local, sustainable and durable materials should be promoted.

Figure 0.4: Bhimakali temple complex at Sarahan (Thakkar, Morrison, & Ahtushi, 2010)
List of symbols

$\Delta m$ Elongation of the maawi body

$\alpha$ Distribution factor (for precompression load)

$\beta$ Difference between the embedment strengths in the Johansen equations

$\gamma$ Cooperation factor between two out-of-plane loaded wall beams

$\gamma_P$ Partial safety factor on permanent load

$\gamma_M$ Partial safety factor on material strength

$\gamma_V$ Partial safety factor on variable load

$\delta$ Used to denote virtual displacement $\delta u$ or virtual rotation $\delta \theta$

$\theta_1$ Rotation over height of stone layer

$\theta_2$ Rotation over height of timber beam

$\theta_3$ Rotation over height of infill piece

$\mu$ Friction coefficient

$\mu_b$ Ductility factor

$\rho$ Density of the Kath-Kuni wall

$\rho_k$ Characteristic timber density (12% moisture content)

$\rho_{mean}$ Mean timber density (12% moisture content)

$\sigma_v$ Overburden stress on top of the wall

$\chi$ Constant used in out-of-plane loaded wall calculation

$\psi$ Shape function used in out-of-plane loaded wall calculation

$\omega_n$ Natural frequency

$A_i$ Cross-sectional area of a single Kath-Kuni wall beam

$a_m$ Distance from the maawi to the wall

$d_{p1}$ Distance of maawi (widest part of taper) to the edge of the perpendicular beam

$d_{p2}$ Distance of maawi (smallest part of taper) to the edge of the perpendicular beam

$E_0$ Timber modulus of elasticity used for instantaneous loading

$E_{0,mean}$ Mean timber modulus of elasticity used for short term loading

$E_{0,05}$ Characteristic timber modulus of elasticity

$E_{0,mean}$ Mean timber modulus of elasticity perpendicular to the grain

$F$ Precompression force

$F_{fr}$ Lateral friction force capacity

$F_{k,d}$ Design shear force capacity of kadil dowel

$F_{k,b,d}$ Design shear force calculated by Johansen equations for mechanism $b = 1..6$

$F_{m,t,d}$ Design tensile capacity of the maawi at the height of the wall

$F_{m,t,d,real}$ Design tensile capacity of the maawi connection (due to embedment failure)

$F_{c}$ Maximum allowable shear force

$F_V$ Applied precompression in vertical actuators

$F_{v,n,d}$ Vertical design force on the beams and infill pieces at perpendicular beam locations $n = 1..4$

$F_{v,tot}$ Total precompression force in the infill pieces per layer

$f$ Factor of safety applied to go from ultimate timber stress to permissible/working timber stress (IS 883:1994), added suffixes 1..6, for the different safety factor components (Table 3.1)

$f_c$ Concrete compression strength

$f_{c,0}$ Compression strength of timber parallel to the grain, added suffix $m$ for mean value, $k$ for characteristic value and $d$ for design value

$f_{c,90}$ Compression strength of timber perpendicular to the grain, added suffix $m$ for mean value, $k$ for characteristic value and $d$ for design value

$f_d$ Design timber strength property

$\mu_k$ Characteristic timber strength property
Bending strength of timber, added suffix \( m \) for mean value, \( k \) for characteristic value and \( d \) for design value

\( f_{m} \)

Mean permissible/working timber stresses

\( f_{m,p} \)

Used mean timber properties

\( f_{m,u} \)

Mean ultimate timber stresses

\( f_{m,u} \)

Tensile strength of timber parallel to the grain, added suffix \( m \) for mean value, \( k \) for characteristic value and \( d \) for design value

\( f_{t,0} \)

Tensile strength of timber perpendicular to the grain, added suffix \( m \) for mean value, \( k \) for characteristic value and \( d \) for design value

\( f_{t,90} \)

Shear strength of timber parallel to the grain, added suffix \( m \) for mean value, \( k \) for characteristic value and \( d \) for design value

\( f_{s,1} \)

Rolling shear strength of timber, added suffix \( m \) for mean value, \( k \) for characteristic value and \( d \) for design value

\( G_{\text{mean}} \)

Timber shear modulus

\( h \)

Height of the Kath-Kuni wall

\( h_{b} \)

Height of beam in Kath-Kuni wall

\( h_{b}^{*} \)

Height of infill piece

\( h_{i} \)

Height of floor \( i \) measured from the base

\( h_{k} \)

Height of the kadil dowel

\( h_{l} \)

Distance in height of the point of application of the lateral load and the top of the wall

\( h_{m} \)

Height of application of maanwi from the bottom longitudinal beam

\( h_{s} \)

Height of the stone/concrete layer

\( h_{x} \)

Height above considered layer, where \( x \) is the number of layers above the considered layer

\( I \)

Importance factor

\( I_{yy,y} \)

Moment of inertia, where \( y = 1 \) denotes full cooperation and \( y = 0 \) denotes no cooperation between the out-of-plane loaded wall beams

\( j \)

Number of floors where the masses are located

\( K \)

Stiffness

\( K_{el} \)

Elastic stiffness of bilinear curve

\( k \)

Exponent in vertical force distribution equation, is 1 for linear distribution and 2 for quadratic distribution

\( k_{mod} \)

Modification factor for load duration and moisture content

\( l \)

Length of Kath-Kuni wall, added suffix 1..4 denote length of the walls 1..4 (Figure 5.1)

\( l_{m} \)

Total length of maanwi

\( l_{m,taper} \)

Length of maanwi taper

\( M_{k,a,d} \)

Kadil design moment capacity for \( a = 1..3 \) (connection options)

\( M_{k,ld} \)

Design bending moment capacity of the kadil dowel itself

\( m \)

Seismic mass

\( Q_{i} \)

Design lateral force at floor \( i \)

\( q \)


\( R \)

Behaviour factor IS 1893 (Part 1) for full building assessment (IS 1893 (Part 1), 2002)

\( R_{l} \)

Behaviour factor of out-of-plane loaded wall

\( S \)

Snow load

\( S_{al/g} \)

Spectral acceleration coefficient

\( T \)

Time period

\( t_{m} \)

Thickness of maanwi

\( u \)

Displacement of the layer relative to the layer below

\( u_{el} \)

Bilinear yield displacement

\( u_{u} \)

Bilinear ultimate displacement

\( V_{b-building} \)

Maximum base shear force capacity of the building

\( V_{e} \)

Elastic shear force capacity in bilinear curve

\( V_{h} \)

Horizontal lateral force capacity of an in-plane loaded wall layer

\( V_{\text{max}} \)

Maximum shear force of capacity curve
\( V_{\text{max-wall}} \)  Maximum shear force capacity of the Kath-Kuni wall
\( V_r \)  Maximum shear force as consequence of rocking
\( V_{\text{sl}} \)  Maximum shear force as consequence of bottom sliding
\( V_t \)  Maximum shear force as consequence of toe crushing
\( V_u \)  Ultimate shear force capacity in bilinear curve
\( W \)  Self-weight of the Kath-Kuni wall or building
\( W_i \)  Seismic weight of floor \( i \) in building
\( W_T \)  Weight of top steel system in test setup
\( w^* \)  Maximum displacement criteria of rotating perpendicular beam at location \( n = 1 \)
\( w \)  Width of Kath-Kuni wall
\( w_p \)  Width of beam in Kath-Kuni wall
\( w_k \)  Width of the \( kadil \) dowel
\( w_l \)  Length of the longitudinal infill piece
\( w_m \)  Width of the \( maanwi \) body
\( x \)  Distance over which toe crushing is happening
\( x_n \)  Maximum eccentricity of pre-compression force on beam at location \( n \)
\( x_{n,l} \)  Maximum eccentricity of pre-compression force on long infill piece at location \( n \)
\( Z \)  Zone factor, is for the maximum considered earthquake and service life of structure in a zone
\( z \)  Distance of the centre of gravity of the individual beam to the centre of gravity of the total wall cross section

**Acronyms**

- COV  Coefficient Of Variation
- DBE  Design Basis Earthquake
- DSHA  Deterministic Seismic Hazard Assessment
- ELF  Equivalent Lateral Force
- LCA  Life-Cycle Assessment
- LS  Load Step
- LVDT  Linear Variable Differential Transformer
- MCE  Maximum Considered Earthquake
- PGA  Peak Ground Acceleration
- PSHA  Probabilistic Seismic Hazard Assessment
- RP  Return Period
- St. Dev.  Standard Deviation
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Figure 0.4: Himalayan and Karakoram mountain range – highlighted: Western Himalayas (Adapted from: Himalayas World Map, 2018)
1. INTRODUCTION

For centuries, the Himalayan architecture was dominantly built with building materials that could be found in the direct surrounding. With these materials, people learned and developed architectural designs that are well suited in the local climate and could withstand regional hazards. One of these traditional evolved building styles is the subject of this Master’s thesis. This traditional building method named “Kath-Kuni” is historically used for both temples and residences in the area and its efficiency was empirically proved by its ability to survive significant seismic events in the region, while also providing a thermally comfortable living space. Nowadays, these vernacular buildings are slowly replaced by poorly designed/executed concrete buildings, as a consequence of scarcity of the main materials (resulting in high costs) used in Kath-Kuni buildings. This chapter will highlight the relevance, objectives and methodology of this project.

1.1. Earthquakes in the Indian Himalaya

See figure 0.4 for a sketch of the location of the Himalayas, with the Indian Western Himalayas highlighted, which is the main focus area of this research.

The Himalaya mountain range began to form about 40 to 50 million years ago when the two large landmasses of the Indian plate and the Eurasian plate began to collide. The plates did not subduct, but both landmasses moved upwards, since they both have about the same rock density. Still, the Indian plate is putting pressure on the Eurasian plate and enormous stresses are build-up, which are released by earthquakes (Kious & Tilling, 1996).

(a) Indian Plate collision with Eurasia Plate about 40 to 50 million years ago

(b) Meeting of the two continental plates

Figure 1.1: Formation of the Himalayas (Kious & Tilling, 1996)
Several devastating earthquakes have occurred in the Indian Himalaya in the past, for example the Kangra earthquake in 1905, which killed more than 20,000 people. Recently, India has been subjected to several more earthquakes, like the Uttarkashi earthquake in 1991 (6.8 Mw), Sikkim earthquake of 2011 (6.9 Mw) and the Imphal earthquake in 2016 (6.7 Mw). Not far away from the Indian Western Himalayas and located in the same Himalayan range, Nepal was hit by the devastating Gorkha earthquake in 2015. With a magnitude of 7.8 Mw it caused an enormous amount of damage and killed many people. All these earthquakes are a consequence of the same tectonic movement explained in figure 1.1. The traditional Kath-Kuni architecture, investigated in this Master’s thesis is mostly found in the Western Himalayan range in the provinces Himachal Pradesh and Uttarakhand and in the Karakarom mountain range, which is located just above the Indian Western Himalayas and spanning the borders of Pakistan, India and China. Important findings from literature research to the Kath-Kuni building style in the Karakoram mountain range are also considered in this thesis. The two main fault lines crossing Himachal Pradesh and Uttarakhand are shown in figure 1.2.

![Image](image1.png)

(a) Research area (Mridula, Sinvhal, & Wason, 2016)

(b) Recent epicentres in the research area around the main boundary thrust and the main central thrust (Mridula S., Sinvhal, Hans, & Rajput, 2016)

Figure 1.2: Seismic fault lines in research area

Mridula S., Sinvhal, Hans, & Rajput (2016) studied the seismic hazard of a great earthquake using two different methods: deterministic seismic hazard assessment (DSHA) and probabilistic seismic hazard assessment (PSHA). The large study area, defined by the coordinates 73–80°E and 29–36°N (Figure 1.3), is centred on the epicentre of the Kangra earthquake of 1905.

According to the DSHA method calculated by Mridula, Sinvhal, Hans, & Rajput (2016), the highest Peak Ground Acceleration (PGA) of 0.581g was observed in Kangra district.

A return period (RP) of the seismic event of 475 years is equivalent to a 10% probability of exceedance in 50 years (yearly probability of 0.002) and a RP of 2475 years is equivalent to 2% probability of exceedance in 50 years (yearly probability of 0.0004). The PGA for RPs of both 475 and 2475 years, were calculated using the PSHA method. The main advantage of PSHA is that it allows uncertainties in the size, location, and rate of recurrence of earthquakes and in the variation of ground motion characteristics with earthquake size and location to be explicitly considered in the evaluation of seismic hazard. In table 1.1 an overview of the PGA in the Western Himalaya area is given.
The seismic hazard estimation in India is currently based on dividing the country into four seismic zones (II, III, IV and V) as in the Indian Seismic design code IS 1893 (Part 1):2002 (IS 1893 (Part 1), 2002). The code uses a deterministic approach to obtain the PGA values, with no recognition for various uncertainties, where they covert the PGA into a design factor $Z$. The code design $Z$-factor for the Maximum Considered Earthquake (MCE) is 0.36 and the $Z$-factor of the Design Basis Earthquake (DBE) is 0.18 for the service life of structure, for the most seismic prone area, Zone V. For zone IV, these values are 0.24 and 0.12 respectively. A factor 2 is applied between the MCE and DBE. The zone factor for MCE is considered to be equivalent to a PGA with RP of 2475 years used in a Limit State approach and the zone factor for DBE is considered to be equivalent to a PGA with RP of 475 years (see paragraph 1.5 for more information). To go from design PGA to a realistic PGA which can be used for a Limit State assessment, a load factor 1.5 is applied. In Eurocode 8, EN 1998-1:2004 (EN 1998-1, 2004) and EN 1998-3:2005 (EN 1998-3, 2005), calculations are only performed with PGAs with a certain probability of exceedance and no reduction factor is applied for extreme loading conditions.

According to EN 1998-3, the following RPs can be linked to the different Limit State assessments:

- Near Collapse – 2475 year RP
- Significant Damage – 475 year RP

<table>
<thead>
<tr>
<th>Seismic zone</th>
<th>DSHA</th>
<th>PSHA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indian Code</td>
<td>DBE</td>
<td>MCE</td>
</tr>
<tr>
<td>IV</td>
<td>0.18 g*</td>
<td>0.36 g*</td>
</tr>
<tr>
<td>V</td>
<td>0.27 g*</td>
<td>0.54 g*</td>
</tr>
<tr>
<td>Literature</td>
<td>0.0039 - 0.581 g</td>
<td>0.0039 - 0.289 g</td>
</tr>
</tbody>
</table>

* Safety factor of 1.5 is applied on DBE and MCE.
1.2. Relevance and sustainability

1.2.1. Himalayan heritage

A large portion of the traditional building stock in the Indian Himalayas (and also in the Karakoram mountain range in Northern Pakistan (Figure 1.5), near the Chinese border and parts of Afghanistan (Hughes, 2000) (Langenbach, 2015) is represented by the “Kath-Kuni” style of building. Kath-Kuni literary means “wooden corner” in the local Himachal language. Kath-Kuni buildings are never conventionally engineered, but a consequence of progressive design optimisation, built with knowledge of local people and improved over centuries of time. The Indian Himalaya has always been an earthquake prone area and inhabitants of the mountains learned to deal with this unavoidable natural hazard by improving the techniques in their local architecture. As a consequence, apparent earthquake resilient structures evolved. Provided that the structures are well maintained, centuries old Kath-Kuni buildings are robust and well preserved, which shows this building’s high durability.

Figure 1.5: Details of Cator and Cribbage Construction in Pakistan (Hughes, 2000)

Nevertheless, there is an increased trend in building on hilly slopes using new building techniques like reinforced concrete (Figure 1.6). The traditional Kath-Kuni building method is nowadays not so often build and building knowledge slowly gets lost, mostly due to the scarcity of the traditional building materials. Nowadays, local carpenters are influenced by modern techniques, which might not always be an improvement. Besides the reduction in newly build Kath-Kuni buildings, many existing (heritage) buildings are being abandoned by families who prefer modern architecture over their traditional home. The increasing population and the migration of people from villages to urban areas, is forcing people to live in residences on unstable hilly slopes.

Figure 1.6: Comparison Kath-Kuni and current concrete building practices
The “modern” multi-storey concrete buildings (Figure 1.7b and Figure 1.8a) are often poorly designed, with irregular configurations to suit the sloping terrain, which results in complex seismic behaviour. The buildings are often designed without taking seismic forces into consideration, and are prone to collapse during even moderate seismic events (Singh, Lang, & Narasimha, 2015). Research to two cities in the Indian Himalaya shows that these concrete buildings, which are designed just for vertical loading and a service life of 50-years, have an inadequate seismic resistance. Nonlinear space frame models are created of concrete buildings with the most common “step-back” configuration with a varying height between 3 and 9 stories. The building is designed for pure gravity loading according the regulations from IS 1893 (Part 1). An incremental non-linear dynamic (time history) analysis is performed using the bi-directional components of seven recorded ground motion time histories. The maximum sustained PGA turns out to be 0.03 g (Singh, Lang, & Narasimha, 2015). This is significantly lower than the demand PGA of a DBE (equivalent to a PGA with a probability of 10% of exceedance in 50 years with RP of 475 years) of 0.12 g in zone IV and 0.18 in zone V. Figure 1.8b validates this result by showing the collapse of a multi-storey building, more than 100 km from the epicentre of the Sikkim earthquake (6.9 Mw).

The increased risk due to the man-induced rapid urbanisation, which requires a fast and cheap building method like concrete, in combination with unavoidable natural hazards, like earthquakes, are compromising people’s safety significantly. In a lot of areas in the Himalayas, concrete construction is a trend that came up only approximately 25 years ago. This trend is not only reducing the climate resilience of communities in the mountains, but it is also effecting the environment. A rapid growth in the construction sector is mostly a consequence of the increased production of the materials cement, bricks and steel. In India, the construction sector is responsible for 20% of the total CO₂ emissions (Cocchi, 2018).

This raises the question: Why did people abandon their traditional building techniques and nowadays prefer to build poorly-designed concrete houses?

This question will be answered by making use of the stories from local people in several locations in the mountains and literature on vernacular Himalayan architecture in the next paragraphs.
1.2.2. Kath-Kuni building locations

The Kath-Kuni building style is also referred to as “Cator-and-Cribbage”, “Koti Banal” and “Chaukat” by other references in different mountain regions. An overview of Kath-Kuni building locations in the Western Himalayas and Karakoram mountain range, found by means of a literature survey and several field trips, is given in figure 1.9. The field and literature research areas are highlighted in a map as points, however research is usually spread over a larger areas in the districts. In every region visited during own filed work or by other researchers, multiple villages are visited.
Relevant literature research areas:

<table>
<thead>
<tr>
<th>Map</th>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2)</td>
<td>(a)</td>
<td>Architectural research to Kath-Kuni structures in several villages in the Shimla and Kinnaur district of Himachal Pradesh (Dave, Thakkar, &amp; Shah, 2013)</td>
</tr>
<tr>
<td>3)</td>
<td>(b)</td>
<td>Architectural research and initial seismic assessment in Koti Banal village in the Uttarkashi district in Uttarakhand (Rautela &amp; Joshi, 2008) (Rautela P. , Joshi, Singh, &amp; Lang, 2008)</td>
</tr>
<tr>
<td>4)</td>
<td>(c)</td>
<td>Architectural research to vernacular Himalayan building practices in several villages in the Chamba district in Himachal Pradesh (Thakkar &amp; Morrison, 2008)</td>
</tr>
<tr>
<td>5)</td>
<td>(d)</td>
<td>Field observations to earthquake resistant Cator-and-Cribbage construction in Pampore, near Srinagar (Langenbach, 2015)</td>
</tr>
<tr>
<td>6)</td>
<td>(e)</td>
<td>Research to Cator-and-Cribbage constructions in the Hunza district in Northern Pakistan (Hughes, 2000)</td>
</tr>
<tr>
<td>7)</td>
<td>(d)</td>
<td>Conservation case study to the 200-year-old (Cator-and-Cribbage) Khaplu Palace in the Ghanche district in Pakistan (Muhammad, Ali , Ghazi, &amp; Wenzel, 2016)</td>
</tr>
<tr>
<td>8)</td>
<td>(f)</td>
<td>Research to different vernacular architecture typologies in the Karakoram in Baltistan, with an example in the Skardu district in Pakistan (Hughes, 2007)</td>
</tr>
</tbody>
</table>

Figure 1.10: Relevant research areas obtained from literature
Relevant field research areas:
Several field trips are undertaken during the duration of the thesis. In each trip, several villages are visited and multiple buildings were assessed. In figure 1.11, pictures of a single building evaluated during each of the field trips are shown, where the reference numbers of the location in the map in figure 1.9 are given in between brackets.

(a) Kullu district in Himachal Pradesh: Manali, Vashist and Naggar – September 2016 (Map no. 1)
(b) Shimla district in Himachal Pradesh: Old-Jubbal and Kashani – April and July 2017 (Map no. 2)
(c) Uttarkashi district in Uttarakhand: Nandgaon and Gangtari – May 2018 (Map no. 3)
(d) Dehradun district in Uttarakhand: Mangtaad, Kandhar, Bisoi – May 2018 (Map no. 4)

Figure 1.11: Relevant field research areas related to location in figure 1.9 (source: own pictures)

1.2.3. Sustainability
Sustainability is a very wide concept and different definitions are given in the past. The definition of sustainable development that will be used in this thesis is the definition of The Brundtland Commission from their report 'Our common future' (World Commission on Environment and Development, 1987):

'Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs'

One way to qualitatively assess the sustainability of buildings is by making use of formula (1.1) of Mueller, Haist, Moffatt, & Vogel (2017). The formula is originally designed to evaluate the sustainability of different types of concrete, hence importance should be given to the service life and performance of the material/structure, besides simply looking at the amount of CO₂ emissions. The formula is in line with the three basic pillars of
sustainability; environmental aspects (planet), social aspects (people) and economic aspects (profit). The latter are represented in the service life and performance parameters in the formula. It is a simple tool to compare the two materials in terms of sustainability. In this thesis, the formula is applied to compare two construction typologies; the Kath-Kuni building method and a typical concrete construction method, which is designed purely for vertical loading, as this is common practice nowadays in the Indian Himalayas.

\[
\text{Building Material Sustainability Potential (BMSP)} = \frac{\text{Service Life} \cdot \text{Performance}}{\text{Environmental Impact}}
\]

**Service Life**: the design life time of the building.
**Performance**: the performance of the building related to safety against hazards, climate comfortability, costs and social aspects.
**Environmental Impact**: the impact the structure has over the different life stages of the building (for more information see the paragraph “planet” further in this paragraph).

During the field trips, the question "Why did people abandon their traditional building techniques and nowadays prefer to build poorly-designed concrete houses?" stated in paragraph 1.2.1, was asked to different people from different social background in the different research areas (Figure 1.12). Interviews with local people gave information regarding the benefits and disadvantages of the Kath-Kuni building style. Several research papers, which elaborate about field research to cator-and-cribbage architecture in different areas in the Himalayas and Karakoram mountain ranges (paragraph 1.2.2), likewise tried to find an answer to the above stated question. There turned out to be a general consensus in the answers in both field and literature research and they can be found in this paragraph according to the three generic pillars of sustainability.

![Figure 1.12: Interviewing local people (source: own pictures)](image)

An initial sustainability assessment performed by Dave, Thakkar, & Shah (2013) resulted in the overview shown in figure 1.13 of sustainable characteristics of the Kath-Kuni building style.

**Planet**

*Take into account the depletion of finite and (biotic and abiotic) resources and emission of compounds harmful to health and environment in the following phases:*

- **Design phase**, for example: material choices
- **Building phase**, for example: transportation
- **User phase**, for example: thermal comfort
End-of-life phase, for example recycling of materials

The main materials in the Kath-Kuni building style are Deodar timber and natural stone (slate for the roof) and both are durable and environmental friendly materials (see chapter 3 for more information about the materials used in Kath-Kuni construction). Traditionally, the building materials are obtained from the direct surrounding of the building plot and therefore almost no transportation emissions have to be taken into account. Since no mortar is used, the building is completely circular and reusable and furthermore bio-degradable.

Gauri, Mudgal, Morrison & Mayers (2004) did research to how to improve policy processes around forest use in Himachal Pradesh and the impact on poorer people. They are highlighting the fact that timber scarcity is a serious issue and changes in building design are needed to ensure continuity of the Kath-Kuni structures. They say that there is a diversion among people towards the approach to the generation of climate resilient and sustainable communities in the mountains; some promote modern building materials like cement, whereas others say timber construction should be favoured due to the high risk of earthquakes and landslides in Himachal Pradesh.

Despite the difficulties in obtaining the main materials of the Kath-Kuni buildings, interviews with local people showed that there is a general consensus about the indoor climate comfortability of the Kath-Kuni buildings (Figure 1.14). This suggest a good isolative capacity of the building and thus a low CO2 emission during the service life of the building. According to Dave, Thakkar, & Shah (2013), the shape of the roof ensures that the snow accumulates in winter which creates an insulating blanket on the roof. The thick “double” walls, filled
INTRODUCTION

with rubble in the middle and the large amount of timber used, keep the heat in the house and the high mass of the structure functions as a temperature storage, so differences temperature differences are minimalised. The balconies provide shade which hence also adds to the thermal comfort of the building throughout all seasons. Last, the kettle was traditionally kept on the lower floor in order to generate heat for the living spaces above.

From field research it could be concluded that most of the traditional Kath-Kuni buildings are more than 200 years old. Nonetheless, also a few recently (about 40/50 years ago) build Kath-Kuni houses and temples were found. Moreover, there are cases found in literature that confirm the high durability of these buildings, for example the Baltit Fort, (Northern Pakistan) which is said to be at least 800 years old and the Shigar Fort (Baltistan) which dates back at least 600 years (Hughes, 2000). Another example from Langenbach (2015) is the Khankah in Pampore, near Srinagar, dating from ca. 1600, which makes this structure more than 400 years old. This all combined indicates a high service life of Kath-Kuni buildings. Compared to concrete structures, which are designed for a 50 year duration, the materials used in a Kath-Kuni building show little degradation over time. Local people highlighted that in order to perserve the timber well, the roof needs to be kept water tight. In case of leaking, degradation of the timber is a fast process (Figure 1.15). Last, the timber needs to be maintained by regularly washing the full building with water. This is considered as a time consuming task by local house owners.

Besides being a climatically comfortable building to live in, the performance of a Kath-Kuni house is enhanced by the empirically proven earthquake resistance of the structure. A greater life time of the structure (service life) results in an increased sustainability. Several stories from people indicate how the structure is able to deform a significant amount without being damaged, which shows their great trust in these structures.

See figure 1.16, for a sketch of Kath-Kuni structures from the report of Middlemiss, an English geologist who performed a post-earthquake assessment after the Kangra earthquake in 1905 (7.8 Mw, 0.581 PGA). He stated that “Kath-Kuni performed significantly better than sun-dried brick buildings” (Middlemiss, 1910). Eye-witnesses of the Uttarkashi earthquake in 1991, explain that new build buildings collapsed, while the Kath-Kuni structures did not suffer any damage (Rautela P., Joshi, Singh, & Lang, 2008).

Modern design wishes from local people resulted in the current fashion for modern houses build with brick and concrete (Hughes, 2000). According to Hughes (2000), post-earthquake aid agencies used to introduce local
people to these “modern” construction materials and local people have stopped seeing the benefits of their traditional skills. In Pakistan, for over a year after the Kashmir earthquake in 2005, government assistance in post-earthquake construction could only be obtained when building using reinforced concrete block and slab construction techniques, even though the structures that mostly survived were the traditional timber construction technologies (Langenbach, 2015). However, empirical evidence is not always ignored. In the Uttarkashi district, it has been observed that people regained faith in the traditional building practices after the mainly poorly constructed concrete buildings got destroyed during the Uttarkashi Earthquake in 1991 (Shankar, 2006).

**People**

* Takes into account: social aspects, access to education, general well-being, job opportunities, etc.

Traditionally, Kath-Kuni structures were built to suit the lifestyle of local people. Most likely these structures where build by and for richer people in the area, where the amount of timber could be a sign of wealth. The living spaces were situated on top, with balconies to look out over their lands. The living spaces on the top floor also meant that the kitchen was located there. The completely wooden stair cases are small and steep. From the balconies, people could locate their kettle in the fields and guard for invaders; both wild animals and intruders. In some regions, along with each Kath-Kuni building, a granary used to be build and a chain was connected between the granary door and a bell on the balcony. This would alarm the residents in case people would try to steal their valuable harvest. Below the high balconies, the thick wall had just one small door, which prevents invaders from entering the buildings. These building features were contributing to the well-being of people’s lives in traditional times and it strengthened the community. Nowadays, local people still showed a general fondness of their traditional homes and its robustness; however, modernisation requires changes in design to ensure its sustainability. Many locals for example prefer the kitchen downstairs as the carrying of water and firewood upstairs is intensive and the benefits of having a kitchen on the top floor vanished.

Not many people have been able to satisfactorily modernise and renovate their traditional homes. A few recently build (last 50 years), Kath-Kuni buildings nevertheless showed a slight change in height of ceilings, size of window and door openings and change in assignment of functional spaces. Another observed trend is the “upgrading” of the traditional Kath-Kuni structures by adding concrete extensions and hiding the timber and stone materials behind a cladding of modern materials. Among local communities, the traditional houses are felt to be decreasing their image, since they associate concrete houses with wealth and modernity.

According to Dave, Thakkar & Shah (2013), building together creates bonding in community. Because the work involves carrying heavy timber logs and stones, working together is a necessity. Furthermore, the raw materials found in the direct surrounding had to be divided, for which good community relations are required. Building Kath-Kuni houses was done by use of local knowledge transferred from father to son and it provided livelihood among local people. Since concrete came from outside, the skills also needed to be obtained from other regions, which decreased the job opportunities for local people. Nowadays, vernacular Kath-Kuni structures are important for the cultural identity of the local community. Vernacular architecture provides character to the region, which is beneficial for tourism and thus generates livelihood.
Profit
*Takes into account: material, transportation and construction costs, no child labour, fair wages, healthy and safe working conditions, economic compensation of damage done to health and environment etc.*

Traditionally the main material, timber, natural stone and slate were free of cost, hence they were found in the direct surrounding of the village. Furthermore, maintenance costs are low, as the timber needs to be treated with water. Stones falling out of the wall can simply be hammered back in place. However, the traditional used materials, timber and stone are nowadays hard to obtain. Due to unregulated and illegal deforestation in the past, the government applied strict regulations on the timber felling. Hughes (2000) explains the following about the situation in the Karakoram mountain range in Pakistan related to the sustainability of the Kath-Kuni buildings: *“In Pakistan the growth timber speculators who take over forests solely for rapid profits. The villages are therefore becoming disassociated with the surrounding wood supplies.”* (Hughes, 2000, p. 5). Therefore, the sense of responsibility in local villagers in terms of sustaining and maintaining the forests is decreasing.

The scarcity of timber and stone leads to a significant increase in the price of these materials. Due to the increase costs of materials and decreased image, local household are less inclined to build in Kath-Kuni style. According to Langenbach (2015), alternative building methods have been adopted due to the depletion of the timber resources, which made Kath-Kuni buildings too extravagant: *“This may in part explain the origins of the taq and bhatar systems, where the timber lacing is limited to a series of horizontal interlocking timber bands around the building, thus requiring significantly less wood in its construction.”* (Langenbach, 2015, p.86). Furthermore, people found that using concrete and bricks is much more affordable, even though with the latter materials transportation costs need to be taken into account.

A considered disadvantage of the Kath-Kuni building style is the construction speed. Traditionally the buildings were constructed without machinery and it could take many years to complete a single house. A time-consuming aspect is the shaping of the stones, even though it is depending on the region into what extend the stones are shaped in perfect fitting blocks. Furthermore, the stones are heavy to transport, even for small distances. Concrete construction is fast compared to Kath-Kuni construction.

Furthermore, the labour costs of Kath-Kuni construction is nowadays higher as compared to the concrete labour costs. Nowadays, only a few craftsmen know how to construct Kath-Kuni buildings and are therefore costlier. Concrete labours are generally hired from outside, for example from Nepal, where the asked daily wages are lower.

To address the current obstacles in scarcity in materials, image, building speed and modern design requirements, the system needs to be re-interpreted. It is recommended in future research to perform a Life-Cycle Assessment (LCA) to compare the traditional Kath-Kuni building technology with modern alternatives in terms of environmental impact and with alternative materials used in re-interpreted designs. An LCA quantifies the environmental impact over the whole life cycle of a building in terms of monetary value.

**1.2.4. Preservation of knowledge**

There is a lot of knowledge embedded in the vernacular building techniques. As a consequence of the sustainability problems (described in paragraph 1.2.3) and the recent globalisation, which led to the availability of different construction methods, Kath-Kuni buildings are slowly vanishing from the Himalayan landscape. This results in a degradation of the traditional building practices knowledge, which has been proven to be important in creating safe and climate resilient communities in the past. As shown in paragraph 1.1, earthquakes have always been known to be a serious safety problem in the Himalayas. Because concrete construction only became a trend in the Indian Himalaya over the last few decennia and is often not designed and executed
properly, people might not realise the importance of the earthquake resistance of buildings. However, vernacular architecture is evolved over centuries of time and a consequence of a real-time design optimizing process lead by local people, the process of sticking to the traditional building methods ensured automatically that the buildings were designed taking into account the local conditions and hazards.

Gaining knowledge about the earthquake resistant features in these traditional structures, is the first step to generating conservation guidelines for Kath-Kuni buildings, which are currently not represented in the Indian Building code. In the Indian Building code, building with reinforced/confined masonry or reinforced concrete are mostly promoted as safe buildings and plenty of codes, standards and guidelines can be found for these types of buildings. According to general earthquake safe design rules, earthquake safety can be provided by providing horizontal and vertical reinforcement when brittle materials are used. Furthermore, wall openings should be kept small and it is important to design using a regular and symmetric layout, to prevent torsion. On first instance, it looks like the Kath-Kuni building method seems to fulfil these general requirements of earthquake safe design. However, scientific proof of the exact behaviour and the extend of the earthquake resistance of this building style is lacking.

At first, the Kath-Kuni structures should be assessed using the existing knowledge in IS 1893 (Part 1) (where parameters from other construction typologies can be taken). If this calculation does not show the empirically established well-performance of these structures, different parameters, calculation methods and guidelines need to be established specifically for the Kath-Kuni building method to ensure safe construction practices in the future. Taking into account the scarcity in the traditional building materials, it is important to understand the construction technology to possibly re-interpret the construction technology using materials that are currently available. This will hopefully lead to a regained interest and confidence of people using this building method.

Besides the seismic functionality of the traditional building techniques, the traditional architecture evenly evolved over the past centuries, where great attention is paid to wood carving details. Local construction practices and the artwork of local craftsman enhance the cultural identity of the area. Preserving heritage can play a key role in the attraction of tourism, which increases the livelihood in the region and will consequently lead to more prosperity for local people and for the country.

This leads to the following question:

*Can the knowledge embedded in the traditional building technologies contribute to re-generating environmentally and economically resilient communities in the mountains?*

Besides the local relevance of this research, the general knowledge embedded in earthquake resistant building techniques is not only relevant in regions with similar conditions, but applicable all over the world. Knowledge of the structural systems, which generate earthquake resistance in the Kath-Kuni buildings, will be disseminated to global knowledge platforms.

### 1.3. Objectives

The statement that Kath-Kuni construction is earthquake resistant is based on empirical results, rather than scientific research. There is almost no data of the construction details of this building method and no structural analysis has been done. To draw attention to the relevance of traditional building techniques and to preserve the vernacular knowledge embedded in the traditional building practices in the Indian Himalaya, research needs to be done to the functionality of these buildings. The main characteristics of earthquake safe building practices are; building configuration, strength, stiffness, ductility. The focus of this Master’s thesis is towards
the earthquake resistance of the Kath-Kuni building style and which of these features is represented in this traditional architecture.

The main objectives of this research are to:

a) Create awareness towards the knowledge embedded in the centuries old building techniques used in Kath-Kuni.

b) Understand the dominant features which generate lateral force capacity and wall ductility.

c) An initial understanding of the extent to which those features are contributing to the overall seismic resistance of a Kath-Kuni building.

The global objectives of the project are to:

a) Regained popularity of sustainable and climate resilient building practices in the Indian Himalaya

b) A re-interpreted design of the traditional building method (by for example investigating alternative materials) to ensure feasibility of building in Kath-Kuni style

c) Heritage conservation guidelines

This leads to the following research question:

“Which earthquake resistant features and embedded traditional knowledge of Kath-Kuni walls are essential in generating adequate seismic performance of this vernacular architecture?”

The secondary aim of this research is to investigate why people are abandoning their traditional homes and replace them by often poorly designed concrete homes and whether traditional building technologies can continue in contributing to generating climate and economic resilient communities in the mountains.

1.4. Methodology and scope

The flowchart in figure 1.18 is showing the methodology of this thesis. In order to answer the above research question, the Kath-Kuni walls need to be assessed either analytically or by performing experiments. Input parameters for both assessments are obtained from literature and from field work. Hence, the research approach is divided in four sections:

1) Construction assessment

2) Equivalent Lateral Force (ELF) assessment

3) Experimental assessment

4) Conclusions

Construction assessment

The construction assessment consists of an initial literature survey, a main literature survey and field research. The initial literature survey defines the research area. The main literature survey and different building codes establish the input parameters needed for the analytical calculations and experimental tests performed. They moreover establish the structural demand requirements in case of an earthquake hazard. The main input parameters are the behaviour factor \( R \) (describes ductility) and the time period \( T \), which will initially be taken similar to the parameters used for more standard construction methods. Furthermore, literature is used to get an initial understanding of the construction configuration and the sustainability issues related to the Kath-Kuni building style. Field work is required to obtain exact measurements of a case building in the Himalayas. The Kath-Kuni method is generally non-engineered and components will be non-uniform and different in every
building. The case building is chosen on the basis of requirements: well preserved, willingness of house owners and representative for an average residential Kath-Kuni building. Exact measurements of the building are taken to be able to determine the seismic weight and to use as input in further calculations. The connections and other construction details are evaluated, which will directly contribute to the preservation of the traditional knowledge. Last, an overview of recent changes in building practices due to modernisation requirements is established. Field research is moreover required to qualitatively assess the sustainability issues with the traditional Kath-Kuni building method. The output of these findings can be found in chapter 8.

**ELF assessment**

To perform an initial assessment of the seismic capacity of a building the ELF method is used. This method is also the first method used in the different building codes to design buildings. This method only takes the linear elastic capacity of the building into account and the dynamic effects are assessed trough a static approach. Other assessment methods exist to go into more depth of the seismic performance of structures. These methods can be divided using the graph in figure 1.17. Since no earlier research has been done to the seismic performance of these structures, the scope of this thesis is limited to linear static modelling methods.

![Figure 1.17: Modelling methods (van Wijnbergen, 2016)](image)

In order to do a full building seismic assessment, the different components of the building will be investigated separately. An important feature of earthquake resistant structures is the ability to behave as a box to ensure that the lateral force is transferred from the out-of-plane to the in-plane loaded walls. The latter have the greatest lateral force resisting capacity due to the relative high stiffness of the in-plane loaded walls. Box-generating features are stiff floor diaphragms, connectivity between the in-plane and out-of-plane loaded wall and horizontal beams in the out-of-plane walls. Hence, the first step is to check the floors in the building for its rigidity. Secondly, the out-of-plane loaded wall will be assessed for its capacity to resist lateral forces due to vibrations in its own local natural frequency. The capacity of the wall is presented as the maximum sustainable PGA for the building in its elastic design capacity. The in-plane walls provide the global stiffness to the building and will therefore be assessed for its capacity on building level. An in-plane wall model is created based on the virtual work method, taking several hypothetical failure mechanisms into account. The capacity is given as the maximum allowable PGA for which the structure is considered to be safe. Both the in-plane and out-of-plane loaded wall PGA capacity are compared to the demand PGA obtained from literature. It is expected that an elastic calculation method is able to show the seismic well-performance of the Kath-Kuni in-plane loaded walls, as non-linear second order effects are important in this direction to ensure structural integrity of the building. Therefore, the obtained behaviour factor $R$ and time period $T$ from the Indian Building code is re-investigated in greater detail. Both design calculations are performed using characteristic material characteristics. The ductility (and general non-linear behaviour) of the structure can be assessed by performing a quasi-static pushover test on a scaled down in-plane wall specimen.

**Experimental assessment**

Quasi-static pushover tests will first of all be able to generate greater comprehension of the wall capacity, especially in its non-linear range. Post failure and second order mechanisms are hard to assess without a full engineering understanding of the building. Furthermore, the experimental results will be able to give an initial estimation of the actual behaviour factor $R$ and time period $T$ for the Kath-Kuni construction method. Important to note is that these values are based on upscaling of the results, for which many assumptions need to be taken. A lower bound approach is applied and therefore this can be considered as an initial Kath-Kuni building
assessments. Further research is required to establish the parameters with more certainty. The in-plane loaded wall model designed in the ELF assessment is validated to the experimental results, where the model is scaled down to the dimensions of the experimental specimen.

Conclusions
The conclusions are divided in the following sections:

1) Field research: construction configuration and case-study building dimensions.
2) Background information: seismic design codes, material characteristics and PGA demands in the research area.
3) Out-of-plane loaded wall: maximum allowable design PGA and box-action generating features.
4) Full building (in-planed loaded wall model): maximum allowable design PGA by making use of the input characteristics obtained from IS 1893 (Part 1) and by making use of the indicative revised parameters (behaviour factor $R$ and time period $T$).
5) Quasi-static in-plane loaded wall experiment: engineering understanding of in-plane wall behaviour, strength capacity, ductility (behaviour factor $R$), stiffness (time period $T$) and failure mechanisms of tested wall.
6) Sustainability: qualitative sustainability comparison of Kath-Kuni versus conventional concrete building practices.
Figure 1.18: Scope of thesis

* Mean timber values are used for validation calculations.
1.5. Difference between the Indian Seismic Design code and Eurocode 8

The ELF method is adopted in both seismic codes; the Indian Seismic design code and the Eurocode 8 are based on elastic calculations.

<table>
<thead>
<tr>
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</thead>
<tbody>
<tr>
<td>Indian Seismic code</td>
<td>Design</td>
<td>Design</td>
<td>Assessment</td>
</tr>
<tr>
<td>Eurocode 8</td>
<td></td>
<td></td>
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</table>

The ELF method is only valid for buildings for which the dominant mode of vibration is the first natural frequency mode in each principle direction and not by higher vibration modes. Furthermore, the time period of the building should not exceed four times the upper limit of the period of the constant spectral acceleration branch (= 1.6 s for the horizontal elastic design response spectrum of IS 1893 (Part 1) and buildings should meet the criteria for regularity in elevation. Because Kath-Kuni buildings have different structural configurations over the height of the building, the lateral stiffness’s of the different stories might differ. In that case, more vibration modes than just the first mode of the building are activated and a modal analysis should be performed. However, because the lateral stiffness’s of the different stories of a Kath-Kuni building are unknown and this research is in its initial stages, it is assumed that the buildings’ first mode is most dominant and the ELF method has been chosen as modelling method.

Ductile structural systems are able to resist seismic actions in the non-linear range, which allows the structure to be designed for smaller seismic forces than the forces taken into account in a pure linear elastic design calculation. In order to take the ability of the structure to dissipate energy, through for example ductile behaviour, but at the same time avoid explicit inelastic structural analysis in design, the elastic response spectrum can be reduced by making use of the behaviour factor $q$ (Eurocode) or $R$ (Indian code).

The Limit State design approach was implemented first in the Soviet Union in 1955 and is nowadays enforced by the Eurocode for all materials. In the Limit State approach the design demand and the design capacity are compared. The actual stress-strain curve of the material is considered and the design stresses are based on a certain allowable probability of exceedance. Before the Limit State was implemented, the working stress method was used, where all the safety is implemented by applying a safety factor on the material. The working stress method only takes the linear portion of the stress-strain curve of the material into account. For timber, the IS 1893 (Part 1) is still based on the working stress method. Because the IS 1893 (Part 1) is not well developed in terms of timber construction, some aspects from the Eurocode are adopted. In Table A.1 (in appendix A) the main differences between IS 1893 (Part 1) and EN 1998-1 are given.

**Approach**

1. **Obtain material capacity values:**
   - Take material input values from Indian Building code IS 883:1994 (IS 883, 1994)
   - Covert from permissible/working stress to ultimate stress to mean stress for common size grade II
   - Derive characteristic, 5 percentile values for the material properties.

2. **Strength capacity calculation:**
   - The maximum allowable shear force in the different walls can be calculated by making use of in-plane loaded wall models and out-of-plane loaded wall models in the manners described in table 1.2.
   - Determine the maximum allowable base shear force of the building by applying both quadratic and linear lateral force distribution as bounds, hence the exact displaced shape of the building is unknown.
3. **Calculate spectral acceleration coefficient**  
   Use time period $T$ and earthquake spectrum from IS 1893 (Part 1)

4. **Determine the PGA**  
   - Use the Limit State approach from the Eurocode, EN 1990 (EN 1990, 2002), where the partial factors of safety for loads on the permanent and seismic load are considered to be 1.0 (additional imposed load is not taken into account)  
   - Implement behaviour factor $R$ and importance factor $I$

5. **Obtain demand values:**  
   Obtain PGA values from literature and the Zone factor $Z$ from the IS 1893 (Part 1) for the MCE and service life of structure in the respective zones depicted in a hazard map in IS 1893 (Part 1). The $Z$-factor is not a PGA with a certain RP.

   In this thesis it is chosen to compare the capacity PGA values calculated using the Limit State approach, with realistic demand values. The values obtained from literature are either deterministically obtained realistic PGA values or probabilistically determined PGA with a certain RP. To convert to $Z$-values used in IS 1893 (Part 1) to realistic PGA values, the author found that a multiplication factor to the MCE $Z$-values gives a comparable value to the near collapse Limit State values (near collapse: 2% probability of being exceeded in 50 years – 2475 year RP) used in EN 1998-1. Moreover, the DBE multiplied with a factor 1.5 gives comparable values with a significant damage Limit State (significant damage: 10% probability of being exceeded in 50 years – 475 year RP).
**Table 1.2: Strength capacity calculation options**

<table>
<thead>
<tr>
<th>Force capacity</th>
<th>Design EN 1998-1</th>
<th>Assessment EN 1998-3</th>
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</thead>
<tbody>
<tr>
<td>Out-of-plane loaded walls</td>
<td>Use characteristic material values</td>
<td>Use mean material values</td>
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<tr>
<td></td>
<td>Timber:</td>
<td>Timber:</td>
</tr>
<tr>
<td></td>
<td>Ductility Class – DCL (low)</td>
<td>Ductility Class – DCL (low)</td>
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<td></td>
<td>Behaviour factor ( R = q = 1 ) (conservative!)</td>
<td>Behaviour factor ( R = q = 1 ) (conservative!)</td>
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<tr>
<td></td>
<td>Leads to ( \gamma_M = 1.3 )</td>
<td>Leads to ( \gamma_M = 1.3 )</td>
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<tr>
<td></td>
<td>( k_{mod} = 1.1 ) (for very short load duration and climate class 1/2)</td>
<td>Confidence Class CF2 – See National Annexes (recommended: 1.2)</td>
</tr>
<tr>
<td></td>
<td>Design material properties:</td>
<td>Design resistance:</td>
</tr>
<tr>
<td></td>
<td>( f_d = k_{mod} \frac{f_k}{\gamma_M} )</td>
<td>( R_d = \frac{R_m}{CF \cdot \gamma_M} )</td>
</tr>
<tr>
<td>Full building (in-plane loaded walls)</td>
<td>Use characteristic material values</td>
<td>Use mean material values</td>
</tr>
<tr>
<td></td>
<td>Timber:</td>
<td>Timber:</td>
</tr>
<tr>
<td></td>
<td>Ductility Class – DCH (high)</td>
<td>Ductility Class – DCH (high)</td>
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<tr>
<td></td>
<td>Behaviour factor ( R = q = 2.5 ) (masonry walls with concrete bands)</td>
<td>Behaviour factor ( R = q = 2.5 ) (masonry walls with concrete bands)</td>
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<tr>
<td></td>
<td>Leads to ( \gamma_M = 1.0 )</td>
<td>Leads to ( \gamma_M = 1.0 )</td>
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<td></td>
<td>( k_{mod} = 1.1 ) (for very short load duration and climate class 1/2)</td>
<td>Confidence Class CF2 – See National Annexes (recommended: 1.2)</td>
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<tr>
<td></td>
<td>Design material properties:</td>
<td>Design resistance:</td>
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<tr>
<td></td>
<td>( f_d = k_{mod} \frac{f_k}{\gamma_M} )</td>
<td>( R_d = \frac{R_m}{CF \cdot \gamma_M} )</td>
</tr>
<tr>
<td>Experimental in-plane loaded wall</td>
<td>In the analytical model validation calculations, a linear capacity model of the</td>
<td>In the analytical model validation calculations, a linear capacity model of the wall</td>
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<tr>
<td></td>
<td>wall is compared to the actual behaviour of the experimental wall. Therefore,</td>
<td>is compared to the actual behaviour of the experimental wall. Therefore, mean values</td>
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<td></td>
<td>mean values of material characteristics are used in this calculation. The</td>
<td>of material characteristics are used in this calculation. The material input</td>
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<td>material input parameters obtained and adapted from the Indian Code, might</td>
<td>parameters obtained and adapted from the Indian Code, might be under or overestimated</td>
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<tr>
<td></td>
<td>be under or overestimated and hence there is no confidence factor or partial</td>
<td>and hence there is no confidence factor or partial material safety factor applied</td>
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<td></td>
<td>material safety factor applied (which are usually applied in the assessment</td>
<td>(which are usually applied in the assessment calculations in -3). EN 1998.</td>
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<td></td>
<td>calculations in -3). EN 1998</td>
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</table>

**NOTE:**

- In this thesis, only a design calculation is performed for both the full building (in-plane loaded walls) and out-of-plane wall as part of the total building.
- To calculate the vertical permanent load on the walls, 100% of the permanent load and 25% of the variable load is taken into account as done in IS 1893 (Part 1). To calculate the permanent load, mean material values are used and the partial factor of safety for the load is set on 1, according to the Limit State approach in EN 1990.
PART I - CONSTRUCTION ASSESSMENT
2. KATH-KUNI BUILDING STYLE

Previous chapter showed the relevance of investigating the seismic capacity of the traditional Kath-Kuni buildings in the Indian Himalaya. In order to research the seismic capacity, it is first of all required to obtain the exact building configuration. The Kath-Kuni building method is a vernacular construction method, which is non-standardized and based on craftsmanship. This results in differences in this building style over different regions in the Indian Himalaya. Furthermore, diversity can be observed between residential buildings and temples. This research will focus on residential buildings, in accordance with the general objective of contributing to resilient communities in the Himalayas. Field and literature research leads to a general idea of the common construction features in all Kath-Kuni buildings. Moreover, it provides information on which features are traditional Kath-Kuni and which features changed due to recent modernisation influenced by techniques and materials from outside the mountain regions. Due to the differences in building style, it is necessary to establish an average case study building. This building will be analysed in more detail and the results will be consequently used for seismic calculations.

This chapter summarises the results of the field trips undertaken in April 2017 and July 2017 to villages in the Shimla district in Himachal Pradesh and to a smaller extent the field trip in May 2018 to villages in the Dehradun and Uttarkashi district in Uttarakhand. The motivation for selecting the Shimla region in Himachal Pradesh is based on the literature from Dave, Thakkar, & Shah (2013) and Thakkar & Morrison (2008). The Faculty of Design of CEPT University Ahmedabad (India) executed several projects to investigate vernacular architecture in the Indian Himalaya. For their research of the Kath-Kuni building style they undertook several field trips to the province Himachal Pradesh, describing multiple villages with well-maintained Kath-Kuni architecture. Moreover, the field trip gives an insight in the current sustainability issues of the Kath-Kuni building style, which are described in paragraph 1.2. This chapter will focus on the building configuration, construction details and the loss of traditional knowledge.

2.1. Case study - building in Old-Jubbal

During the first field trips in the Shimla district in Himachal Pradesh, it became clear that Kath-Kuni structures are common everywhere in this area. Many Kath-Kuni houses were visited, of which one house in Old-Jubbal (Shimla district, Himachal Pradesh) was taken as an average example for Kath-Khuni structures in Himachal Pradesh (hereafter referred as ‘case study building’ for the scope of this thesis). This building was also extensively researched in an architectural way by Dave, Thakkar, & Shah (2013). This more than 200 years old structure, is very well build and maintained (Figure 2.1). It was build for an upper class family and the quality of the building construction and detailing is showing a great craftsmanship. It is a four storey high structure with the living areas still located at the upper storeys. A lot of houses in this area were build using similar techniques, however the quality of the construction and perservence was often observed to be less. The case study building is taken as example in this research and detailed measurements are taken. There is a lot of variation in the exact building configuration of the Kath-Kuni building method and the definition established according to the findings in this thesis is as follows: ‘Kath-Kuni’ is a building method where in the corners the timber horizontal beams are
connected over most of the height of the wall by making use of timber *kadils* (timber dowels) and no nails are applied in the connections.

![Image](image1.png)  
(a) Obtained by research group from CEPT university  
(Thakkar & Morrison, 2008)  
(b) Approx. 10 years later; surrounded by concrete buildings (source: own picture)

Figure 2.1: Change of case study Kath-Kuni building over past two decennia

### 2.2. Building configuration

Extensive measurements of the case study building resulted in a Sketch-Up model focussed on the structural aspects of the building. This model forms the basis of all the further work done in this thesis, an overview of this model is given in figure 2.4. The building outer dimensions are 14.8 m in the long direction by 4.8 m in the short direction at the foundation level (in figure 2.6 rough measurements are given). See appendix 0 for a comparison of the documentation of the Sketch-Up model.

![Image](image2.png)  
Figure 2.2: The cuboid concept of Kath-Kuni buildings

![Image](image3.png)  
Figure 2.3: Scale model 1:10 of principle of Kath-Kuni wall from IDEA, Ahmedabad (Shah, 2018)

Typical Kath-Kuni buildings are built up from several cuboids. The smallest house is just a single cuboid, where the Kath-Kuni house in Old-Jubbal is build up from 3 cuboids in horizontal direction and 4 in vertical direction (Figure 2.2). This means that the building has one long direction and one short direction and is four storeys high. The configuration of every cuboid is the same. The walls of the cuboid are thick Kath-Kuni walls of
approximately 0.5 meters on all sides. Two cuboids are separated by one thick Kath-Kuni wall. A Kath-Kuni wall is built up from stone and timber (mostly Deodar). Two timber beams are placed parallel on the stone layers and filled up with stone rubble in between. On the higher located storeys the Kath-Kuni walls are replaced by full timber walls with stone rubble infill (see figure 2.3 for an example of a wall section).

The four cuboids stacked in vertical direction are separated by floors into 4 storeys (and an attic). The top floor level in this Kath-Kuni house is completely made of timber, and the connections are made differently compared to the lower Kath-Kuni walls. This phenomenon was seen in more houses; the higher in the building, the more timber is used. Therefore, a completely different stiffness and force capacity is expected of this floor level. The absolute bottom storey is made solely from dry stone masonry and again a different lateral capacity is expected from this floor level compared to the floor levels of which the walls are built in the typical Kath-Kuni style. The behaviour of these aberrant walls is not investigated in this thesis research.

The lower storeys have a height of 1.5 and 1.3 meter (Figure 2.6) and mostly used for cows and storage of cow food. The upper storeys are about 1.8 meters height and used for living and storage. The attic (2.6 meters high) is not used in the case study building but are sometimes used for cooking or storage in other houses.

The roof of Kath-Kuni buildings in Himachal Pradesh are shaped in the characteristic curved manner visualised in figure 2.4. Kath-Kuni buildings in Uttarakhand have straight shaped roofs. The roofs consist of slate stone plates nailed with a single nail to the timber purlins. The purlins are spanning between the Kath-Kuni wall gables. There are no stability measures in the roof and the roof can be considered as a flexible diaphragm. At roof level the Kath-Kuni gables consist of timber and mostly filled up with stone and plaster.
In every cuboid, three timber beams (250 x 250 mm) are spanning in the short direction of the building in order to support the 6 cm thick floor. Two of the floor beams are located close (but not connected!) to the parallel walls, and one floor beam is located in the middle of the cuboid (Figure 2.2). The floor planks are connected to the floor beams by nails. Important to note is that therefore none of the floor load is carried by the Kath-Kuni walls in short direction of the building (the walls parallel to the floor beams), hence the floor beams are carrying the load to the Kath-Kuni walls in the long direction of the building. The floor beams are clamped in the thick Kath-Kuni walls in long direction of the building. From the field observations, it was assured that the beams are clamped over approximately 70% of the thickness of the wall. It seems that the floors are not able to perform as stiff diaphragms, hence the floor planks are nailed to the beams and no other stiffening measures are taken. The floors are not connected to the parallel outer Kath-Kuni walls. These walls are able to deform freely over the height of the construction in out-of-plane loaded direction. These walls need to have other box-generating mechanism in order to collapse under out-of-plane lateral loading.

On the higher floors, balconies are present, which are build up from cantilevering beams (150*150 mm) (clamped in the thick Kath-Kuni wall) and support a 6 cm thick floor. For the approximate lengths of the balcony floor beams cantilever see figure 2.6.

![Figure 2.5: Cross-section of the case study building (adapted from Thakkar & Morrison, 2008)](image-url)
2.3. Construction details

One layer in the Kath-Kuni wall consists of alternating dry stone masonry and timber beams. The dry stone masonry in the case study building is well shaped into ashlar masonry. However, stone blocks in other buildings, especially in Uttarakhand, are kept more rough. On top of the dry stone masonry, two parallel timber beams are placed. The parallel timber beams are connected with *maanwi’s* (timber dovetail connections), spaced with distance of approximately one meter. The *maanwi’s* keep the parallel timber beams together.

The corner connections in the case study building (see Figure 2.7b) differ from the corner connections shown in literature (see figure 2.7a). Kath-Kuni corners have a continuous timber connection over the height of the structure. The stacked timber beams are connected with *kadils* (timber dowels) in the corners. However, the amount of stone in between two timber layers can vary between different buildings. The height of the layer (consisting of one timber layer and one stone layer) can increase by adding timber infill pieces in the corners. In this way the timber structural integrity is maintained, and less timber is used. Locals claim that for bigger buildings, more stone in between the timber horizontal beams is beneficial as more friction between the stone blocks can occur.

![Figure 2.6: Dimensions of the case study building](image)

(a) Typical corner connection mainly observed in temples
   (Dave, Thakkar, & Shah, 2013)

(b) Corner connection in case study building

![Figure 2.7: Kath-Kuni corner connections](image)
In general, it has been observed that more timber is used for temples than for the houses. For temples the availability of the wood is higher, as for religious purposes the timber distribution regulations and especially the following up of the regulations is not as strict.

*Kadils* are thick square timber dowels of approximately 50 x 50 mm width and 100 mm high. For the case study buildings’ corner connection it means that there are either 3 or 4 *kadils* per corner per horizontal shear plane present. For the temple corner connection, always four *kadils* per plane are implemented to bring the shear force from the top of the building down to the foundation.

Another important construction detail is the connection of the longitudinal beams at the long side of the wall in between the different cuboid sections (Figure 2.2) in horizontal direction. In the analyses performed in this thesis, the cuboids in horizontal direction are assumed to behave independently of each other. However in reality, there is a connection between the horizontal located cuboids, which will slightly change the configuration of the corner connections at the location of the intermediate walls (Figure 2.8).

(a) Connection between longitudinal beams in wall with great craftsmanship  
(b) Zoomed detail of connection

![Figure 2.8: Connection between two horizontal cuboid sections (Figure 2.2) in Old-Jubbal (source: own pictures)](image)

### 2.4. Loss of traditional skills

Due to the diminishing amount of Kath-Kuni structures and therefore less skilled carpenters, the knowledge embedded in the traditional Kath-Kuni technology is slowly vanishing. Several building practices were observed during the field trips, which are not conform the traditional practices. The changes are mostly inspired on new “modern” building practices, like the use of concrete, cement and nails in the walls. Below, the most important observations are listed and supported by some pictures in figure 2.9:

- Fewer timber beams; no timber connectivity in the corner
- Different corner connection configurations
- Nails are used
- Walls filled with concrete instead of rubble
- Mortar is used for the natural stone masonry

There were a lot of differences in the traditional Kath-Kuni buildings and the buildings designed in the time span of now and 50 years ago. According to Rautela, Joshi, Singh, & Lang (2008) variations in building style of the original Kath-Kuni building style started to creep in between around 728 and 60 years ago. Examples of those changes are the bigger door and window openings, additional verandas for which support was needed, higher floor heights and fewer horizontal timber beams.
An important change is the lack of completely timber corners in some buildings (Figure 2.9a). According to Langenbach (2015) and local people, this is a consequence of the lack of timber resources. Cator-and-cribbage techniques have proven to be particularly earthquake resistant. However, in Pakistan, Nepal and parts of India, timber-laced masonry is also a frequently observed construction method, where the timber beams are intentionally spaced with a certain distance in vertical direction. These construction methods are also known as ‘Bhatar’ construction in Pakistan and ‘Taq’ construction in Nepal.

The second sign of loss of building skills is the fact that nails are used in more recent buildings in the connections. In figure 2.9c, the maanwi is connected to the timber longitudinal beam by making use of nails. Connections need to be able to have a certain flexibility in case the building is subjected to an earthquake, but also when subjected to changes in timber volume due to the variation in humidity during the different seasons. Improper connections cause cracking of wood, as can be seen in a house in the village Kashani (Figure 2.9a).

Figure 2.9b shows that the timber beams are connected at the same height to each other, instead of alternatingly interlocking in the corners. Changing the corner connection configuration, will significantly change the behaviour and capacity of the structure.
2.5. Initial structural assessment

Rautela and Joshi (2008) describe the features that contribute to the earthquake safe performance of the Koti-Banal architecture. Koti-Banal architecture is named after the village Koti-Banal in Uttarakhand. As mentioned before, this name is a regional variation for the same Kath-Kuni structures. Nevertheless, slight variations in structural principles will be there between different regions. The main difference (also observed in a field trip undertaken in May 2018), is the prevalence of a so-called shear key (Figure 2.10). The floor beams are supported by an perpendicular beam in the middle of the room, which extends through the thick outer walls parallel to the main floor beams. This perpendicular beam is fixated with the shear key over several storeys and supports the walls in out-of-plane loaded direction (Rautela P., Joshi, Singh, & Lang, 2008).

The features that are contributing to the earthquake resistance of this building method are summed up below:

- The mass and rigidity are distributed equally and symmetrically in the building, which prevents torsion.
- The structure’s high ductility, which makes that a lot of energy is absorbed due to friction.
- The elasto-plastic material behaviour of timber and the semi-rigid joints result in high power absorption capacity.
- Wood has a high strength to weight ratio.
- The horizontal wooded beams work as a group space stress system.
- The timber beams have are over-dimensional for its function as bending member.
- The presence of little and small wall openings.
- The stone plinth keeps centre of gravity low, which minimises overturning effect.

Aspects of Kath-Kuni that are not conform the general rules of earthquake resistance are:

- Flexible floor and roof diaphragms.
- No connection to the foundation.

Rautela and Joshi (2009) performed a design seismic base shear calculation according to the Indian Building code (linear-static analysis), where the total seismic weight of the building was calculated to be approximately 3109 kN. This results in a lateral base shear force of 700 kN, assuming that the building is located in seismic region V. The resulting lateral forces are shown for the 5 storeys high Koti Banal structure shown in figure 2.11.
3. MATERIALS

In the previous chapter, the construction configuration is assessed. This chapter evaluates the main construction materials used in Kath-Kuni building style. These main materials are Deodar timber, natural stone masonry and stone rubble in the walls and slate in the roof. Furthermore, extensive literature has been reviewed to investigate and assess substantial information about these materials. The material properties are obtained from the Indian timber design code IS 883 (IS 883, 1994) and from small tests performed in the laboratory of the Indian Institute of Technology Roorkee. Working stress method is still used in case of timber in IS 1893 (Part 1) (IS 1893, 2002), where all safety is applied on the material capacity. The working stress timber material characteristics are converted to mean and characteristic values that can be used in a Limit State calculation, as performed in the Eurocode.

3.1. Timber

Timber is one of the main resources needed to build in Kath-Kuni style. The timber that is most often used for construction purposes is Deodar (Cedrus deodara) and Kail. Deodar is an evergreen coniferous tree that is originating from the Himalayas. According to Dave, Thakkar, & Shah (2013) Deodar, growing at high altitudes, can grow up to 50 meters in height and 3 meters in width. It is known to possess good durability and weather rot resistance. Moreover, its oil acts as an insect repellent. Folk stories say that Deodar may last up to 1000 years in water and 10 times that long in air (Dave, Thakkar, & Shah, 2011). EN 350 (EN 350, 2016) classifies Deodar as a Durability Class 1-2 of wood and wood-based materials to attack by decay fungi. Furthermore it scores as “durable” against attack by wood-boring beetles, “durable” against attack by termites and “moderately durable” against attack by marine organisms.

According to Gupta (2014) Himachal Pradesh has a forest cover of 67%. In figure 3.3 forest distribution by canopy density is shown. Even though a considerable portion of the state is under forest cover, as depicted by figure 3.2, the state administration has laid down strict regulations against cutting down of trees. Since 1865, any land with trees has been claimed to be a government forest. This, in turn, leaves the local people with a limited access to these rich forests, who otherwise, had been using them unrestrainedly. The villagers are disassociating from the forests and sustaining them. Conflicts between the government and villagers about the forest management was essential in the massive deforestation in Uttarakhand (Awasthi, Uniyal, & Rawat, 2001).

According to Himachal Pradesh Forest Department (2017), at present the forests are not being looked at as a source of revenue and sustained supply of raw material. Rather, the emphasis lays upon on protection and conservation of forests, environment and wild life. The removals from forests are therefore limited to removal of dead, diseased, decaying trees and salvage lots and removals for meeting bonafide requirements of the local people.
The ‘Himachal Pradesh Forest (Timber Distribution to the Right Holders) Rules, 2013’ (Government of Himachal Pradesh Department of Fores, 2013) lists down the regulations to be followed to distribute timber amongst the locals. This document states that for new construction 7 m$^3$ of timber is available every 20 years to inherited land owners in HP. For repair and maintenance work this is 3 m$^3$ every 10 years. The timber can only be obtained from salvage (fallen, dry standing) trees.

The timber characteristics are obtained from IS 883, which uses the working stress method to check the structural components. The allowable stresses (working stresses/permissible stresses) are obtained from the Indian code and converted to mean and characteristic stresses (5% lower limit) for common size grade II timber. These stresses can be used in the Limit State approach applied according to EN 1990 (EN 1990, 2002). For the design calculations, characteristic material values are needed, whereas for the validation assessment of the analytical model to the experiment mean material properties are used (Table 1.2). Appendix B explains how the timber properties obtained from IS 883 are derived.

Assumptions:
- Grade II timber (strength degradation between 25 and 37.5% compared to small clear specimen).
- The other safety factors (f2, f3 and f5) are not taken into account in the used mean and characteristic values. This is partly taken into account by $k_{mod}$ in the Limit State approach according to EN 1998-1.
The fundamental stresses (ultimate stresses) are the mean value stresses obtained from testing on small clear specimen. The permissible (working stresses) stresses for deodar obtained from IS 883 are given in the column one in table 3.2. From permissible to ultimate stresses the following step is taken:

\[ f_{m,ult} = f_{m,p} \]

Where \( f \) is the total safety factor that can be found in figure 3.4. This total safety factor includes the variability of the timber properties, the uncertainty in the load and the duration and climate influences on the timber characteristics.

The remaining ultimate capacity values are derived using EN 384, table 2 (EN 384, 2016). Wherever needed, first the characteristic values are calculated and converted to mean values by using the probability factor \( f_1 = 0.72 \). This factor represents a Coefficient Of Variation (COV) of 0.17 \( (f = 1 - 1.645 CV) \). For the used values \( f_{m,u} \), the mean values are multiplied by a grade factor to take into account the strength-reduction of a normal size grade II (“common grade”) specimen:

\[ f_{m,u} = f_{m,ult} \cdot f_4 \]

To convert ultimate stresses to characteristic stresses the factor \( f_1 = 0.72 \) is used.
Table 3.2: Deodar timber properties

<table>
<thead>
<tr>
<th></th>
<th>Permissible IS 883</th>
<th>Ultimate(^4)</th>
<th>Used mean(^3)</th>
<th>Used char.(^3)</th>
<th>Eurocode C24</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Grade I, 5% fractile, inside</td>
<td>Small clear, Mean value</td>
<td>Common size, Grade II, Mean value</td>
<td>Common size, Grade II, 5% fractile</td>
<td>Common size, Mean values(^1)</td>
<td></td>
</tr>
<tr>
<td>Density-char (12% moist.)</td>
<td>(\rho_k)</td>
<td>-</td>
<td>464(^2)</td>
<td>464</td>
<td>464</td>
<td>350</td>
</tr>
<tr>
<td>Density-mean (12% moist.)</td>
<td>(\rho_{mean})</td>
<td>557</td>
<td>557</td>
<td>557</td>
<td>557</td>
<td>420</td>
</tr>
<tr>
<td>Modulus of elasticity //</td>
<td>(E_{0,mean})</td>
<td>9480(^*)</td>
<td>11850</td>
<td>11850</td>
<td>11850</td>
<td>11000</td>
</tr>
<tr>
<td>Modulus of elasticity //</td>
<td>(E_{0,05})</td>
<td>-</td>
<td>7940(^2)</td>
<td>7940</td>
<td>7940</td>
<td>7400</td>
</tr>
<tr>
<td>Modulus of elasticity (\perp)</td>
<td>(E_{90,mean})</td>
<td>-</td>
<td>316(^2)</td>
<td>316</td>
<td>316</td>
<td>370</td>
</tr>
<tr>
<td>Bending/tension/extreme fibre strength</td>
<td>(f_\text{m})</td>
<td>10.2</td>
<td>61.2</td>
<td>38.3</td>
<td>27.6</td>
<td>40.8</td>
</tr>
<tr>
<td>Tension strength //</td>
<td>(f_{1,0})</td>
<td>-</td>
<td>40.4(^3)</td>
<td>25.3</td>
<td>18.2</td>
<td>23.8</td>
</tr>
<tr>
<td>Tension strength (\perp)</td>
<td>(f_{1,90})</td>
<td>-</td>
<td>-</td>
<td>2.0(^6)</td>
<td>1.4</td>
<td>-</td>
</tr>
<tr>
<td>Compression //</td>
<td>(f_{c,0})</td>
<td>7.8</td>
<td>39.6(^2)</td>
<td>24.8</td>
<td>17.9</td>
<td>35.7</td>
</tr>
<tr>
<td>Compression (\perp)</td>
<td>(f_{c,90})</td>
<td>2.7</td>
<td>4.5(^2)</td>
<td>2.8</td>
<td>2.0</td>
<td>4.2</td>
</tr>
<tr>
<td>Shear //</td>
<td>(f_{01})</td>
<td>1.0</td>
<td>5.5(^2)</td>
<td>3.4</td>
<td>2.4</td>
<td>6.8</td>
</tr>
<tr>
<td>Shear (rolling)</td>
<td>(f_{02})</td>
<td>-</td>
<td>-</td>
<td>2.0(^7)</td>
<td>1.4</td>
<td>-</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>(G_{\text{mean}})</td>
<td>-</td>
<td>741(^2)</td>
<td>741</td>
<td>741</td>
<td>690</td>
</tr>
</tbody>
</table>

1) Mean values calculated with a COV of 0.25 from the characteristic C24 values in the Eurocode.
2) Derived from EN 384, table 2.
3) Use \(f_1 = 0.72\) (taking into account the probability distribution) to change from characteristic to mean value (COV = 0.17).
4) Ultimate values are derived from permissible values according to total safety factors in figure 3.4.
5) All values are derived from the ultimate values using a factor \(f_4 = 0.625\) for timber grade II (Table 3.1).
6) Obtained taking an average tensile strength for Cedar (1.5 Mpa – 2.5 Mpa) (Kretschmann, 2010).
7) Obtained from (Ehrhart, Brandner, Schickhofer, & Frangi, 2015).

\* This modulus of elasticity value is based on small clear specimen

NOTE: Compression \(\perp\) will always happen before shear \(\perp\) and is, therefore, not implemented in this table
3.2. Stone/concrete

Kath-Kuni buildings use different types of stone in different regions. The shape of the dry stone masonry varies from neatly cut ashlar to rough rubble. The stones used for the case study building under consideration are stone ashlar and this technique, is therefore, used in the tests. As a consequence of the recent dearth of craft skills for working with vernacular building materials, there has been a decline noticed in the workforce skilled in stone shaping. Nowadays, using a slate type stone for the dry-stone masonry infill is more in practice, as this stone is easier to cut with machines. Nevertheless, stone shaping with machines is still a type consuming process and according to local craftsmen it usually takes one day for one person to shape approximately 4 to 5 stones.

However, similar to timber, the government also maintains restrictions on stone mining. This makes it hard for local people to access this construction material for their buildings. Due to these difficulties in obtaining natural stone for the experimental specimen covered in this thesis, concrete blocks are used. A strength of 30 MPa is chosen for the minimum strength of the concrete blocks, as this concrete strength is generally used for construction purposes in India. The outcome of this experiment can be compared to future tests performed with natural stone, which can lead to an indication if concrete blocks could possibly be a good replacement material for the scarce natural stone. The concrete blocks are made with concrete of grade M35 – having a characteristic strength of minimum 35 MPa after 28 days. The higher grade of concrete is chosen because the aim is to have a minimum strength of 30 MPa at the day of testing. The blocks were used before the 28 days, due to time limitations. Apparently, instead of a standard curing process that involves immersion in water for a 28 day period, the concrete blocks were coated with a curing compound. This allowed usage of the blocks well before the 28 day period.

Stone has a much higher compressive strength (between 110-250 MPa for granite). Thus, crushing of the stone during the pushover test is very unlikely. Kath Kuni building is expected to show similar behaviour using either stone or concrete. In absence of concrete crushing during the tests, a compressive strength of 30 MPa would also be sufficient for the infill material in the Kath-Kuni wall. This would be a start for approaching more alternative materials for the re-interpretation for the traditional structure.

The concrete blocks were made of size 350 mm long by 220 mm wide by 100 mm thick. Sample blocks were kept in different concrete pours and were tested the day after conducting the main lab test to evaluate the achieved strength. These blocks were cut in two pieces of approximate size 175 mm x 220 mm x 105 mm and nine such blocks were tested. The test was performed in a universal compression machine with a displacement control rate of loading of 0.05 mm/s. This rate of loading confirms to a quasi-static test and the results can be found in Table 3.2. The day 1 poured concrete blocks show an average strength of 59.02 MPa, the day 3 poured blocks indicate an average strength of 50.29 MPa and the day 5 poured blocks indicate an average strength of 45.51 MPa. The average results achieved are well above the minimum target strength of 30 MPa. While the values depicted in table 3.3 do not correspond to the characteristic strength of concrete f_{c,28}, they indicate a lower bound for these values. The blocks would indicate that the minimum strength achieved is about 45 MPa for the last cast blocks and the first cast blocks give a much higher value of about 59 MPa.

The wall has also been built using graded blocks with the lower layers of the wall been built using the earlier cast blocks and the higher layers with those cast later. For M35 concrete, these results are significantly higher than expected and this can, in part, be attributed to the scale of the blocks being tested. M35 is standardized against a concrete cube of size 150mm x 150mm x 150mm. The significantly smaller thickness of the block would indicate a much higher value of stress which is seen from the results.
Table 3.3: M35 concrete blocks tested for IITR_PILOT_02

<table>
<thead>
<tr>
<th>Block number</th>
<th>Pour day</th>
<th>Testing date from day of pour</th>
<th>Load (kN)</th>
<th>Size of block</th>
<th>Compressive strength fc (MPa)</th>
<th>Mean</th>
<th>St. Dev.</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>19</td>
<td>1852.4</td>
<td>224x180x105</td>
<td>45.94</td>
<td>45.51</td>
<td>0.44</td>
<td>0.01</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>19</td>
<td>1794.8</td>
<td>224x176x105</td>
<td>45.52</td>
<td>45.07</td>
<td>0.66</td>
<td>0.01</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>19</td>
<td>1804.8</td>
<td>220x182x105</td>
<td>45.07</td>
<td>50.29</td>
<td>0.66</td>
<td>0.01</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>21</td>
<td>2029.8</td>
<td>224x180x105</td>
<td>50.34</td>
<td>50.92</td>
<td>0.66</td>
<td>0.01</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>21</td>
<td>2012.2</td>
<td>222x178x105</td>
<td>50.92</td>
<td>50.29</td>
<td>0.66</td>
<td>0.01</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>21</td>
<td>2062.4</td>
<td>226x184x105</td>
<td>49.60</td>
<td>59.02</td>
<td>0.77</td>
<td>0.01</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
<td>23</td>
<td>2316.4</td>
<td>220x176x105</td>
<td>59.82</td>
<td>59.02</td>
<td>0.77</td>
<td>0.01</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>23</td>
<td>2324.2</td>
<td>224x178x105</td>
<td>58.29</td>
<td>59.02</td>
<td>0.77</td>
<td>0.01</td>
</tr>
<tr>
<td>9</td>
<td>1</td>
<td>23</td>
<td>2287.4</td>
<td>218x178x105</td>
<td>58.94</td>
<td>59.02</td>
<td>0.77</td>
<td>0.01</td>
</tr>
</tbody>
</table>

3.3. Rubble

As described in the previous chapter, the walls are filled with natural stone rubble. This is basically the left over from the stone shaping, explained in paragraph 2.2. There are no parameters known for rubble. The contribution of rubble is neglected in the analytical models.

3.4. Material densities

The densities of the different materials are given in the table 3.4.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Density [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deodar (12% moisture)</td>
<td>560</td>
</tr>
<tr>
<td>Slate stone (roof)</td>
<td>2750</td>
</tr>
<tr>
<td>Natural stone</td>
<td>2590</td>
</tr>
<tr>
<td>Concrete C35</td>
<td>2500</td>
</tr>
<tr>
<td>Dry rubble</td>
<td>2040</td>
</tr>
<tr>
<td>Lime plaster</td>
<td>1730</td>
</tr>
</tbody>
</table>

3.5. Friction coefficients

The Coulomb Friction model is used for evaluating frictional resisting capacity of shear force. The formula used is: \( V \leq \mu W \), where \( \mu \) is the friction coefficient, \( V \) the shear force and \( W \) the resultant normal force. Different values for the friction coefficient of dry stone masonry are found in literature. Experiments done on single dry-stone masonry joints give values around \( \mu = 0.60 \), while for complete shear walls, the friction value is measured to be \( \mu = 0.32 \). However, these joint values and wall values of the friction coefficient have different physical meanings (Lourenço, Oliveira, Roca, & Orduña, 2005) (Lourenço & Ramos, 2004).

The timber friction coefficient of softwood species with a specific gravity of approximately 0.50 (specific gravity of Deodar is 0.56) is found to be 0.24 (static) (Hirai, et al., 2008). For timber log-houses, the timber friction coefficient is found to be around 0.52 – 0.665 (Bedon, Fragi, Amadio, M.ASCE, & Sadoch, 2014). Several friction coefficients obtained from literature by Aira, Arriaga, Iniguez-Gonzalez, & Crespo (2014) are given in
Their research on Scots pine (*Pinus sylvestris* L.) found friction coefficient of 0.24 between transverse surfaces of 0.24 and 0.12 for friction between radial surfaces were found.

![Figure 3.5: Several friction coefficients obtained from literature (Aira, Arriaga, Íñiguez-González, & Crespo, 2014)](image)

The final friction coefficient assumed in this thesis is 0.35, for the friction between all different materials; timber-timber, timber-stone and stone-stone.
PART II – EQUIVALENT LATERAL FORCE ASSESSMENT
4. STATIC INITIAL CALCULATIONS

This chapter forms the basis for the seismic Equivalent Lateral Force (ELF) analysis performed in chapter 5. In chapter 2 the Kath-Kuni building configuration is explored and the measurements of one well preserved residential building in Old Jubbal are taken to perform a weight calculation. The mean material parameters from chapter 3 are used as input parameters. Furthermore, the floors and roof of the building are assessed for its rigidity. Rigid floor will enhance the box-working of the building, which is beneficial for the seismic performance of the building. In case of flexible floor diagrams, the out-of-plane loaded wall is more prone to failure and needs to be assessed over the full height of the building.

4.1. Weight calculation

4.1.1. Input parameters

The weight calculations are supported with 3D sketches from Sketch-Up, where the load path considered is visualised (see appendix D, for the full weight calculation performed in Excel). Besides the live load, the self-weight of the structure is taken into account. Note that it is assumed that none of the weight is taken by the column under one of the diagonal corner beams supporting the balcony (Figure 2.4). This column is neglected in all calculations. Furthermore, the attached balconies (visible on the photo in Figure 2.1), used as wash and toilet rooms, are neglected. These rooms are extra supported by columns and their weight is considered negligible.

The following loads are assumed to act on the structure according to IS 875 (Part 2) (IS 875 (Part 1), 1987) and IS 875 (Part 4) (IS 875 (Part 4), 1987):

**Dead load**
See the densities in paragraph 3.4.

**Live load**

**Residential building – Dwelling**

<table>
<thead>
<tr>
<th>Load</th>
<th>Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>All areas inside</td>
<td>1.75 kN/m²</td>
</tr>
<tr>
<td>Balconies</td>
<td>2.5 kN/m²</td>
</tr>
<tr>
<td>Roof &lt; 10°</td>
<td>0.8 kN/m²</td>
</tr>
<tr>
<td>Roof &gt; 10°</td>
<td>0.75-0.02x kN/m²</td>
</tr>
<tr>
<td>Top of 60° roof</td>
<td>0.0 kN/m²</td>
</tr>
</tbody>
</table>

**Snow/rain**

Indian building Code, $S_0$ -

Eurocode, $S_0$ 0.7 kN/m²

Snow pressure $S_0$ 1.0 kN/m² 50 cm snowfall with 20% moisture content, 200 kg/m³
Slope of upper roof: 60°
Slope of lower roof: 8°

\[ S = \mu S_0; \text{ linear interpolation in between the following values:} \]

- \( S_{\text{upper}} \): 0 kN/m²
- \( S_{\text{transition}} \): 1.1 kN/m²
- \( S_{\text{lower}} \): 0.8 kN/m²

The live loads on the floors in inside areas in the Kath-Kuni building are adapted from 2 kN/m² to 1.75 kN/m². The balcony load is adapted from 3 kN/m² to 2.5 kN/m² in accordance with NEN-EN 1991-1-1 (NEN-EN 1991-1-1+C1, 2011). These live loads on floors in inside areas and balconies are assumed for residential buildings, hence the live loads assumed by the Indian code seem to be overestimated for a Kath-Kuni building.

In order to calculate the snow load on the roof, a snow fall of 50 cm on the ground is considered, which is equivalent to 1 kN/m² of snow pressure. Due to the shape of the roof, there is a chance of snow accumulation. This is taken into account with the S-factors. At the top of the roof, where the slope is higher than 60°, the snow load is assumed to be 0 kN/m² (Figure 4.1).

**Reduction factor**

There can be a reduction in live load assumed for the floor levels where change in live load is probable. The first two floor levels in the building are used for storage purposes and are not assumed as floors with large changes in variable load. In case two floor levels have to be carried, the reduction factor is 0.1. For 3 floor levels to be carried (and more), the factor is set to 0.2. The floor reduction is not taken into account when calculating the seismic weight of the building. In this case 100% of the permanent load and 25% of the unreduced variable load is considered.

**Working stress method**

The timber beams in the building are evaluated by making use of the working stress method and no partial safety factors are applied to the load. All the safety is incorporated in the timber material characteristics, where the values for the values for the permissible timber stresses (Table 3.2) are converted to grade II timber stresses (multiplying with \( \frac{5}{6} \)). In paragraph 1.5 the working stress method used for timber design in the Indian Building code is explained in more detail.

4.1.2. Results

The full weight calculation can be found in appendix D. The results of the calculation show that several beams in the roof area and the diagonal cantilever balcony beams have a Unity Check higher than 1. A building would generally not be designed with a UC higher than 1, as the accepted risk of failure would be exceeded, which will reduce the reliability of the building. However, this 200-year-old structure shows that the timber beams
strong enough to withstand loads occurred over the past centuries. The working stress method is a more conservative method than the Limit State approach, where a high safety factor is applied to calculate the permissible material characteristics. Furthermore, timber is known to have a great deviation in the timber properties, for which a high safety factor is taken into account.

The total weight of the building calculated with the floor reduction applied (as explained in paragraph 4.1.1), which is used for a working stress method calculation (so no partial safety factors are applied on the load) is 5459.7 kN.

Figure 4.2: Results of weight calculation on the different storeys (in kN/m for the loads on the walls and kN for the loads acting on the corners)
The weight calculation of the case study building shows that the vertical loads on the walls in both longitudinal and transverse direction of the building are of the same order of magnitude (Figure 4.2). The results are therefore averaged per storey and shown in table 4.1. Generally a weight calculation is performed to assess the strength capacity of the different components of the structure. Underestimating the load would be dangerous and the loads obtained from the weight calculation are therefore taken as overestimates of the actual forces. However, for the seismic design and assessment of a structure where friction and uplift are predominant lateral force resisting mechanisms it is dangerous to overestimate the vertical loads on the structure. Furthermore, Kath-Kuni structures are non-engineered and designs differ significantly between buildings, which also results in differences in load estimations between different buildings. The decision is taken to take a lower bound of the seismic weight per storey (100% dead and 25% live loads), where the values in table 4.1 are rounded down to the nearest ten. In figure 4.3 the seismic weight is shown on the storey walls of the storey below (for example 15kN/m is acting on the 3rd storey walls). These forces are used to determine the pre-compression force on the tested wall in chapter 6.

The total mass of the building considered for the working stress calculation (including floor reduction factor) is 5459.7 kN and the total seismic mass of the building is 4785.9 kN.

### 4.2. Floor calculation

In most structures the out-of-plane wall is prone to failure during an earthquake, because these walls vibrate in their local mode of vibration and when no rigid floor diagrams or wall ties are present, the wall will collapse like shown in figure 4.4a. The inertial forces in the out-of-plane walls need to be transferred to the in-plane walls or directly to the ground by the walls own capacity. Wall ties, which are present in the Kath-Kuni structure in the form of timber corner connections, ensure box-action of the building to a certain extend (Figure 4.4b and Figure 4.5). Rigid floors transform the span of the wall in vertical direction from cantilever to pinned-pinned (Figure 4.4c), which will enhance the box-action behaviour of the building. The floor rigidity is qualitatively assessed in this paragraph, which will give more insight in the necessary capacity of the out-of-plane loaded walls.
The floors in the case study building are 6 cm thick floor planks spanning in between three perpendicular timber beams (Figure 4.6). The floor planks are sometimes nailed to the timber beams, but no reliable stiff connections can be expected. These beams are spanning between the Kath-Kuni walls in the short direction of the building. One beam is located in the middle of the space, and two beams are located at the sides. The floors are categorised as flexible floors and no diaphragm action is expected.

The floor beams are spanning in the short direction of the building and are not connected to the parallel Kath-Kuni walls (see paragraph 2.2). Even with rigid floors, the Kath-Kuni (only in Himachal Pradesh province) out-of-plane loaded walls would be free to vibrate and it is crucial to assess the out-of-plane loaded wall on box-action generating capacity. In Uttarakhand the floors have a slightly different configuration, where the floor beams are supported by one beam in perpendicular direction (parallel to the floor planks), which is coupling the two walls together by making use of a shear key outside the wall (Figure 2.10).
In this chapter the Equivalent Lateral Force (ELF) method is used to perform a seismic assessment of the Old-Jubbal case study building described in chapter 2.1. The ELF method is a linear static approach to establish the equivalent seismic forces for which the building needs to be designed. Non-linear behaviour of the structure cannot directly be taken into account, however a behaviour factor can be applied to increase the capacity of the wall in case of a design approach. The capacity of structural systems to resist seismic actions in the non-linear range allows to design the structures for resistance to seismic forces smaller than those corresponding to a linear elastic response. This analysis is based on the formulas obtained from IS 1893 (Part 1) (IS 1893 (Part 1), 2002). Since IS 1893 (Part 1) is quite conservative in some respects, some adaptations from the Eurocode are considered (see chapter 1.5 and appendix A).

From the seismic capacity of the building, the maximum allowable Peak Ground Acceleration (PGA) will be determined. Because static modelling methods are chosen, and the different storey levels of the building are not all built with the same construction technique, the capacity of the building cannot be assessed for the building as a whole. However, by a lower bound approximation of the behaviour factor and time period of the building, an indication can be given of the validity of the input parameters (behaviour factor \( R \) and time period \( T \)) obtained from the Indian code. The different walls of the case study building will be assessed in out-of-plane and in-plane loaded direction. It is assumed that the rigidity of the in-plane wall is much higher than the out-of-plane wall. For the global mode of vibration, the in-plane wall stiffness will be governing. In paragraph 4.2 it is discussed how the floors in the Kath-Kuni buildings are considered as flexible floor diaphragms. Furthermore, the short walls of the building are not connected to the floors to begin with in their out-of-plane direction. Hence, the out-of-plane loaded wall will be assessed for their local natural mode of vibration. Both in-plane and out-of-plane calculations are design calculations and characteristic timber values are used (see paragraph 1.5).

In this chapter both the out-of-plane wall and in-plane wall are analytically modelled, and the maximum allowable PGA of the building is assessed.

![Figure 5.1: The different assessed Kath-Kuni wall parts of the building (only the 1st and 2nd floor level)]
5.1. Input parameters

The ELF method uses a formula derived from Newton’s second law of motion to calculate the maximum shear force (when assessing a full building called “base shear force”). The seismic mass of the building is determined in paragraph 4.1, where 100% of the permanent mass and 25% of the variable load on the structure is taken into account. This mass is multiplied with the expected PGA in the region of the building to obtain the base shear force the building needs to be designed for. Several factors are added to this formula to take the importance, the ductile behaviour and damping of the structure into account. Furthermore the natural frequency of the structure versus the dominant frequencies of a general accepted earthquake are taken into consideration in the response spectra. These factors are obtained from empirical formulas in the Indian code. Hence, the Kath-Kuni structure is a regularly built structure in horizontal direction, which is positive for the resisting capacity of the building, no torsional effects have to be taken into account. Although the code does prescribe accidental torsional forces, these have not been considered in this thesis as they are not expected to be significant for a building of this nature. Furthermore, only the horizontal effect of the earthquake acceleration is considered.

The following formula describes how the base shear of the building can be calculated according to IS 1893 (Part 1), with an expected PGA as input parameter. However, in the following calculations, the goal is to compare the maximum allowable PGA (calculated by making use of wall models to calculated the resisting capacity of the building) to the demand PGA obtained from IS 1893 (Part 1) and literature (see paragraph 1.1). According to the Limit State approach in the Eurocode, the partial safety factor on the load side $\gamma_P$ is 1.0 for seismic calculations, which ensures that the formula can be rewritten as shown below. The partial safety factor applied to the material is incorporated in the analytical wall models (resulting in a maximum shear force capacity).

$$F_s = \frac{Z I S_a W}{R g} \rightarrow PGA_{\text{max}} = Z = \frac{F_s R g W}{I S_a}$$

(5.1)

Where, according to IS 1893 (Part 1), the following parameters have to be implemented:

- $F_s$ = maximum allowable shear force (base shear force $V_b$ when full building is assessed).
- $Z$ = Zone factor, is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. Can be related to PGA by multiplying the Z-factor with 1.5 as described in paragraph 1.5.
- $I$ = importance factor, which takes into account the consequences of failure ($= 1$).
- $R$ = response reduction factor, which takes into account the ductility of the building ($= 2.5$ for stone masonry buildings reinforced by horizontal concrete bands in IS 1893 (Part 1) and hyperstatic portal frames with doweled and bolted joints according to EN 1991-1 (EN 1991-1, 2004). For the out-of-plane local wall behaviour a R is considered to be 1.
- $T$ = natural time period of the structure determined by an empirical formula and figure 5.2 can consequently be read to determine the $S_a / g$ -value ($= 0.21$ and $0.37$ for the total building in longitudinal and transverse direction respectively)
- $S_a / g$ = average response acceleration coefficient based on the period of the structure built on rock or other rock-like geological formation and takes 5% damping into account (maximum value of $S_a / g$ is 2.5 at the plateau)(Figure 5.2).
- $W$ = seismic weight of the building (see paragraph 4.1), where 100% permanent load and 25% of the variable load is taken into account.
The spectrum used to obtain the spectral acceleration coefficient is obtained from IS 1893 (Part 1) and can be found in figure 5.2. In general, the design acceleration response spectrum is the smoothened envelope of all acceleration response spectra of the ground motions for which the building should be designed.

The response reduction factor might be taken too conservatively, as the timber in the Kath-Kuni structure, will ensure a greater ductility of the building. For now, a stone masonry building with concrete bands is the most realistic equivalent in IS 1893 (Part 1). A better estimation of the response reduction factor, will be obtained by comparing the point of final failure in an experiment, to the calculated elastic point of failure.

The time period of the total building is estimated by an empirical formula from the Indian code, which is normally used for building typologies as moment resisting frame buildings with brick infill panels. Higher time periods of the Kath-Kuni building style than calculated are expected as a consequence of the flexible behaviour. Timber ductility and the timber connections give the building a certain flexibility. With elaborated analytical models, the stiffness of the in-plane wall can be assessed, and a more realistic time period can be obtained.

Last, a friction coefficient of 0.35 is assumed in this thesis, for the friction between all different materials; timber-timber, timber-stone and stone-stone (see paragraph 3.5). The friction coefficient will be implemented in the analytical wall models in chapter 5.

An overview of the input parameters chosen for the static ELF method is given in the table 5.1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T$</td>
<td>0.21 &amp; 0.37</td>
<td>s</td>
</tr>
<tr>
<td>$S_a/g$</td>
<td>2.5</td>
<td>-</td>
</tr>
<tr>
<td>$I$</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>$R$ (full building)</td>
<td>2.5</td>
<td>-</td>
</tr>
<tr>
<td>$R_l$ (local: out-of-plane)</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>$W$</td>
<td>4785.9</td>
<td>kN</td>
</tr>
<tr>
<td>$\mu$</td>
<td>0.35</td>
<td>-</td>
</tr>
</tbody>
</table>

In the next paragraphs both the out-of-plane loaded wall and in-plane loaded wall will be assessed for the maximum PGA that they can individually resist and compared to the following Limit States, which are generally applied for ordinary new buildings under earthquake loading:
• Significant damage during an earthquake with 10% probability of exceedance in 50 years (Return Period (RP) of 475-years)
• Near collapse during an earthquake with 2% probability of exceedance in 50 years (RP of 2475-years)

The characteristic timber stress values which are used for this assessment are the timber working stress characteristics obtained and converted from the Indian code. The obtained and used characteristic stress values can be found in table 3.2. All the structural configuration details in the calculations are derived from the case study building in the Indian Himalaya (see Figure 5.4). For the densities of the Kath-Kuni wall materials (timber, stone and rubble), see paragraph 3.4.

Parameters
\[ l = 5400 \text{ mm is the length of the wall (maximum span in example building)} \]
\[ w = 460 \text{ mm is the width of the wall} \]
\[ w_b = 100 \text{ mm is the width of one single beam} \]
\[ h_b = 175 \text{ mm is the height of one single beam} \]
\[ h_i = 113 \text{ mm is the height of the infill piece} \]
\[ E_0 = E_{0,\text{mean}} \cdot 1.1 = 13035 \text{ N/mm}^2 \text{ is the E-modulus used for instantaneous loading (10% higher than the E-modulus used for short term loading according to EN 1998-1)} \]

Figure 5.3: Cross-section of wall layer at span

5.2. Out-of-plane loaded wall model

The out-of-plane analysis is performed by the generic assumption that the in-plane wall is much stiffer than the out-of-plane wall. In this calculation the wall with the largest span (number 2 in figure 5.1) is checked for the maximum PGA acceleration capacity and compared to the values obtained from IS 1893 (Part 1) (paragraph 1.1).

The out-of-plane wall should be analysed for two different modes of vibration, both with the horizontal displacement as single degree of freedom:
• Local wall mode of vibration (assessed per layer): layers are considered to vibrate in horizontal direction and the beams spanning in between the in-plane walls. The timber beams generate the stiffness of the wall in horizontal direction, much more than the dry-stone masonry can generate in both the horizontal and vertical direction. Hence only the timber beams are considered to compute the stiffness for the vibration mode. The timber beams including half of the stone/concrete masonry layer below and above the timber beam are taken into account for calculating the seismic mass. The natural frequency of the layers in the out-of-plane loaded wall is used for computing the maximum spectrum value and this value is used to compute the resulting maximum seismic force demand. It is investigated if the capacity of the out-of-plane loaded wall layers is sufficient to resist the seismic forces.
• Global wall mode of vibration: the out-of-plane loaded wall will displace as a consequence of the in-plane loaded wall movement, hence torsional forces are induced in the out-of-plane loaded wall. These forces can be divided into firstly, internal forces due to the deformation for the wall, secondly, the torque generated by the eccentricity of the vertical force and lastly, the torque caused by the lateral seismic force on the wall. With this calculation the maximum possible deformation and the contribution of the out-of-plane wall to the total global stiffness of the in-plane wall can be assessed as a combination of the elastic stiffness and the geometric stiffness. The geometric stiffness is load dependent and generates lateral...
instability. The stiffness of the out-of-plane wall is so complex that it is left out of this thesis and is recommended for future research.

The intermediate connection at midspan in the Kath-Kuni walls of the case study building (Figure 5.4) are neglected in the analyses, hence these connections are not prevalent in all Kath-Kuni buildings.

![Image of Kath-Kuni walls](image)

(a) Maximum considered length out-of-plane loaded wall  
(b) Intermediate coupling Kath-Kuni walls

Figure 5.4: Out-of-plane loaded wall assumptions for case study building

### 5.2.1. Local wall mode of vibration

The out-of-plane loaded wall vibration is expected to happen according to the mode shape depicted in Figure 5.5a. The corners of the wall create wall ties, which connect the in-plane loaded wall and the out-of-plane loaded wall together. The corner connection is expected to have a certain rotational stiffness. Because, the floors are not connected to some of the walls in the building, these walls are prone to out-of-plane vibrations over the total height of the building, without floors restricting the movement. This results in the expected free end on top of the wall in figure 5.5a. However, the exact boundary conditions are unknown and the calculation will be performed with different assumed boundary conditions for the wall in both horizontal and vertical direction (Figure 5.5b)

![Image of out-of-plane loaded wall behaviour](image)

(a) Out-of-plane loaded wall behaviour  
(b) Different possible shape functions depending on the assumed boundary conditions

Figure 5.5: Out-of-plane loaded wall behaviour
The out-of-plane wall consists of multiple stacked layers of alternating timber and dry-stone masonry. It can be assumed that the horizontal timber beams generate much more stiffness to the wall than the dry-stone masonry in flexural direction. The wall will therefore predominantly transfer the lateral load (resulting in bending moments) in the horizontal direction and the wall can be divided in horizontal layers to assess the maximum lateral wall capacity. The axial forces in the wall are however carried in the vertical direction. The wall deformation can be described by shape functions in both the x-direction (horizontal) and y-direction (vertical). The possible shape functions are based on the assumed different boundary conditions (see Figure 5.5b). The vertical and horizontal shape function have a unit displacement at the same layer, here the shape function in horizontal direction can be described as \( \psi(x) \). In every other layer the value of this shape function is reduced by the shape function in y-direction to \( \chi_i \psi(x) \), where \( \chi_i \) is a constant and \( i \) is the evaluated layer.

The natural frequency is described by the following formula:

\[
\omega_n = \sqrt{\frac{K}{m}}
\]  

(5.2)

Because \( \chi_i \), \( EI \) (flexural stiffness in horizontal direction) and \( \bar{m} \) (mass per unit length) are constants, the seismic activated (consistent) mass and the stiffness of the full wall can be given as follows: (see appendix E for the derivation of these formulas)

\[
K_i = EI \chi_i^2 \int \psi''(x)^2 \, dx \\
m_i = \bar{m} \chi_i^2 \int \psi(x)^2 \, dx
\]

(5.3)

\[
K = \sum_{i=1}^{n} EI \chi_i^2 \int \psi''(x)^2 \, dx \\
m = \sum_{i=1}^{n} \bar{m} \chi_i^2 \int \psi(x)^2 \, dx
\]

(5.4)

The natural frequency for each layer and the total wall \( K = \sum_{i=1}^{n} K_i \, dx \) and \( m = \sum_{i=1}^{n} m_i \, dx \) will be equal, as both the stiffness and the mass are scaled by the same squared constant.

The resulting maximum seismic force (using the following concept: \( F_i = m_i \, a_g \)) is depending on the changing amount of activated mass in each layer. As \( \chi_i = 1 \) at the location of the unity displacement and \( \chi_i < 1 \) for the other layers, the maximum seismic force can never exceed the force calculated in the maximum displaced layer. Because it is in this layer that the maximum mass is activated. Since the time periods of all layers are the same, the maximum mass activated would correspond to maximum force. The following calculation is performed with a unity displacement of shape function in x-direction and the maximum force will thus be independent of the shape function chosen in y-direction.

In order to calculate the maximum seismic force activated in the wall a lower and upper bound analysis will be performed. A high stiffness and low activated mass results in a high natural frequency and low time period and
a low stiffness and high activated mass results in a lower frequency and high time period. If the natural frequency of the out-of-plane wall is similar to the dominant frequencies which can be found in the assumed earthquake spectrum (obtained from IS 1893 (Part 1), see figure 5.2), a higher seismic force needs to be taken into account. However, a high activated seismic mass will also lead to a high seismic force on the structure (see the shear force formula in paragraph 5.1). The mass is thus not only of influence in the determination of the stiffness of the component of the building, but also has an effect on the amount of inertial forces needed to activate the seismic mass. A very flexible structure with a high mass, can still result in significant seismic forces on the structure.

The out-of-plane wall can be modelled with rotation springs on the point of the maanwi and kadil connections. Both pinned-pinned and fully clamped corner connections will be modelled (Figure 5.8). Furthermore, the two beams can be assumed to work together as one combined beam with some distance in between (full cooperation, \( \gamma = 1 \)), or as two individual acting beams (no cooperation, \( \gamma = 0 \)).

The stiffness of the wall in the lateral direction can be described by the stiffness of the timber beams. The mass can be described by the mass of the timber beams, the mass of the rubble and the mass of half a stone/concrete layer above and below the timber beam (Figure 5.3).

This leads to a mass of per unit length of the out-of-plane wall layer considered of:

\[
\bar{m} = 560 \times 0.200 \times 0.175 + 2040 \times 0.260 \times 0.175 + 2590 \times 0.460 \times 0.288 = 455.6 \text{ kg/m}
\]

The stiffness of the out-of-plane wall layer (is the stiffness of the total wall) in horizontal direction can be calculated by assuming the simplified models. The stiffness for the beam simplification is calculated using a 3rd order shape function (see appendix E for the derivation):

\[
K_{\text{pin}} = \frac{48E_Iy}{l^3} \quad K_{\text{cl}} = \frac{192E_Iy}{l^3} \quad (5.5)
\]

| Table 5.2: Stiffness for different model configurations for out-of-plane loaded wall |
|-------------------------------|----------------|----------------|
| STIFFNESS [N/m]              | Double clamped | Pinned-pinned  |
| Full cooperation \( \gamma = 1 \) | 18487276        | 4621819        |
| No cooperation \( \gamma = 0 \) | 463573          | 115893         |
Where \( I_{yy} = \sum I_i + \gamma z^2 A_i \) is the moment of inertia of the total wall cross section, where \( I_i \) is the moment of inertia of an individual beam, \( \gamma \) is the cooperation factor between the beams, \( z = 180 \text{ mm} \) is the distance of the centre of gravity of the individual beam to the centre of gravity of the total wall cross section and \( A_i \) is the cross-sectional area of a single beam. This results in \( I_{yy,1} = 2 \cdot \frac{1}{12} (h_b w_b^3 + 1 \cdot z^2 h_b w_b) = 11.6 \cdot 10^8 \text{ mm}^4 \) is the moment of inertia of the composite beam (full cooperation, \( \gamma = 1 \)) and \( I_{yy,0} = 2 \cdot \frac{1}{12} (h_b w_b^3 + 2 \cdot z^2 h_b w_b) = 29.2 \cdot 10^6 \text{ mm}^4 \) is the moment of inertia of independent beams (no cooperation, \( \gamma = 0 \)).

The seismic mass can be calculated in different manners. One is the lumped mass calculation, where a first order shape function is assumed. This mass calculation method is independent of the shape function of the stiffness of the wall beams. This leads to the following inertial mass:

\[
m_{\text{lumped}} = \frac{\bar{m} l}{2} = 1230.1 \text{ kg}
\]

The second manner is the consistent mass calculation, where a 3rd order (cubic) shape function is assumed, this shape function is the same as the shape function used to calculate the stiffness of the wall (see appendix E for the derivation). This leads to the following mass:

\[
m_{\text{pin,consistent}} = 0.486 \bar{m} l = 1195.7 \text{ kg} \quad m_{\text{cl,consistent}} = \frac{13 \bar{m} l}{35} = 913.8 \text{ kg}
\]

The time period can be calculated using the following formula:

\[
T = \frac{2\pi}{\sqrt{K/m}}
\]

Table 5.3: Time periods for out-of-plane loaded wall (local mode of vibration)

<table>
<thead>
<tr>
<th>TIME PERIOD [s]</th>
<th>Double clamped</th>
<th>Pinned-pinned</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \gamma = 1 )</td>
<td>( \gamma = 0 )</td>
</tr>
<tr>
<td>Consistent mass</td>
<td>0.04</td>
<td>0.28</td>
</tr>
<tr>
<td>Lumped mass</td>
<td>0.05</td>
<td>0.32</td>
</tr>
</tbody>
</table>

Looking at the response spectra from IS 1893 (Part 1) in figure 5.2, shows that a time period larger than 0.1 s and smaller than 0.4 s is giving a spectral acceleration coefficient in the plateau of the seismic spectrum, which has a value of 2.5. This value will be used to calculate the seismic force on the out-of-plane wall with the formula given in paragraph 5.1. A double clamped simplification model is also assessed and gave a time period smaller than the spectral plateau value.

Table 5.4: Spectral acceleration coefficients for out-of-plane loaded wall

<table>
<thead>
<tr>
<th>SPECTRAL ACCELERATION COEFFICIENT ( S_a/g ) [-]</th>
<th>Double clamped</th>
<th>Pinned-pinned</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \gamma = 1 )</td>
<td>( \gamma = 0 )</td>
</tr>
<tr>
<td>Consistent mass</td>
<td>1.04</td>
<td>2.5</td>
</tr>
<tr>
<td>Lumped mass</td>
<td>1.05</td>
<td>2.5</td>
</tr>
</tbody>
</table>

The maximum allowable design timer strength is calculated by formula (5.9).
\[ f_{m,d} = k_{mod} \frac{f_{m,k}}{\gamma_m} = 1.1 \cdot \frac{f_{m,k}}{1.3} = 23.4 \text{ N/mm}^2 \]  

(5.9)

Where, \( f_{m,k} \) is the characteristic bending strength on Deodar timber (Table 3.2), a factor taking into account the duration of the loading \( k_{mod} \) is equal to 1.1 and the partial safety factor on the material \( \gamma_m \) is equal to 1.3.

The formula to calculate the seismic force on the structure can be found in paragraph 5.1. The force acting on the out-of-plane wall takes 5% damping into account. A behaviour factor \( R_l = 1 \) is taken into account, whereas the ductility of the wall in out-of-plane direction is expected to be brittle and no research is available to prove otherwise.

Calculating with the higher lumped mass, will give more conservative results of the maximum allowable design PGA. The influence of the mass calculation method to the spectral acceleration coefficient is negligible and the calculations will be continued with the lumped mass of 1230.1 kg.

This results in maximum allowable design PGA accelerations for the out-of-plane loaded wall given in table 5.5.

<table>
<thead>
<tr>
<th>Table 5.5: Maximum allowable PGA for the out-of-plane loaded wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAX. PGA [g]</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Clamped</td>
</tr>
<tr>
<td>Pinned-pinned</td>
</tr>
</tbody>
</table>

The out-of-plane loaded wall is most critical for the model without beam cooperation and a pinned-pinned corner connection. It can be seen from the large range of maximum PGA acceleration between the different simplification assumptions, that the amount of cooperation between the beams is very important for the capacity of the beams, especially in the case of fully clamped connections. In reality the actual amount of beam cooperation and corner connection stiffness will be somewhere in between the two bounds, which means that a maximum allowable PGA of 0.53 g is the absolute lower bound.

5.2.2. Global wall mode of vibration

Under the assumption that the in-plane wall is significantly more rigid than the out-of-plane walls, it can be stated that the global deformation and the natural frequency is defined by the deformation of the in-plane wall and has negligible contribution from the out-of-plane wall. It is furthermore assumed that the walls are perfectly connected at the locations of the timber beams, which ensures that the out-of-plane wall will deflect
according to the same shape function and magnitude as the in-plane wall. Due to this deformation and additional secondary effects, torsion will develop in the out-of-plane wall. Torsion in the out-of-plane wall is induced by the following forces:

Immediate consequence of deformation:
- Torque due to the in-plane wall deformation (resulting in a curvature in the out-of-plane wall)

Secondary consequence of deformation:
- Torque due to horizontal seismic force
- Torque due to eccentricity in the vertical load (instability)

Possible torsion resisting mechanisms in the wall are:
- Moment resisting wall action
- *Maanwi* tensile capacity
- Torsion capacity of the beams
  - Embedment of the beams in the wall
  - Torsion capacity of the free beam

It is recommended to research the effect of the global in-plane movement and the additional vertical force effect on the out-of-plane wall stability. In this thesis, this is not taken into account.

5.2.3. *Conclusion: out-of-plane loaded wall model*

The maximum PGA an out-of-plane loaded wall is able to resist is determined by making use of a modelling range. This as a consequence of the uncertainty in the connectivity and stiffness of the corner connections and the *maanwi’s* in between the beams. In a lower bound model, the layers in the wall are assessed as pinned-pinned corners with no cooperation between the beams, whereas the upper mound model takes completely clamped corners and full cooperation into account. The out-of-plane loaded wall results are shown in figure 5.10.

The range in the out-of-plane loaded wall capacity is very large, where the lower bound is a maximum allowable PGA of 0.53 g and the upper bound 13.53 g. These values are higher than the design demand PGA value obtained from IS 1893 (Part 1) for a Design Basis Earthquake (DBE). They are similar to the design demand PGA value of a Maximum Considered Earthquake (MCE) and the deterministic determined PGA values from literature. The values are also exceeding the PGA obtained from literature with a RP of 475 years (probability of exceedance of 10% in 50 years). In case the actual building would be conform the lower bound assumptions of fully flexible corners and no cooperation between the timber beams, the wall would not have sufficient capacity in its linear region to withstand an earthquake with an RP of 2475 years (near collapse limit state). However, due to the very conservative approach in this calculation, where no non-linear behaviour and no rigidity in the connections is taken into account, it can be concluded that an out-of-plane loading is not critical for the seismic capacity of the total building. Despite the flexible floors and the lack of floor to wall connection in the building, walls are able to transfer the forces in out-of-plane direction sufficiently to the in-plane loaded walls. The horizontal timber beams in the out-of-plane loaded walls are providing the necessary box-action.

Several more uncertainties in the calculation of the out-of-plane loaded wall are listed below:
- Stiffness of the *kadil* and *maanwi* connections and the amount of cooperation between the timber beams.
- The material input characteristics, are now obtained from the Indian code. Experimental tests should be performed to obtain more certainty.
- The input behaviour factor $R$: is now not taken into account in the calculation but can significantly improve the calculated capacity of the wall.
- The amount of damping of the structure; now taken as the default value from IS 1893 (Part 1) of 5%. For timber structures this can be significantly higher.
- The effect of the global building vibration mode on the out-of-plane loaded wall (see paragraph 5.2.2).
- The effect of the intermediate connection in the out-of-plane wall, like shown in figure 5.4, which is neglected and could enhance the capacity of the wall.
- Tests or non-linear dynamic modelling methods could give a better insight in the behaviour of the wall outside the elastic regime and with dynamic effects taken into account.
- The out-of-plane behaviour of the Kath-Kuni gables, under the roof. They have to be checked separately.
- The possible influence of wall opening is not considered.

![PGA acceleration Demand vs. Capacity](image)

**Figure 5.10: Out-of-plane wall PGA demand versus capacity results**
5.3. In-plane loaded wall model

The in-plane walls contribute the most to the stiffness of the total building and therefore the walls are assessed for their capacity in in-plane direction and consequently extrapolated to the capacity of the total building. The building configuration of the Kath-Kuni building is not consistent over the height of the building and in this thesis only the Kath-Kuni part of the building is researched (Figure 5.1). The ground floor of an average Kath-Kuni building consists of dry stone. The Kath-Kuni wall part is located over a height of approximately 3.2 meter in the building from the first storey upwards. Higher in the building, the walls are consisting more predominantly of timber and the typical Kath-Kuni corner joints disappear. The force capacity in the different walls (walls 1-4) specified in figure 5.1 is evaluated. This force capacity will be converted to the maximum required base shear force capacity of the full building by making use of the ELF method. Finally, the PGA capacity of the full building is calculated using the maximum base shear force capacity of the building and the seismic weight of the building. Because the calculation is performed by making use of the Limit State approach, a partial safety factor \( \gamma_p = 1 \) is assumed on the load side. The partial safety factor on the material side \( \gamma_M \) is also 1, due to the high ductility class the Kath-Kuni in-plane loaded walls are categorised in. Last, a duration factor \( (k_{mod} = 1.1) \) is applied to the timber capacity.

\[ f_d = k_{mod} \frac{f_k}{\gamma_M} = 1.1 \cdot f_k \]  

(5.10)

The conversion from characteristic material values to design values is only performed for the timber properties. The other rubble is neglected in the model and the stone is only contribution by friction. The friction capacity is determined by an estimated friction coefficient and the vertical weight established by mean material densities. In paragraph 4.1.2 the vertical seismic weight is established and the values are for convenience purposes and safety rounded down to the closest ten, which is approximately 90% of the calculated values. Applying a load duration factor on the friction capacity would not make sense as it would increase the capacity.

First, a global failure assessment is performed for Kath-Kuni (only wall number 1), for sliding in the bottom layer, rocking and toe crushing. The considered wall is modelled with 7 timber layers (and 6 stone masonry infill layers) and is 2953 mm high (timber height per layer is approximately 175 mm, stone height per layer is approximately 288 mm, 7 timber layers and 6 concrete layers are considered in the model). The wall has a length of 4800 mm and a thickness of 460 mm. The calculated density of the wall is 18.94 kN/m\(^3\), where material densities from paragraph 3.4 are assumed\(^2\). The global failure analysis is performed in paragraph 5.3.1, where the total wall is considered as one solid element. However, in reality the wall cannot be computed as a solid element and the local layer failure behaviour will be assessed in paragraph 5.3.2.

The calculation is performed for characteristic timber values – to perform a strength validation and obtain the maximum allowable PGA.

\(^2\) A stone density of 2500 kg/m\(^3\) is used instead of the 2590 kg/m\(^3\) specified in paragraph 3.4.
5.3.1. Global in-plane loaded wall

The following maximum global lateral force capacity values are determined for in-plane loaded wall number 1 (Figure 5.1). The pre-compression on top of the wall is 30kN/m (obtained from figure 4.3) and the total wall weight is 123.5 kN. This results in a pre-compression force at the bottom layer of 267.5 kN.

Bottom sliding

Rocking:

Toe crushing (timber failure):

\[ V_{sl} = 93.6 \text{ kN} \]
\[ V_r = 217.4 \text{ kN} \]
\[ V_t = 201.0 \text{ kN} \]
\[ x = 528.6 \text{ mm} \]

Figure 5.11: Global failure mechanisms

1. Sliding failure:
   Sliding can only occur when the frictional resistance is exceeded. The lower the normal force is, the lower the friction resistance will be. This means that the frictional failure would be more likely to happen at the top of the wall, however this would only be valid if timber frame would not be present. The timber frame ensures an increased capacity to the wall layer, which is assessed in paragraph 5.3.2. At the bottom layer, in between the stone and the first timber layer, there are no kadils present. This layer has friction as only shear force resisting capacity, which results in global sliding failure in case the friction capacity is exceeded. The initial friction capacity in the different layers in the wall are given in table 5.6

<table>
<thead>
<tr>
<th>Layer</th>
<th>Pre-compression force W</th>
<th>Lateral force V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>151.3 kN</td>
<td>53.0 kN</td>
</tr>
<tr>
<td>Layer 2</td>
<td>170.7 kN</td>
<td>59.7 kN</td>
</tr>
<tr>
<td>Layer 3</td>
<td>190.0 kN</td>
<td>66.5 kN</td>
</tr>
<tr>
<td>Layer 4</td>
<td>209.4 kN</td>
<td>73.3 kN</td>
</tr>
<tr>
<td>Layer 5</td>
<td>228.8 kN</td>
<td>80.1 kN</td>
</tr>
<tr>
<td>Layer 6</td>
<td>248.1 kN</td>
<td>86.8 kN</td>
</tr>
<tr>
<td>Layer 7</td>
<td>267.5 kN</td>
<td>93.6 kN</td>
</tr>
</tbody>
</table>

2. Rocking mechanism:
   The wall will rotate around the right bottom corner. This capacity is material strength independent, but directly depended on the amount of overburden applied on the wall.

3. Toe crushing:
   Toe crushing occurs, if the compression force in the right bottom corner is exceeding the compression strength of the stone or the timber. The greater the rotation of the wall is, the smaller the compression area of the wall will be. Because vertical equilibrium has to be maintained, the stresses will increase significantly. The timber
compression strength perpendicular to the grain \((f_{t,90,4} = 2.0 \times 1.1 = 2.2 \text{ N/mm}^2\), taking into account a \(k_{mod}\) of 1.1) is much less than the compression strength of stone. The bottom layer is a combined stone and timber layer. The layer above that is a complete timber beam of 127 thickness, which will compress in case the compression strength of the timber perpendicular to the grain is exceeded.

### 5.3.2. Local in-plane loaded wall

The internal layer capacity of the wall (as a consequence of the connected timber elements) is not taken into account in the global in-plane wall calculation. However, it is expected that the layers will internally deform significantly during the lateral loading and will furthermore be governing as final failure mode. To assess the internal layer capacity of the wall, the wall is modelled with the boundary conditions as depicted in figure 5.12 (the model is conceptual, as the actual model contains 7 layers), where part of the perpendicular wall beams are taken into account as well. This to ensure that the influence of the *maanwi’s* located in between the perpendicular wall beams are incorporated.

![Conceptual wall model with applied boundary conditions to assess in-plane capacity](image)

**Figure 5.12: Conceptual wall model with applied boundary conditions to assess in-plane capacity**

Figure 5.13 shows a sketch of a single layer in the wall (the bottom timber beam in the figure belongs to the next layer). The layers can fail according to different failure mechanisms. The model is based on a virtual work approach, where the different failure mechanisms are assessed for their virtual work contribution. The lowest mechanism is assumed to be the final failure mechanism of the wall. The layers are individually assessed for the different height locations in the wall. All the failure mechanisms assessed can be found in appendix G, here a detailed analysis of these failure mechanisms can be found.

![Numbering of the location of the perpendicular beams from right to left \(n = 1..4\) (in red) and amount of *kadil* connection in the wall (in black)](image)

**Figure 5.13: Numbering of the location of the perpendicular beams from right to left \(n = 1..4\) (in red) and amount of *kadil* connection in the wall (in black)**
The failure mechanisms assessed are different for the left side and the right side of the wall. An overview is given for the possible failure modes considered in this research for the wall on the left hand side (Figure 5.14a-b) and right hand side (Figure 5.14c-d). Three of the most critical combinations of the following failure mechanisms are evaluated (see appendix G). The failure mechanism of Figure 5.14d is unlikely due to the large amount of energy that is required to rotate the longitudinal infill piece and is therefore not incorporated in the mechanisms in appendix G.

![Possible failure mechanisms considered on left and right side of the wall](image)

Figure 5.14: Possible failure mechanisms considered on left and right side of the wall

The force capacity of a single layer is determined by the following components:

- Friction
- Pre-compression perpendicular beam and infill piece rotation
- Kadil connection (timber dowel connection) – ductile
- Maanwi connection (timber dovetail connection) – brittle

![Assumed force-displacement graphs of connections](image)

Figure 5.15: Assumed force-displacement graphs of connections

**Assumptions:**

- The in-plane loaded wall is assessed by assessing an individual layer and no global wall resisting behaviour, for example the global rotation of the wall as a consequence of the moment action of the wall, is taken into consideration. Global resistance capacity will contribute to the total amount of virtual work and thus increase the capacity. Because no kinematics between the layers is considered, the actual displaced wall shape (different deflections over the height of the wall) is not taken into account.
- Only plastic contributions to the virtual work is taken into account, except the contribution of the maanwi connection. In order to assess the stiffness of the structure, the stiffness’s of the individual connections need to be taken into account in the model.
- The virtual work is only valid for small displacements and the calculation in not incremental, which means that no development of the force over the increasing displacement can be shown and no initial rotations
are incorporated. For an incremental analysis, every assessed displacement step should be calculated, using the output of the previous step.

- All the virtual work equations are valid for an $\delta u$ smaller than the elastic deformation of the maanwi (see formula (5.11)). After the elastic capacity of the maanwi is exceeded, the maanwi connection will fail and the non-elastic phase will start. This is not taken into account in this calculation.

$$\delta u < \Delta_m = \frac{F_{m.t.d.\,real} \cdot l_{m-2l_{m\,taper}}}{t_m w_m \theta_0} = 0.0366 \text{ mm}$$ (5.11)

Where,

$\delta u =$ the virtual displacement of the timber beam in the layer relative to the timber beam of the layer below,

$\Delta_m =$ elongation of the maanwi body,

$F_{m.t.d.\,real} =$ the maximum elastic design capacity of the maanwi in tension (see paragraph 5.3.4)

$l_m =$ 380 mm is the length of total maanwi,

$l_{m\,taper} =$ 76 mm is the length of the maanwi taper,

$t_m =$ 45 mm is the thickness of the maanwi,

$w_m =$ 120 mm is the width of the maanwi body.

The timber compression failure behaviour is simplified in this analysis with a bilinear stress-strain curve, where the maximum elastic stress capacity is equal to the ultimate/plastic stress.

- **Maanwi** is only contributing in tension and when the connected perpendicular beams are subjected to different rotations. The rotational capacity of the maanwi is not taken into account.
- No tension capacity of the kadil connection is assumed.
- The kinetic friction coefficient is not taken into account.
- The equations do not take all virtual work contributions into account, so it does not give a lower bound assumption of the force capacity of the layer.
- Assumption of small angle rotations:

$$\tan \theta \approx \theta, \quad \delta \theta_1 = \frac{\delta u}{h_z}, \quad \delta \theta_2 = \frac{\delta u}{h_b}, \quad \delta \theta_3 = \frac{\delta u}{h_i}$$ (5.12)

$h_z =$ height of stone layer  \hspace{1cm} $\theta_1 =$ rotation over height of stone layer

$h_b =$ height of timber beam \hspace{1cm} $\theta_2 =$ rotation over height of timber beam

$h_i =$ height of infill piece \hspace{1cm} $\theta_3 =$ rotation over height of infill piece

---

Figure 5.16: Connections

---

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5.3.3. Kadil connection capacity

The *kadil* connections are timber dowel connections, where timber square dowels of 45x45x90 mm are inserted in the beams and infill pieces to create a connectivity in vertical direction (Figure 5.16b). The *kadils* transfer the shear force (for which their capacity is sufficient) and generate moment resistant capacity against the failure rotation in a layer. Because the *kadil* is embedded in the beams and the failure is based on the compression strength of timber, the ductility of the connection is assumed to be high. Even though the *kadil* “fails” in earlier stages of the wall behaviour, the *kadil* will be assumed to remain its capacity and is expected to generate a certain post failure capacity.

The pure shear capacity of the *kadil* can be approximately calculated with formula (5.13) from the German Historische Holztraerke (Parmessar, 2004).

\[
F_{k,d} = 4w_k^2 f(\rho) = 6.6 \text{ kN}
\]

where \(f(\rho)\) gives the relation between the characteristic density of Deodar 464 kg/m³ compared to that of Oak 570 kg/m³.

The *kadil* can be embedded in the following manners:

**Option 1:** Embedment in the longitudinal timber beam and the perpendicular timber beam/infill piece  
**Option 2:** Embedment in between the perpendicular beam and perpendicular infill piece  
**Option 3:** Embedment in between the longitudinal beam and the longitudinal infill piece

A model incorporating dowel failure was proposed by Johansen EN 1995-1-1 (EN 1995-1-1, 2011). This model indicates the different modes of failure and estimates the ultimate shear force corresponding to these modes. One important aspect of the Johansen’s model is that it postulates that the two main members operate in pure sliding conditions and are not subject to any relative rotation with respect to each other. Any rotations happening in any of the failure modes are restricted to the dowel only. For the Kath-Kuni wall under consideration, this assumption is not valid as it is expected that the two main members would definitely undergo a rotation relative to each other (see the possible failure mechanisms in figure 5.14). Hence the Johansen’s model is strictly not applicable to this joint. However, since a corresponding model which incorporates the relative rotation of the members is not available, the estimation of the joint is carried out with the Johansen’s model expecting that the results from this model give an initial estimation and that the relative member rotations would not introduce significant estimation errors in this model. However, the knowledge that the members do rotate relative to each other is incorporated in the model by ruling out some of the failure modes which fundamentally do not allow any rotation possibilities.
The modes of failure are listed in figure 5.17 from a to f. Each of these figures illustrates one schematic mode of failure. The modes of failure a and b are predicated on the embedment failure in only one of the members, resulting in a sliding failure and are not based on possible rotations in the connection. Furthermore, failure modes a and b are influence by the fact that the kadil is not piercing the members fully, but embedment is also present on both the ends of the kadil, which is expected to enhance the capacity significantly. Hence, modes a and b are assumed to be inappropriate for the Kath-Kuni wall. Modes c to f indicate failures in both the members which is consistent with the expectation of relative member rotation and hence these are considered for further analysis.

According to the formulas of Johansen, the minimum of the following capacities is giving the capacity of the connection:

\[(a) \quad F_{k,1,d} = w_k \frac{1}{2} h_k f_{c,90,d} \]  \hspace{1cm} (5.14)

\[(b) \quad F_{k,2,d} = w_k \frac{1}{2} h_k f_{c,90,d} \]  \hspace{1cm} (5.15)

\[(c) \quad F_{k,3,d} = \frac{f_{c,90,d} \frac{1}{2} h_k w_k}{1 + \beta} \left( \sqrt{\beta + 6\beta^2 + \beta^3} - 2\beta \right) \]  \hspace{1cm} (5.16)

\[(d) \quad F_{k,4,d} = \frac{f_{c,90,d} \frac{1}{2} h_k w_k}{2 + \beta} \left( \frac{2\beta^2(1 + \beta) + 4\beta(2 + \beta)M_{kl,d}}{f_{c,90,d} w_k (\frac{1}{2} h_k)^2} - \beta \right) \]  \hspace{1cm} (5.17)

\[(e) \quad F_{k,5,d} = \frac{f_{c,90,d} \frac{1}{2} h_k w_k}{1 + 2\beta} \left( \frac{2\beta^2(1 + \beta) + 4\beta(1 + 2\beta)M_{kl,d}}{f_{c,90,d} w_k (\frac{1}{2} h_k)^2} - \beta \right) \]  \hspace{1cm} (5.18)

\[(f) \quad F_{k,6,d} = \frac{2\beta}{1 + \beta} \sqrt{2M_{k,d} f_{c,90,d} w_k} \]  \hspace{1cm} (5.19)

Where, \(\beta = \frac{f_{c,90,d}}{f_{c,90,d}}\)

\(w_k = 45\) mm; width of the kadil (in two directions)
\(h_k = 90\) mm; height of the kadil
\(f_{c,90,d} = \) design compression stress perpendicular to the grain
\( f_{c,0,d} \) = design compression stress longitudinal to the grain

\( M_{k,i,d} \) = design bending moment capacity of the kadil dowel itself (not the full connection)

### Table 5.7: Kadil characteristic strength values

<table>
<thead>
<tr>
<th>Mechanism</th>
<th>Option 1</th>
<th>Option 2</th>
<th>Option 3</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max. capacity</td>
<td>Max. capacity</td>
<td>Max. capacity</td>
<td></td>
</tr>
<tr>
<td>-</td>
<td>( F_{k,d} )</td>
<td>6.6</td>
<td>6.6</td>
<td>6.6</td>
</tr>
<tr>
<td>a</td>
<td>( F_{k,1,d} )</td>
<td>4.5</td>
<td>4.5</td>
<td>39.9</td>
</tr>
<tr>
<td>b</td>
<td>( F_{k,2,d} )</td>
<td>39.9</td>
<td>4.5</td>
<td>39.9</td>
</tr>
<tr>
<td>c</td>
<td>( F_{k,3,d} )</td>
<td>7.5</td>
<td>1.8</td>
<td>16.5</td>
</tr>
<tr>
<td>d</td>
<td>( F_{k,4,d} )</td>
<td>13.3</td>
<td>8.2</td>
<td>32.8</td>
</tr>
<tr>
<td>e</td>
<td>( F_{k,5,d} )</td>
<td>9.8</td>
<td>8.2</td>
<td>32.8</td>
</tr>
<tr>
<td>f</td>
<td>( F_{k,6,d} )</td>
<td>12.8</td>
<td>9.6</td>
<td>28.6</td>
</tr>
</tbody>
</table>

The response of the Kath-Kuni wall is being addressed in this thesis using a virtual work formulation. In this formulation, the rotation of the main members are the main variables and a minimum virtual work formulation correlates these to the applied horizontal forces. Since the variables in the formulation are the member rotations, this needs the forces computed above to be represented in a moment-rotation form. Through equilibrium the forces are balanced as a couple on the dowel as indicated in figure 5.18. From this the moment can be computed related to each rotational configuration and incorporated in the virtual work equation.

Assumed moment contributions:

Option 1: \( M_{k,1,d} = 14.6 \cdot 10^3 \text{Nmm} \)

Option 2: \( M_{k,2,d} = 2.6 \cdot 10^3 \text{Nmm} \)

Option 3: \( M_{k,3,d} = 23.4 \cdot 10^3 \text{Nmm} \)

Besides the internal rotation, a check is performed to evaluate if the shear force can be transferred through the kadil connections before the onset of internal rotation. Either three or four kadil connections per plane per corner are present. From the total of 6 kadils per shear plane (two corners), two kadils are present corresponding to option 1 and four kadils corresponding to option 2. The weakest shear plane in the wall is the plane in between the infill pieces and the perpendicular beams of the first (top) layer, hence the lowest precompression force is applied here. The total shear capacity of the weakest plane is an addition of the lowest
kadi capacity (two times option 1 = 6.6 kN and four times option 2 = 1.8 kN), where failure modes a and b are not considered to be representative for the Kath-Kuni wall, plus the friction capacity of the first layer (53 kN, see table 5.6). This results in a total initial pure shear capacity of 80 kN.

5.3.4. Maanwi connection capacity

In order to take the contribution of the maanwi’s in the perpendicular walls into account the maanwi connection is modelled as shown in figure 5.19. The maanwi’s are embedded in the perpendicular beam on both sides at a distance of approximately 125 mm. The tensile strength of the maanwi is governed by the embedment capacity. The final maanwi connection capacity is brittle, therefore only the elastic contribution to the work of the maanwi is taken into account. The capacity of the maanwi connection is calculated assuming moment equilibrium around point S (and multiplied by 2 to model the capacity on both sides of the tapered part of the maanwi). The maximum force is divided by 2 to take the elastic work contribution into account.

\[ F_m.t.d,\text{real} = f_{t,90,d}d_{p1}t_m + f_{v,1.6}\frac{1}{2}(d_{p1} + d_{p2})t_m = 11.3 \text{ kN} \]  \hspace{1cm} (5.20)

\[ F_m.t.d = 0.5 F_u.m.d,\text{real} \frac{2d_m w}{a_m + w} = 6.9 \text{ kN} \]  \hspace{1cm} (5.21)

\[ M_{m.t.d} = h_bF_m.t.d = 1225 \cdot 10^3 \text{ Nmm} \]  \hspace{1cm} (5.22)

\( F_{m.t.d,\text{real}} \) = design tensile capacity of the maanwi connection (due to embedment failure).

\( F_{m.t.d} \) = the design tensile capacity of the maanwi at the height of the wall.

\( d_{p1} = 80 \text{ mm} \) is the distance of maanwi (widest part of taper) to the edge of the perpendicular beam.

\( d_{p2} = 95 \text{ mm} \) is the distance of maanwi (smallest part of taper) to the edge of the perpendicular beam.

\( a_m = 125 \text{ mm} \) is the the distance from the maanwi to the wall.

\( w = 460 \text{ mm} \) is the width of the wall.

\( h_b = 175 \text{ mm} \) is the height of the perpendicular beam and location of maanwi connection.

\( t_m = 45 \text{ mm} \) is the thickness of the maanwi.
5.3.5. Friction capacity

Besides the initial friction capacities of the layers, which is depending on the vertical load per layer and the friction coefficient, there is a remaining amount of friction capacity in the layer. The contact area of the beams will reduce, as the rotation of the perpendicular beams will lift the beam above the layer up. A part of the vertical force is transferred through these connections and the remaining part will contribute to the friction capacity of the layer. Due to the rotation of the perpendicular beams, the longitudinal beam will deflect and will be transferring vertical load to the layers below. The perpendicular beams in the corners will carry the remaining vertical load. This leads to the following assumed $\alpha$-values:

For 5 layers, corners are 30% of length of wall: $\alpha = 0.7$
For 7 layers, corners are 20% of length of wall: $\alpha = 0.8$

$$F_{fr} = \mu \alpha F_{v, tot} \quad (5.23)$$

$F_{v, tot} = l (\sigma_v + h_x \rho)$, where:
- $l = 2995$ mm is the length of the wall,
- $\sigma_v = 0.067$ N/mm$^2$ is the pre-compression on the wall,
- $\rho = 18.94$ N/mm$^3$ is the density of the wall,$^3$
- $\mu = 0.35$ is the friction coefficient.
- $h_x$ = height of the wall above considered layer, where $x$ is the number of layers above the considered layer.
- $\alpha$ = vertical weight distribution factor, based on the surface area of the layer in contact in vertical direction.

5.3.6. Rotational beam capacity

$$x_n = w_b - \frac{F_{v,n,d}}{2 w_b f_{c,90,k}}$$  $$x_{n,t} = w_l - \frac{F_{v,n,d}}{2 w_b f_{c,90,k}} \quad (5.24)$$

Where,
- $F_{v,n,d} = \frac{1}{4} (1 - \alpha) F_{v, tot}$ = vertical force on the beams and infill pieces,
- $x_n$ = maximum eccentricity of pre-compression force on beam no. $n$,
- $x_{n,t}$ = maximum eccentricity of pre-compression force on long infill piece no. $n$,
- $w_b$ = width of the beams,
- $w_l$ = length of the longitudinal infill piece and $w_l = 3.6 \cdot w_b$.

---

$^3$ A stone density of 2500 kg/m$^3$ is used instead of the 2590 kg/m$^3$ specified in paragraph 3.4.
Consequently, the horizontal force component is calculated as follows (directly in the virtual work equation):

\[
F_r = \frac{x_n F_{v,n,d}}{h_{b,n}}
\]

\[
F_{r,l} = \frac{x_n L F_{v,n,d}}{h_l}
\]  

(5.25)

Where, \( h_{b,n} \) is the height of rotating beam or infill piece at location \( n \) and \( h_i \) the height of the infill piece.

Due to the global moment on the structure and the rotation that is related to this moment, the vertical stress distribution in the wall changes. More compression will be present at the right side of the wall, whereas at the left side of the wall the compression force will reduce and can even become zero. This phenomenon will create an iterative loop in the calculation and it not taken into account.

5.3.7. Failure mechanism

Three layer failure mechanisms are considered, composed from the combined left and right corner connection failures in figure 5.14.

Failure mechanism A1: combination of Figure 5.14a and Figure 5.14c.
Failure mechanism A2: combination of Figure 5.14a and Figure 5.14d.
Failure mechanism A3: combination of Figure 5.14b and Figure 5.14c.

For a more detailed overview of the failure mechanisms see appendix G. The failure mechanism giving the lowest force value according to the virtual work assessment, is given below:

Failure mechanism A3:

In this mechanism the left-hand side failure is governed by the rotation of the perpendicular beams and not by the infill pieces. The right hand side corner connection failure is. Only the right corner connection has an internal difference in rotation and the maanwi's will on this side. The maximum lateral in-plane loaded wall force capacities with this governing failure mechanism for the different walls shown in figure 5.1 are given in table 5.8.
5.3.8. **Conclusion: in-plane loaded wall model**

The lowest lateral in-plane wall capacity is 57 kN for wall number 4. It can be observed that for all of the walls the critical layer failure will happen in the top layer (layer number 1). The friction contribution to the total lateral force capacity is approximately 90 % for the first layer and increases to approximately 96 % for the lower layers. The further down the wall, the friction resistance is playing a more significant role.

This analytical model is based on a lot of assumptions (paragraph 5.3.2) and not all possible mechanisms are taken into account, which does not lead to a lower bound assumption of the force. In future research the analytical model should be upgraded, improving the assumptions taken for this model.

Following, a list of the uncertainties in the in-plane loaded wall model:

- Material input values; experimental tests should be performed to validate the material characteristics.
- The friction coefficient established in paragraph 3.5. This value is assumed looking at similar cases in literature. Experimental tests should be performed to validate the friction coefficient(s) for a Kath-Kuni wall.
- Assumed $\alpha$-value, for the amount of bending of the longitudinal beams, in reality the $\alpha$-value is depended on the amount of rotation of the perpendicular beams
- Connection models: Johansen’s model for the *kadils* and the *maanwi* should be improved. Full capacity curves of the connections should be implemented to make sure the initial stiffness and post-failure behaviour of the total wall can be taken into account.
- The model is not incremental: every displacement step, inducing different possible rotations configurations in the perpendicular beams and infill pieces, should be assessed for its lowest amount virtual work (several boundary conditions should be applied to the possible configurations). The configuration with the lowest amount of virtual work is determining the failure configuration.
Calculating the actual work will consequently assess the lateral force needed to obtain this configuration. In the next displacement step the virtual work should be calculated with the previous displaced configuration as basis.

- The model does not take global wall behaviour into account, like the global moment resisting capacity of the wall and the combined behaviour of the layers when proper kinematics of the system would be in place.

### 5.4. Lateral capacity of full building

From just a wall, now the translation will be made to the full buildings capacity. For this assessment the ELF method is used, where first the base shear force in the building is determined and converted to the maximum allowable PGA for the full building. The lateral design capacities of the different walls in the building are the maximum lateral capacities of the walls in the building below the first storey. The ground floor level walls are constructed with dry stone masonry and is not taken into account in the wall model. To assess the maximum PGA capacity of the building, knowing the maximum lateral force capacity of the Kath-Kuni wall part ($V_{\text{max-wall}}$), the base shear force needs to be calculated on ground level ($V_{\text{b-building}}$) (Figure 5.23). In order to do this, the following lateral force distribution formula is used:

$$Q_i = V_b \frac{W_i h_{ii}^k}{\sum_{i=1}^j W_i h_{ii}^k}$$

Where,

- $Q_i$ is the design lateral force at floor $i$,
- $W_i$ is the seismic weight of floor $i$,
- $h_{ii}$ is the height of floor $i$ measured from the base,
- $j$ is the number of floors where the masses are located,
- $k$ is 1 for a linear distribution and 2 for a quadratic distribution.

This formula is however actually not valid for buildings with a non-homogenous composition over the height of the building and it is expected that the actual wall shape for Kath-Kuni buildings will be non-continuous. Nevertheless, the formula will be used to get an initial insight in the buildings behaviour. The distribution in IS 1893 (Part 1) is assuming to be quadratic. However, EN 1998-1 states that if the mode shape is approximated by horizontal increasing linearly over the height of the structure. The results are given for both distributions. The weight of half of the height of the walls is calculated to be located at the floor levels. For the roof 2/3 of the height is assumed to be located at $F_5$ and 1/3 of the height is located at $F_4$ (see figure 5.23). The forces are assumed to be located at the position of the floors as in most buildings the floors contain most of the mass. However, for Kath-Kuni buildings only 17.5% of the total mass is located in the floors and roof (incl. the mass of the timber balcony façade). The mass of the roof is higher, compared to the mass of the floors, but it is spread over 2.6 meters height. It means that the force distribution in reality is more continuous. Nonetheless, modelling the shear force in a discrete way gives an approximation of reality. Because, the force is determined based on the time period of the complete building and it is assumed that the stiffness of the building is predominantly governed by the stiffness of the in-plane walls, the total lateral force needs to be resisted by the in-plane walls.
The input value $V_{\text{max-wall}}$ is transferred to the base shear capacity of the building taking into account the contributions of the seismic weight on every floor level. The seismic weight of the building is calculated in paragraph 4.1. The used values for the seismic weight/storey can be found in table 5.9, where the finally used values are obtained from Figure 4.3. It is important to note that for the seismic capacity of the in-plane loaded wall the pre-compression load of just the in-plane loaded wall is considered, which is the direct vertical force acting down in the walls. However, for the demand the seismic weight of the either 0.25 times the total seismic mass of the building (for four transverse walls) or 0.5 times the total seismic mass of the building (for two longitudinal walls) is considered. The longitudinal walls are furthermore divided in three separate wall sections. Hence, in all the inertial seismic mass of the building is assumed to be resisted by the in-plane loaded walls. In reality, the out-of-plane loaded walls also contribute to the seismic resistance of the building and the total mass considered to be acting on the in-plane loaded walls can be reduced.

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<th>No. floor level</th>
<th>Actual seismic weight/storey</th>
<th>Seismic weight/meter</th>
<th>Seismic weight/storey</th>
<th>Seismic redistributed weight</th>
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<td>1119.7</td>
<td>90</td>
<td>976</td>
<td>-</td>
</tr>
</tbody>
</table>

The PGA results are given in table 5.10, where both the linear and quadratic lateral force distribution are taken into account.
Table 5.10: In-plane loaded wall capacity with related maximum allowable PGA

<table>
<thead>
<tr>
<th>Wall no.</th>
<th>Wall length</th>
<th>Total seismic weight (base level)</th>
<th>Vmax – wall (from capacity calculation)</th>
<th>Linear</th>
<th>Quadratic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Vb – building (PGA capacity)</td>
<td>Vb – building (PGA capacity)</td>
</tr>
<tr>
<td>1</td>
<td>4.8 m</td>
<td>1089 kN</td>
<td>59 kN</td>
<td>65.0 kN</td>
<td>0.06 g</td>
</tr>
<tr>
<td>2</td>
<td>5.4 m</td>
<td>801 kN</td>
<td>66 kN</td>
<td>72.7 kN</td>
<td>0.09 g</td>
</tr>
<tr>
<td>3</td>
<td>4.8 m</td>
<td>712 kN</td>
<td>59 kN</td>
<td>65.0 kN</td>
<td>0.09 g</td>
</tr>
<tr>
<td>4</td>
<td>4.6 m</td>
<td>683 kN</td>
<td>57 kN</td>
<td>62.8 kN</td>
<td>0.09 g</td>
</tr>
</tbody>
</table>

The dry-stone masonry ground floor and the top timber floors are not taken into account in the calculation. However, for the dry masonry ground floor level an indicative assessment is performed. The friction resistance of the ground floor is the seismic weight up to the ground floor times the frictional coefficient $\mu = 0.35$. For shear failure in the ground floor, only wall number 4 needs to be assessed, hence this is the shortest wall and therefore is subjected to the least amount of pre-compression force. The assessment results in a lateral force capacity in wall number 4 of 89 kN, which is significantly higher than the base shear force expected on the ground floor (62.8 kN).

The timber top floor level is not evaluated, and it is now assumed that this floor is able to withstand the lateral forces acting on this floor level up to the maximum lateral force capacity of the Kath-Kuni wall. More research is needed to validate this statement.

5.4.1. Conclusion: full building

The most significant wall turns out to be the wall in transverse direction of the building, with a length of 4800 mm and a height of 2953 mm. The wall capacity is 59 kN, which leads to a required base shear capacity of 65 kN and a maximum allowable PGA of 0.06 g.

There are several uncertainties in the in-plane loaded wall calculation. They are listed below:

- The input value for the time period $T$; now calculated by making use of an empirical formula in the Indian Seismic Design code for different construction principles.
- The input value for the behaviour factor $R$; is now taken as standard value from IS 1893 (Part 1), but the expectation is that an research to the ductility of the wall can significantly increase this $R$-value.
- The amount of damping of the structure; now set as 5% (in the elastic response spectra)
- The assumption that the seismic forces can be modelled in a discrete manner acting on the locations of the floors, instead of a continuous manner, which is expected for a Kath-Kuni building, where most of the mass is located in the walls.
- The lateral force distribution over the height of the building is not realistic. Due to the non-homogenous construction configuration over the height of the building, the mode shape cannot be determined. There will be kinks in the mode shapes, as the stiffness of the different parts of the building (dry-stone masonry plinth, Kath-Kuni walls and timber top storey) is significantly different, which will lead to different time periods. This multi-degree-of-freedom system cannot be approximated in a realistic manner by a single-degree-of-freedom system with a single stiffness value.
- The assumption that the out-of-plane loaded walls do not contribute to the in-plane wall force capacity and stiffness might not be valid, as they will restrict the in-plane wall from lifting up as a consequence of the connectivity in the corners. This will enhance the capacity of the in-plane walls significantly. This will also reduce the amount of inertial mass that needs to be calculated to be resisted by the in-plane loaded walls.
5.5. Conclusion

The input parameters taken from the code are based on other building typologies for which the parameters are determined in IS 1893 (Part 1) (mostly by empirical formulas). The material properties and friction coefficient used in this thesis are determined in chapter 3. The material properties are converted from characteristic values to design values where different values for the partial safety factor and load duration factor are used for the different directions in which the walls are assessed.

In order to assess the lateral force capacity of the total case study building, the ELF method is applied. behaviour factor $R$ is used to take the ductility of the building into account. The total wall natural frequency of the complete building is determined by the stiffness of the in-plane walls. The out-of-plane walls are assessed for the local out-of-plane wall natural frequency and the maximum resistible PGA is obtained. This capacity PGA is compared to the demand PGA obtained from several sources (paragraph 1.1). This local frequency of the wall in out-of-plane direction is depending on the stiffness of the out-of-plane loaded wall itself and is governed by the stiffness of the wall in horizontal direction. The timber beams spanning in horizontal direction generate much more stiffness than the dry-stone masonry can do in the vertical direction. Because, the vertical stiffness is neglected in the calculations, the total out-of-plane wall can be assessed by looking at the individual layers. The shape function of the out-of-plane wall deflection shows that the governing wall layer is the top layer in the wall. Per layer, the total vibrating mass is assumed to be the timber beam plus the mass of one stone layer. The force acting on the stone layer can be brought to the timber beams by friction (assumed as $\mu = 0.35$), hence the vertical pre-compression force is sufficiently large. Lower in the wall, less force will be attracted, as less mass will be activated. The amount of pre-compression is however higher in the lower layers. Therefore, it can be stated that only the top out-of-plane loaded wall layer need to be assessed to validate the total out-of-plane wall.
The maximum allowable PGA the out-of-plane wall can resist turns out to be ranging from 0.53 g to 13.53 g, where a behaviour factor $R$ is taken as 1.0. This large range is caused by the different stiffness’s assumed in the model for the corner connection and the cooperation between the horizontal timber beams. Figure 5.10 shows that the capacity of the wall is high compared to the demand. In the lower bound model with the lowest stiffness assumed for the corner connection and the beam cooperation, the capacity will be insufficient for a PGA with RP of 2475 years (2% probability of exceeding in 50 years) established by a deterministic approach, but is sufficient to resist an PGA with RP of 475 (2% probability of exceeding in 50 years). The capacity seems to be sufficient to resist the MCE established in IS 1893 (Part 1).

Compared to a masonry building, the out-of-plane wall of a Kath-Kuni structure is not depending on box-working generated by the floor diaphragms. However, the Kath-Kuni walls have horizontal timber beams which bring the force acting on the out-of-plane wall to the in-plane walls directly. Even if the deflections in the wall are substantial, the maximum damage most likely to be developed is the infill stones falling out of the wall. Local house owners claim that these stones can be hammered back in place easily. Because box-action is ensured, the focus of this thesis goes to the in-plane wall behaviour. The linear-elastic out-of-plane wall calculations showed that despite the flexible floor diaphragms, the out-of-plane wall capacity is sufficient for its own local mode of vibration.

Contradictory to the straight forward out-of-plane loaded wall model, the in-plane wall behaviour is much harder to evaluate. In the in-plane analysis a differentiation is made between the global wall behaviour and the local wall layer behaviour (from which the latter turned out to be critical).

The global capacity of the wall is defined by the friction capacity of the wall at the bottom layer. When the lateral force exceeds 93.6 kN, the wall is expected to undergo global sliding. The layers above are expected to have additional lateral force mechanisms, due to the connected timber beams and infill pieces.

The internal layer capacity is assessed by modelling three different failure mechanisms. Not all mechanisms which contribute to the total virtual work are taken into account, resulting in a possible overestimation of the capacity of the layer. Furthermore, the contribution of the global behaviour of the wall is neglected (moment resisting capacity and wall shape), and the model is not incremental. Last, no non-linear behaviour of the connections is taken into account.

The layers are compared by making use of the virtual work approach. The final failure mechanism resulting in the lowest amount of virtual work is shown in figure 5.22. The wall is expected to fail in the top layer, with a lowest capacity of 57 kN of wall number 4. The ELF method is used to assess the maximum allowable PGA of the wall in its design capacity. The lateral in-plane loaded wall capacity is converted to the maximum allowable base shear capacity of the building. This is done by assuming a linear and quadratic distribution over the height of the building. The base shear force is converted to a maximum allowable PGA of 0.06 g for the most critical wall number 1. This wall is most critical as most of the seismic mass needs to be resisted by these walls.

This PGA does not represent the expected seismic performance of a Kath-Kuni building. There are many uncertainties in the input parameters, like the time period, $R$-value (ductility), material characteristics and lateral force distribution over the height of the building (displaced shape). Furthermore, the in-plane loaded wall model contains a lot of assumptions and does only take very simplified connection models into account. With the model the stiffness and non-linear behaviour of the wall cannot be computed.
To create a full engineering understanding of the actual wall behaviour and to assess the validity of the input parameters (time period $T$ and behaviour factor $R$) obtained from generalisations in IS 1893 (Part 1), an in-plane loaded wall experiment will be performed.
PART III – EXPERIMENTAL ASSESSMENT
6. EXPERIMENTAL SETUP

In chapter 5, an Equivalent Lateral Force (ELF) analysis is performed and results conform the empirically proved seismic capacity of the considered Kath-Kuni building located in Old-Jubbal. The ELF method takes non-linear performance into account by lowering the demand with a behaviour factor $R$, which is assumed to be 2.5. Furthermore, the time period of the building is determined by making use of an empirical formula. Both parameters are not based on the vernacular Kath-Kuni structures. This chapter presents the overview of the experimental program followed. For this thesis, the focus is directed to one quasi-static in-plane loaded wall pushover test, which results in an initial estimation of an appropriate behaviour factor $R$ and time period $T$. A Kath-Kuni wall was built and tested in the laboratory of the Indian Institute of Technology Roorkee. There are two main limitations of the experimental approach to the Kath-Kuni wall used for this thesis:

1. The building construction is non-homogenous over the height of the building and the results from a Kath-Kuni wall cannot be scaled to the full building without assessing the behaviour of the dry-stone masonry built ground floor level and timber built 3rd floor level.
2. Due to size restrictions of the setup, the evaluated Kath-Kuni wall is a scaled version of the actual Kath-Kuni wall in the case study building. However, the slenderness ratio is kept the same. The tested wall had 5 layers, whereas the actual wall consists of 7 layers (Figure 6.1). To be able to scale the experimental wall to the real wall dimensions, an elaborate analytical model is required, which is not covered in this thesis. The dimensions of the experimental wall and the original wall have been presented in Table 6.1.

Last, an important outcome of the experimental program is to increase the engineering understanding of the lateral capacity of an in-plane loaded wall. The damage development was monitored, and a qualitative evaluation of the behaviour is presented in this chapter.

(a) Jubbal house slenderness measurements

(b) In-plane loaded wall in transverse direction

Figure 6.1: Overview of specimen size relative to the total building
### 6.1. Experimental program

This master thesis covers part of a larger experimental program set up at Indian Institute of Technology Roorkee. The project code for the larger experimental program is ‘KK01’ (KK01 stands for the first series of Kath-Kuni tests performed at Indian Institute of Technology Roorkee in 2017-2018). This experimental program contains several sub parts: EP00 – EP05.

#### Table 6.1: Wall dimensions of the case study building and the tested wall

<table>
<thead>
<tr>
<th></th>
<th>Full scale wall (case study building)</th>
<th>Tested wall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Longitudinal direction</td>
<td>Transverse direction</td>
</tr>
<tr>
<td></td>
<td>Min.</td>
<td>Max.</td>
</tr>
<tr>
<td>Length</td>
<td>4600 mm</td>
<td>5400 mm</td>
</tr>
<tr>
<td>Height</td>
<td>3200 mm</td>
<td>3200 mm</td>
</tr>
</tbody>
</table>

#### Table 6.2: Overview of total experimental program KK01 performed at Indian Institute of Technology Roorkee

<table>
<thead>
<tr>
<th>Experimental program</th>
<th>Experimental program parts</th>
<th>Description</th>
<th>Complete?</th>
</tr>
</thead>
<tbody>
<tr>
<td>KK01</td>
<td>EP00 Design of setup</td>
<td>Design of the in-plane pushover setup</td>
<td>✓</td>
</tr>
<tr>
<td></td>
<td>EP01 Masonry small</td>
<td>Tests to obtain masonry material properties</td>
<td>✓</td>
</tr>
<tr>
<td></td>
<td>EP02 Kath-Kuni small</td>
<td>Tests to obtain Kath-Kuni material properties</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>EP03 Friction</td>
<td>Test to obtain friction coefficients of materials use in Kath-Kuni construction</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>EP04 Kadil connection</td>
<td>Tests to the lateral force and stiffness capacity of the kadil corner connection</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>EP05 Pushover</td>
<td>Quasi-static in-plane loaded wall pushover tests</td>
<td>✓</td>
</tr>
</tbody>
</table>

All of the experimental program parts are subdivided in the different tests. Experimental program part EP05 is divided in four tests, where the first tested masonry wall had as aim to validate the experimental setup. The two Kath-Kuni pilot tests are the wall tests with concrete blocks instead of stone masonry. The last test performed is a Kath-Kuni wall with dry-stone masonry infill (Table 6.3).

#### Table 6.3: Tests in KK01EP05

<table>
<thead>
<tr>
<th>Specimen name</th>
<th>Material</th>
<th>Dimensions [mm]</th>
<th>Overburden</th>
<th>Boundary conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>IITR_PILOT_01</td>
<td>Clay brick masonry</td>
<td>3040 220 2670</td>
<td>0.34 MPa</td>
<td>Cantilever</td>
</tr>
<tr>
<td>IITR_PILOT_02</td>
<td>Deodar timber, concrete blocks M35</td>
<td>2995 460 2110</td>
<td>0.067 MPa</td>
<td>Cantilever</td>
</tr>
<tr>
<td>IITR_PILOT_03</td>
<td>Deodar timber, concrete blocks M35</td>
<td>3000 460 2038</td>
<td>0.065 MPa</td>
<td>Cantilever</td>
</tr>
<tr>
<td>IITR_KK_01</td>
<td>Deodar timber, natural dry stone masonry</td>
<td>3000 460 2013</td>
<td>0.065 MPa</td>
<td>Cantilever</td>
</tr>
</tbody>
</table>

This report evaluates only IITR_PILOT_02 quantitatively. Some qualitative results from IITR_PILOT_03 and IITR_KK_01 are used as well (paragraph 7.4).
6.2. Monotonic in-plane loaded wall specimen

During the assessment of the case study building, several carpenters were interviewed about the exact configuration of this building style. Consequently, one of the carpenters was hired to build several specimens in the laboratory at the Indian Institute of Technology Roorkee. Some pictures of the wall building process can be found in figure 6.2 and figure 6.4.

![Figure 6.2: Wall building process](image)

Initially, for the IITR_PILOT_02 specimen evaluated in this thesis (Figure 6.3), left over stones were obtained from a village near Rishikesh at the foothills of the Himalayas. But the stone quality was rejected by the carpenters and hence not used. The traditional stone shaping skills have slowly diminished, thereby limiting the capacity of carpenters to work with a variety of stone qualities. This resulted in the decision to use concrete blocks instead. For IITR_KK_01 test, higher quality stones, as necessitated by the carpenters, were procured and the test was performed with all traditional materials in place. The blocks had approximate dimensions of 500x230x0.10 mm, determined by roughly averaging the stone size used in the case study building. The blocks were prepared in the laboratory with concrete M35, as this is a commonly used concrete grade in the field (paragraph 3.2).

The specimen had a layer of these concrete blocks at the bottom which, in turn, was glued to the concrete floor underneath. The rest of the wall was built over this layer, without any mechanized connection between the two. The wall had a length of 2995 mm, a height of 2027 mm (measured from the top of the glued concrete block layer) and a width of 460 mm. The IITR_PILOT_02 specimen is depicted in figure 6.3. The wall has 5 timber layers and is placed (without connection) to the layer of concrete blocks glued to the floor.
6.2.1. Horizontal timber beams

Simulating the Kath-kuni construction style, the specimen had horizontal layers of Deodar timber throughout the height of the wall, running parallel to the bottom concrete blocks’ layer. There were five such timber layers with three layers of concrete blocks between every two successive timber layers. Each of these 5 layers consisted of two Deodar timber sleepers of cross section 127x203 mm (5"x8") each, with dry stone rubble filled between them. The rubble used is the wastage of the dry stone cutting and coarse aggregates. The final shaping of the timber logs resulted in average dimensions of the timber sleepers of approximately 100x175 mm (4”x7”).

The timber horizontal sleepers were interlocked at the corners. To create more space in between the corner connections height, timber infill pieces were used. These infill pieces were ideally shaped to the right height in order to compensate for slight height differences in the concrete. The vertical space in between two timber beams is filled up with the concrete blocks (three layers). The horizontal space in between two timber beams is filled up with rubble, which is the wastage of the dry stone cutting and course aggregates.

6.2.2. Kadil and maanwi connections

The timber beams are in the corners connected with each other in vertical direction with kadils. The kadils are timber dowels (or tenon) (approximately 45x45x90 mm). The kadils are manually hammered in the their pockets, with a small hammer. Per corner of one single layer, 10 kadils were used as vertical connectors. At each of the 5 timber layers, the two sleepers are horizontally connected to each other through ‘maanwi’s’. A maanwi is used as a dovetail connection. Each layer has two maanwi’s placed at approximately one meter distance from each other.
EXPERIMENTAL SETUP

The Kath-Kuni test was performed on the wall, with perpendicular timber beams at the corner connections that protruding of the wall for approximately 300 mm. This provided sufficient space for a maanwi connection to be applied. The maanwi in the overhanging perpendicular timber beams is important for the degrees of freedom depicted in figure 6.5. The degrees of freedom \( u_z, u_x \) and \( \theta_y \) are partly (with a certain unknown stiffness) restrained by the maanwi connections. One maanwi was fixed in each of the two overhangs of every perpendicular timber beam, where it ensured that the boundary conditions for the corner connection are closely met with the actual corner connection situation.

Figure 6.5: Degrees of freedom that the maanwi (partly) restrains

The specimen dimensions and other relevant parameters can be found in table 6.4. For the exact dimensions and the derivations of the weights see report appendix I.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Symbol</th>
<th>Unit</th>
<th>IITR_PILOT_02</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>-</td>
<td>-</td>
<td>Kath-Khuni with concrete blocks</td>
</tr>
<tr>
<td>Slenderness ratio</td>
<td>( h / l )</td>
<td>mm</td>
<td>0.70</td>
</tr>
<tr>
<td>Height wall</td>
<td>( h )</td>
<td>mm</td>
<td>2027(^2) (2110)</td>
</tr>
<tr>
<td>Height timber layer</td>
<td>( h_b )</td>
<td>mm</td>
<td>175</td>
</tr>
<tr>
<td>Height concrete/stone layer</td>
<td>( h_s )</td>
<td>mm</td>
<td>288</td>
</tr>
<tr>
<td>Length wall</td>
<td>( l )</td>
<td>mm</td>
<td>2995</td>
</tr>
<tr>
<td>Thickness wall</td>
<td>( w )</td>
<td>mm</td>
<td>460</td>
</tr>
<tr>
<td>Distance for eccentricity of the load</td>
<td>( h_l )</td>
<td>mm</td>
<td>220</td>
</tr>
<tr>
<td>Weight of the top steel/concrete system</td>
<td>( W_T )</td>
<td>kN</td>
<td>7.10</td>
</tr>
<tr>
<td>Applied load in vertical actuators</td>
<td>( F_V )</td>
<td>kN</td>
<td>85.00 (82.45(^2))</td>
</tr>
<tr>
<td>Self-weight wall</td>
<td>( W )</td>
<td>kN</td>
<td>52.59</td>
</tr>
<tr>
<td>Overburden stress top wall</td>
<td>( \sigma_v )</td>
<td>MPa</td>
<td>0.067 (0.065(^2))</td>
</tr>
<tr>
<td>Axial stress at mid-height</td>
<td>( \sigma_{v-mid} )</td>
<td>MPa</td>
<td>0.086 (0.083(^2))</td>
</tr>
<tr>
<td>Density</td>
<td>( \rho )</td>
<td>kN/m(^3)</td>
<td>18.94</td>
</tr>
<tr>
<td>Compression strength timber ( \perp ) (mean)</td>
<td>( f_{c90m} )</td>
<td>MPa</td>
<td>2.8(^3)</td>
</tr>
<tr>
<td>Compression strength concrete/stone</td>
<td>( f_c )</td>
<td>MPa</td>
<td>Min. 30</td>
</tr>
<tr>
<td>Amount of kadils per plane</td>
<td></td>
<td></td>
<td>6/8 -&gt; 4(^4)</td>
</tr>
</tbody>
</table>
Lateral resistance \textit{kadil} corner connection \hspace{1cm} MPa \hspace{1cm} 20º

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Coefficient of friction t-c/s ( \mu )</td>
<td>-</td>
<td>0.35º</td>
</tr>
<tr>
<td>Coefficient of friction r-c/s ( \mu )</td>
<td>-</td>
<td>0.35º</td>
</tr>
</tbody>
</table>

1. This value is the actual height of the wall without the bottom concrete layer. In the calculation, the total height is taken into account, which is noted in between brackets.
2. This value was originally determined, but adapted during the test.
3. See table 3.2.
4. Varying between 6 and 8 \textit{kadils} per plane per over the total wall length. In the 3\textsuperscript{rd} layer of the specimen, by accident, 4 \textit{kadils} (instead of the planned 6 \textit{kadils}) were placed in between the longitudinal beams and the infill pieces.
5. These values are approximations, see paragraph 3.5.

### 6.3. Test setup

In order to perform the experimental quasi-static tests in the laboratory at the Indian Institute of Technology Roorkee, a test setup was specially designed (Figure 6.7). The setup consists of the following elements (see Figure 6.6 for the part numbering):

- Kath-Kuni wall specimen (Figure 6.3)
- Horizontal hydraulic actuator with roller system (1)
- Vertical hydraulic jack (2)
- Concrete reaction wall (3)
- Test pit with four concrete foundation blocks (4)
- Reaction beam connected with to two columns to horizontal steel beams (5)
- Steel spreader beam (6)
- Steel load distribution beam (glued on the Kath-Kuni wall) (7)
- Stabilizing system (8)
- Out-of-plane rolling frames (two on each side of the wall) (9)
- Safety ropes (10)

Figure 6.6: Schematisation of test setup in Sketch-Up
A detailed description of the test setup can be found in appendix H. The test setup ensured cantilever boundary conditions for the specimen. The test involved applying a pre-compression load, spread to two points through a simply supported steel beam, at the top of the wall using hydraulic jack. A stiff steel load distributing beam, glued to the specimen was used to distribute the precompression load over the complete top layer of the specimen. A reaction beam was attached to both sides of the wall at the foundation of the test pit and had a rotational degree of freedom. The horizontal actuator, attached to the concrete reaction wall, was used to apply a displacement-controlled force on the steel load distributing beam trough rollers. This ensured that the wall was free to move up and downwards. A stabilizing frame was also installed at the top of the load distributing beam and connected to the reaction beam to ensure that the lateral load on the vertical jack be minimized. An out-of-plane resisting frame is fabricated to ensure that out-of-plane movement is minimalized at the height of the main load distributing beam. An important characteristic of a pushover test setup is that the vertical force should not influence the horizontal movement of the wall.

As Kath-Kuni walls are a craftsman product, every wall built has slightly different dimensions. For each of the four variants tested, walls are built in the test pit and the test setup was adjusted to the exact height measurements of the new wall. The variation in the wall heights was specially taken into account by changing the height of the out-of-plane roller systems and the horizontal actuator and varying the height of the reaction beam or applying filler plates under the vertical jack.

A schematization of the final setup design can be found in figure 6.6. The setup was tested by performing a quasi-static pushover test on a masonry wall specimen and validating it to results of the same test performed at the TU Delft (Esposito, Meulman, Jafari, & Ravenshorst, 2016) (Esposito & Ravenshorst, 2017).

### 6.3.1. Missing connection in 3rd layer

During the building of the wall, two of the twenty *kadil* connections in the third layer were accidentally left out. This led to a final failure in this layer, hence the missing *kadils* triggered other possible failure mechanisms. The *kadils* that were missing had to be implemented in between the longitudinal infill piece and the longitudinal beam (Figure 6.8). The change in configuration was diligently taken into account and is presented in the validation calculations in paragraph 7.3.
6.3.2. Loading scheme and instrumentation

For the loading scheme and instrumentation see appendix I and J.
Chapter 6 describes the basis for the testing of Kath-Kuni wall specimen IITR_PILOT_02. In this chapter the results are presented and analysed. The results are validated by making use of the same analytical model as used in paragraph 5.3, but then scaled to the dimensions as the IITR_PILOT_02 specimen. Moreover, for this calculation mean timber values are used.

In the analysis an initial estimation of the behaviour factor $R$ and the time period $T$ is made, which is used to re-assess the in-plane loaded wall by using the Equivalent Lateral Force (ELF) method as performed in paragraph 5.4.

Furthermore, an important outcome of this research is the engineering understanding of the behaviour of a Kath-Kuni wall, which is a non-straightforward and non-engineered construction technology. Qualitative results, which are mostly obtained by pictures taken during the test, are therefore important. The pictures are related to points on the obtained force-displacement curve in order to generate an understanding of the failure development over the duration of the test.

### 7.1. Obtained data

The results are captured in photos, a time laps movie and force and displacement readings. This paragraph gives an overview of the most important data. The main force-displacement diagram is showing the force in the horizontal hydraulic actuator and the displacement measurements taken by Linear Variable Differential Transformer (LVDT) 1 (Figure 7.1). LVDT1 is located at the point of application of the horizontal hydraulic actuator. A table describing the Load Steps (LS) taken during the test can be found in appendix K.

![Figure 7.1: LVDT 1 Force - displacement diagram with numbering](image-url)
The wall is tested in a quasi-static manner by applying a displacement controlled lateral force. The load is applied in by taking LS and stopped every 5 kN or 5 mm (10 mm towards the end of the experiment). During the breaks the dial gauge readings are taken, and photographs are made. A final displacement of 198.8 mm is reached. In figure 7.2, figure 7.3, figure 7.4, figure 7.5 and figure 7.6, the final failure mechanisms in the wall can be observed, where internal rotation in the third layer is governing. It can be observed that the maanwi connections are broken and several perpendicular beams and infill pieces are cracked.

**Figure 7.2:** Main damage in layer no. 3 at the left side

**Figure 7.3:** Main failure at layer no. 3 at the right side
Figure 7.4: Damage at the back right side of the wall

Figure 7.5: Damage at layer no. 4/5 at the right side
7.2. Data analyses

7.2.1. Description of wall failure

The wall failure is described by making use of pictures taken during the loading breaks (LS). The pictures in figure 7.8 and table 7.1 combined are highlighting the main damage development in the wall during the test (for the full photo documentation see appendix L). In figure 7.10, the observations are summarized in the force-displacement curve.

First of all, it should be noted that the specimen turned out to be not consistently made. In layer 3 (where the main sliding happened), only 9 kadils instead of 10 were present per corner (Figure 6.8). This influenced the behaviour of the wall significantly. However, the data still provides insight in the behaviour of the Kath-Kuni connections and the failure development in the wall and the data is giving a rough idea of the behaviour of the structure. Furthermore, the importance of the kadils in a Kath-Kuni wall proved itself, as the failure of the wall is expected in the top layer and now occurred in the third layer. As a consequence of this missing kadil and the fact that only two kadils were placed in the perpendicular direction infill piece, the infill piece on the left side of the wall could freely rotate, (also called “sliding” in this report).

Figure 7.7: Different failure mechanisms expected
The main difference in possible failure mechanisms between the two cases is the connectivity in between the longitudinal beam and longitudinal infill piece. Full connectivity is required in case of full connection (see difference between 1 in figure 7.5a and 2 in figure 7.5b). The only way of ensuring this is when only the perpendicular beams will rotate (Figure 5.14b), or when the longitudinal infill piece will rotate (Figure 7.5a). However, due to the large length of the longitudinal infill piece, this rotation will have a lot of resistance from the surrounding structure. Uplift is prevented by the stones in between the perpendicular beams and the weight of the wall coming from above. This failure is expected to require a lot of work. In reality it could be possible that the kadil connections at location 1, will fail and slip in order to generate more space for the longitudinal infill piece to stay horizontal.
Figure 7.8: Wall failure documentation (see also Table 7.1)

Figure 7.9: Kadil failure at location a. depicted in (a)
Table 7.1: Description of wall behaviour at the different Load Steps (LS)

<table>
<thead>
<tr>
<th>Zone #</th>
<th>Zone</th>
<th>LS #</th>
<th>End load [kN]</th>
<th>LVDT 1 Displ. [mm]</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Linear Elastic Force increase</td>
<td>v</td>
<td>-1.2</td>
<td>-0.06</td>
<td>A slight compression of the layers is visible in figure 7.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>5.6</td>
<td>0.32</td>
<td>First small cracks in longitudinal beam 3rd layer left side (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>8.8</td>
<td>0.64</td>
<td>Visible bend in the wall (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>10.4</td>
<td>0.98</td>
<td>A slight compression of the layers is visible in figure 7.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>15.9</td>
<td>1.29</td>
<td>First small cracks in longitudinal beam 3rd layer left side (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>23.6</td>
<td>2.20</td>
<td>Visible bend in the wall (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>28.8</td>
<td>3.50</td>
<td>First small cracks in longitudinal beam 3rd layer left side (see figure 7.8)</td>
</tr>
<tr>
<td>2</td>
<td>Non-linear Plastic Force increase</td>
<td>7</td>
<td>33.9</td>
<td>6.34</td>
<td>Crack in infill piece 2nd layer left side (see figure 7.8), crack at location of kadil and crack at the right bottom corner of infill piece</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>37.4</td>
<td>11.08</td>
<td>Blocks in upper layer shifted (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>9</td>
<td>39.8</td>
<td>15.41</td>
<td>Infill piece 4th layer right side crushed (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10</td>
<td>42.2</td>
<td>20.43</td>
<td>Infill piece 4th layer right side crushed (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>47.8</td>
<td>48.52</td>
<td>Infill piece 4th layer right side crushed (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>50.0</td>
<td>57.40</td>
<td>Significant bend in the wall from the 1st to the 3rd layer (figure 1.1)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>17</td>
<td>48.5</td>
<td>66.75</td>
<td>Significant bend in the wall from the 1st to the 3rd layer (figure 1.1)</td>
</tr>
<tr>
<td>3</td>
<td>Non-linear Plastic Constant force</td>
<td>18</td>
<td>50.2</td>
<td>76.04</td>
<td>Left side rotation of 3rd layer starts, causes uplift in the 3rd layer (see uplift graphs in Figure 7.14, Figure 7.15, Figure 7.16 and Figure 7.17</td>
</tr>
<tr>
<td></td>
<td></td>
<td>19</td>
<td>49.5</td>
<td>85.89</td>
<td>Peak load reached of 51.6 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>20</td>
<td>50.2</td>
<td>95.28</td>
<td>Peak load reached of 51.6 kN</td>
</tr>
<tr>
<td></td>
<td></td>
<td>21</td>
<td>49.5</td>
<td>104.44</td>
<td>First cracks in maanwi 3rd layer right front side (back side unknown) (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>22</td>
<td>42.8</td>
<td>114.27</td>
<td>First peak in rotation of the wall in the beginning of LS 22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>23</td>
<td>42.9</td>
<td>123.55</td>
<td>Crack in infill piece left side 3rd layer (see Figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>24</td>
<td>38.8</td>
<td>133.33</td>
<td>First peak in rotation of the wall in the beginning of LS 22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>25</td>
<td>37.1</td>
<td>143.00</td>
<td>Complete failure maanwi 3rd layer right side (front and back) (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>26</td>
<td>39.6</td>
<td>152.35</td>
<td>Complete failure maanwi 3rd layer right side (front and back) (see figure 7.8)</td>
</tr>
<tr>
<td>4</td>
<td>Non-linear Plastic Force decrease</td>
<td>27</td>
<td>42.4</td>
<td>161.64</td>
<td>Failure of concrete blocks 4th layer right side (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28</td>
<td>41.6</td>
<td>171.15</td>
<td>Failure of concrete blocks 4th layer right side (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>29</td>
<td>44.5</td>
<td>179.84</td>
<td>Failure of concrete blocks 4th layer right side (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>42.1</td>
<td>189.12</td>
<td>Failure of concrete blocks 4th layer right side (see figure 7.8)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>31</td>
<td>37.8</td>
<td>198.78</td>
<td>Failure of concrete blocks 4th layer right side (see figure 7.8)</td>
</tr>
</tbody>
</table>
The wall compresses under the vertical loading. In the first linear phase the wall reaches a force of approximately 33.9 kN and a displacement of 3.5 mm. This results in an elastic stiffness of 9.7 kN/m. Some small cracks emerged in the timber and the wall starts bending in the upper layers. Cracks in the infill pieces are the onset of the first non-linear phase.

At a horizontal load of 37.4 kN the concrete blocks in the upper layer are slightly shifted, which ensures an upper bound of the concrete friction coefficient of 0.41 (at a pre-compression load of 92.1 kN). Most likely, the actual concrete friction coefficient is less than 0.41, as the timber connections in the layer also generate lateral resistance. At a horizontal load of 39.8 kN an infill piece in the 4th layer crushes. From a point with a displacement of 76.0 mm and a force of 50.2 kN, the force is more or less constant, for approximately 20 mm of displacement. This is caused by the onset of the internal rotation in the 3rd layer (onset of plastic mechanism). This means that the infill pieces start to rotate. Because the rotation requires more vertical space, the timber infill pieces are pushing the wall upwards. This generates an increase in relative uplift in the 3rd layer. At 51.6 kN the peak load is reached and the wall. A “sliding” starts to be visible, which is actually the internal rotation of the infill pieces in the 3rd layer. Concrete friction plays a reduced role here, as a part of the vertical force is carried by the rotated timber infill pieces. The concrete blocks can therefore slide more easily.

After the first cracks in the *maanwi’s* emerge on the right side of the wall (front side) at a displacement of 95.3 mm, the force starts to drop. Because the wall is asymmetrical over the width, the *maanwi* in the front side initially needs to resist more force. Because of the cracks in the front *maanwi*, increased rotation is possible now in the right corner connection in the 3rd layer. Until this stage, the wall continued rotating, and reaches its first peak at 107.3 mm. Slightly before reaching this point one of the concrete blocks at the 4th layer at the right side started to fail. The global rotation of the wall decreases. The rotated infill piece at the 3rd layer at the left side starts to crack and the perpendicular beam in this corner attached to the cracked infill piece also splits open along the grain. This allows for more rotation on the left side, as the *maanwi* restricts the perpendicular beams from separating/rotating. At a displacement of 133.3 mm, the *maanwi’s* in the 3rd layer on the right side...
EXPERIMENTAL RESULTS

breaks completely. The force is still decreasing, and cracks emerge in the longitudinal beam at the right side in the 3rd layer, as the timber bed of the kadil connection starts to fail. This is caused by a significant internal rotation. After reaching a force of 39.6 kN and a displacement of 152.4 mm, the global rotation of the wall increases, and this results in an increase in lateral capacity as well. The kadil connections in the third layer generate additional capacity, which makes that the wall above this layer starts rotating “globally” on top of the 3rd layer. Figure 7.12 shows the rotation of the wall, calculated with the results of the vertical LVDTs 5 and 6 (Figure 7.13). It can be observed that the rotation decreases with the first force capacity dip of the wall, when the internal rotation is the main failure mechanism. The wall rotation (rotating on top of the 3rd layer) increases after the kadil connections are activated to their ultimate capacity.

After reaching a force of 44.5 kN, a displacement of 179.8 mm and a “sliding” in the 3rd layer of 87.0 mm (Table 7.2), the difference in displacement between layer 3 & 4 (at LS 29), the force drops again. The concept of internal rotation is shown in figure 7.11. It is shown that after a certain displacement in the 3rd layer, the vertical pre-compression force cannot longer be transferred, as the eccentricity exceeds w* (calculated to be 86.8 mm). As a consequence, the lateral capacity of this layer (and thus the capacity of the total wall) decreases after reaching this limit.

![Figure 7.11: Final perpendicular beam rotation in the 3rd layer](image)

While the force is decreasing, the global rotation of the wall increases twice more. The infill pieces on the left side are completely crushed.

From the relative uplift graphs in figure 7.14, figure 7.15, figure 7.16 and figure 7.17, it can be observed that the relative uplift at the right side of the wall is larger at the front, hence the longitudinal infill piece is located here, which is pushed upwards. On the back side more play is present between the concrete blocks. The perpendicular infill piece is rotating at the left-hand side and generates a vertical uplift on both the front and back side of the wall. It can be observed in all graphs that first a compression is measured, as a consequence of the vertical applied precompression load (overburden). The left-hand side uplift is starting immediately after onset of the horizontal displacement of the wall. The relative uplift on the right-hand side is fist negative as a consequence of the global wall rotation and the large accumulated precompression force on the right-hand side of the wall. After the onset of the internal rotation failure in the third layer, the relative uplift becomes positive.
Figure 7.12: Rotation of the wall (based on the vertical displacements in LVDT 5 and 6)

Figure 7.13: Vertical displacement of the wall

Figure 7.14: Uplift in the layers 3 and 4 at the right front side (wooden infill piece is located at the front side of the wall)

Figure 7.15: Uplift in the layers 3 and 4 at the right back side

Figure 7.16: Uplift in the layers 1, 2, 3 and 4 at the left front side

Figure 7.17: Uplift in the layers 1, 2, 3 and 4 at the left back side
The displacement of the separate layer is given in Table 7.2. The displaced shape of the wall at the height of the different layers is plotted against the horizontal displacement in LVDT 1 in Figure 7.18.

Table 7.2: Displacement values and in most significant LS (layer 1 is the top layer and layer 5 the bottom layer)

<table>
<thead>
<tr>
<th>Height</th>
<th>Points</th>
<th>Ductility</th>
<th>El. Stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0</td>
<td>6</td>
<td>17</td>
</tr>
<tr>
<td>Layer 5</td>
<td>184</td>
<td>0,0</td>
<td>0,1</td>
</tr>
<tr>
<td>Layer 4</td>
<td>647</td>
<td>0,0</td>
<td>0,1</td>
</tr>
<tr>
<td>Layer 3</td>
<td>1130</td>
<td>0,0</td>
<td>0,6</td>
</tr>
<tr>
<td>Layer 2</td>
<td>1573</td>
<td>0,0</td>
<td>1,8</td>
</tr>
<tr>
<td>Layer 1 (right)</td>
<td>2036</td>
<td>0,0</td>
<td>3,5</td>
</tr>
<tr>
<td>Layer 1 (left)</td>
<td>2036</td>
<td>0,0</td>
<td>3,5</td>
</tr>
</tbody>
</table>

Difference 1 & 3 [mm]

<table>
<thead>
<tr>
<th>Load [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0,0</td>
</tr>
</tbody>
</table>
7.2.2. Estimation of behaviour factor R

The behaviour factor R describes the ductility of the wall, by evaluating the elastic displacement and the ultimate displacement of the bilinear capacity curve. The ultimate displacement is determined as the displacement at $0.8 \times V_{\text{max}}$ (Esposito & Ravenshorst, 2017). In the force-displacement curve shown in figure 7.19 it can be observed that the force drops at a displacement of approximately 170 mm (slightly under $0.8 \times V_{\text{max}}$), but then regains its capacity. EN 1998-1 states that the maximum computed displacement cannot exceed 150% the ultimate displacement, to ensure a certain safety in the calculation. However, the ultimate displacement obtained by doing experimental research is more certain, than a value obtained by modelling. Furthermore, the other Kath-Kuni wall tested in this experimental program (see paragraph 7.4) shows a higher maximum displacement than the 193 mm reached by specimen IITR_PILOT_02 (measured in the last LS). This higher displacement could be reached because the wall did not have a connection mistake in any of the layers and did not undergo a significant failure behaviour in the different layers. When all layers will contribute to the total wall deformation, a larger displacement can be reached.

The determination of the behaviour factor $R$ is based on the surface under the elastic linear curve and the elasto-plastic curve. The highlighted areas in Figure 7.20 are the same. The $R$-value is the factor that can be applied to the reduced force $V_2$, in case of an increased ductility. The ductility $\mu$ can be calculated with formula (7.1) (NPR 9998, 2017).

$$\mu_b = \frac{u_u}{u_{el}} = 16.5$$

(7.1)

The approach shown in figure 7.20 can be captured in formula (7.2).

$$R = q = \sqrt{2\mu_b - 1} = 5.7$$

(7.2)

This value is a factor 2.3 times the $R$-value obtained from IS 1893 (Part 1). However, this value needs to be scaled for the actual wall calculated in paragraph 5.3 with a height of 7 layers. In order to scale the wall an elaborate analytical model is needed which can take into account the influence of the increased height and the increased length on both the strength and the stiffness of the wall. The analytical model used for this thesis can estimate the shear force, but not the stiffness and non-linear behaviour. Furthermore, the missing connection in the 3rd layer had an influence on the stiffness and moreover the ultimate displacement. However, a qualitative assumption can be made.
The elastic stiffness of the wall with additional connections (without missing kadils) is not expected to be significantly different but is expected to increase as a consequence of the additional connections. This will decrease the yield displacement. Nevertheless, the ultimate displacement capacity of the wall is expected to increase significant, because redistribution of forces in the non-linear regime will ensure contribution of every layer to the deformed shape of the wall. This will lead to a higher ductility factor $\mu$ and consequently a higher behaviour factor $R$. Therefore, this wall is giving a lower bound R-value estimation for a Kath-Kuni wall with the same dimensions.

The stiffness of the wall with 7 layers will decrease, hence the displacement is expected to increase significantly with every added layer (especially in a wall where no connections are missing). The lateral force is not expected to increase significantly for an increased height of the structure but does increase linearly with an increase length. However, the displacement increase of the wall over the height is of higher order. Hence the higher the wall, the lower the expected stiffness and the higher the yield displacement ($u_{el}$). Also, the final displacement (of a wall with all the connections in place) is assumed to increase over the height of the wall. Even though, there will be some slight difference in the increase in yield displacement and ultimate displacement, the R-value will be approximately of the same order.

In the re-assessment of the Kath-Kuni wall, the R-value of the tested size Kath-Kuni wall is expected to give a lower bound for the full-size Kath-Kuni wall.

A Kath-Kuni building is always a non-homogeneous building over the height of the structure. The stiffness is expected to decrease significantly, due to the complete timber floor level on top of the Kath-Kuni wall. The ultimate displacement will evenly increase. These are very rough estimations and in order to assess this fully, the full building needs to be evaluated. This is not done in this thesis.

### 7.2.3. Estimation of time period $T$

Formula (7.3) is used in EN 1998-1 to determine the time period of the wall, which is based on a single-degree-of-freedom (SDOF) system:

$$T = 2\pi \sqrt{\frac{m d_y}{V_u}}$$  \hspace{1cm} (7.3)

The values for the yield displacement ($d_y$) and ultimate force ($V_u$) can be found in paragraph 7.2.2. The time period furthermore depends on the seismic mass activated. To assess the time period for the full building
several simplification assumptions need to be made. Due to the non-homogenous construction typology of the building over the height, the building can in reality not be approximated by a SDOF system. Every construction typology, as the dry-stone masonry plinth and the complete timer 3rd floor have their own stiffness and therefore their own time period. Because the stiffness the total building, due to the timber floor level, is expected to be significantly lower than when the building would be assessed as a full Kath-Kuni structure (and the time period will therefore be higher), the time period assessment can be considered as a lower bound. When the time period exceeds a value of 0.4 s, the elastic spectral acceleration coefficient can be reduced. The SDOF is assumed to start from the top of the ground floor level (above dry-stone masonry plinth) as it is established that the friction resistance is sufficient to withstand seismic forces and this floor level will not contribute significantly to the deformed shape and time period of the building.

Other characteristics that will enhance the lower bound assumption of the time period, are the fact that a high component of the mass is located in the roof. As assessed in the previous paragraph, the stiffness of the scaled wall (of 7 layers) is expected to be lower than the stiffness of the 5-layered tested wall. Which, again will lead to a lower bound of the time period.

The seismic mass used for the estimation of the time period is given in figure 7.21, where the contribution of the top floor levels to the behaviour of the total building is completely neglected. The seismic mass of the out-of-plane wall is assumed to be resisted by the in-plane walls. The seismic mass for the loading in the direction of the transverse walls is 34200 kg/wall (in reality the mass is more for wall number 3, than for wall number 2 or 4, because this wall is located in the middle and more out-of-plane loaded wall mass will contribute to the seismic mass of this in-plane loaded wall) and 22800 kg/wall for the longitudinal walls. This results in an absolute lower bound time period for transverse wall number 1 (Figure 5.1) of:

\[
\text{Transverse: } T = 0.58 \text{ s} \quad \rightarrow \quad \frac{S_a}{g} = \frac{1}{T} = 1.7 \tag{7.4}
\]

\[
\text{Longitudinal: } T = 0.48 \text{ s} \quad \rightarrow \quad \frac{S_a}{g} = \frac{1}{T} = 2.1 \tag{7.5}
\]

Where the formulas (7.4) and (7.5) used to calculate the spectral acceleration coefficient are obtained from IS 1893 (Part 1), where they are describing the decreasing part of the spectrum of figure 5.2 for Type I surfaces (rock or hard soil).

Figure 7.21: Assumed SDOF, to estimate a lower bound for the time period of a Kath-Kuni building
7.3. Analytical validation model

To validate the experimental results the same in-plane loaded wall model is used as in paragraph 5.3. However, now the same dimensions are used for the model as measured from the experimental specimen. The wall has 5 layers and a length of 2995 mm. The remaining dimension are given in table 5.1. Hence this is a validation calculation, mean timber values are used (see paragraph 1.5). Similar to the 7-layer wall calculation, first the global capacity values are calculated and given in figure 7.22.

The pre-compression per layer and its related expected lateral force capacity, taking into account a friction coefficient $\mu = 0.35$, is shown in the table below.

<table>
<thead>
<tr>
<th># layer</th>
<th>Total precompression force</th>
<th>Lateral force $V$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>96.9 kN</td>
<td>33.9 kN</td>
</tr>
<tr>
<td>2</td>
<td>109.0 kN</td>
<td>38.1 kN</td>
</tr>
<tr>
<td>3</td>
<td>121.0 kN</td>
<td>42.4 kN</td>
</tr>
<tr>
<td>4</td>
<td>133.1 kN</td>
<td>46.6 kN</td>
</tr>
<tr>
<td>5</td>
<td>145.2 kN</td>
<td>50.8 kN</td>
</tr>
</tbody>
</table>

Several failure mechanisms are proposed and the mechanism for which the least amount of energy is needed for it to fail will be the governing failure mechanism. Due to a mistake during construction of the wall, two kadil connections are missing in the 3rd layer (Figure 6.8). The locations of the missing kadil connections are highlighted in figure 5.13.

Therefore, the in-plane wall analyses will be performed for two possible configurations:

A. A layer with all kadils in place: three mechanisms (same as in calculation in paragraph 5.3)
B. A layer with a kadil connection missing in between the longitudinal upper beam and the longitudinal infill piece: four mechanisms (Figure 7.24)
The *kadil* and *maanwi* capacities changed, because for this assessment mean material properties are used. Table 7.5 describes the strength of the connections assuming mean material values.

**Table 7.5: Connection mean strength values**

<table>
<thead>
<tr>
<th>Mechanism</th>
<th>Max. capacities</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Option 1</td>
<td>Option 2</td>
</tr>
<tr>
<td><em>kadil</em>: $F_{k,dA}$ (Johansen equations)</td>
<td>$\beta = \frac{f_{c.0,k}}{f_{c.90,k}} = 8.9$</td>
<td>$\beta = 1$</td>
</tr>
<tr>
<td></td>
<td>9.5</td>
<td>2.3</td>
</tr>
</tbody>
</table>

The final failure mechanisms expected to occur in the wall are given separately for the top layer (complete connections) and the third layer (missing *kadil* connections).
**Failure mechanism A3 – complete connections:**
This failure mechanism is the same as the failure mechanism described in paragraph 5.3.7.

![Figure 7.25: Failure mechanism experimental Kath-Kuni wall: complete](image)

**Failure mechanism B3 – missing kadil connections (third layer):**
The ***maanwi*** on the left-hand side will pull beam no. 4 along with beam no. 3. (see figure 7.23 for the beam numbers). The rotation between the beams is equal. Sliding can happen between the longitudinal infill piece and the longitudinal beam, because the *kadil* connection between the two is missing.

![Figure 7.26: Failure mechanism experimental Kath-Kuni wall: missing kadil connections](image)

The failure mechanism observed in the test is failure mechanism figure 7.24a for the left side and figure 7.24c for the right side, where the right side failure can also be observed in the calculated failure mechanism in Figure 7.26. The calculated failure mechanism shows rotation in the perpendicular beams on the left side of the wall, whereas in the actual mechanism the perpendicular infill piece is rotating. This can be a consequence of the redistribution of forces. Where in the initial displacement phases the failure mechanism will be conform figure 7.24b, but during larger displacements the mechanisms changes to figure 7.24a.

The virtual work calculation results in the maximum lateral force capacity values shown in table 7.6. It can be seen that the missing *kadil* in the third layer decreases this layer’s capacity from 51 kN to 47 kN compared to the full connected situation. The experimental specimen fails in the third layer from the top, because in this layer two connections are missing. In order for the third layer to fail, the capacities of the layers on top need to be larger than the capacity of the third layer with the missing connections. In table 7.6 it can be observed, that despite the missing connections in the third layer, the top layers have a slightly lower capacity compared to layer three. The analytical calculation results are therefore not conform the experimental results.

The maximum force capacity is determined as 42 kN. Compared to the actual wall capacity of 51.6 kN, the error is about 18.6% compared to the experimental value. The accuracy compared to the bi-linear curve (shown in
(figure 7.19) is approximately 10%, which is a more valid comparison, as the analytical model is a linear model. As explained in paragraph 5.3, the model needs significant improvement. The force is underestimated, because not all the work contribution mechanisms are taken into account, such as the global moment resisting capacity. This also means that the force capacity of the full-scale wall is underestimated, which gives an underestimated maximum allowable Peak Ground Acceleration (PGA). The only over estimation of the force could be in the fact that not all possible failure mechanisms are considered.

The model is based on a lot of assumptions, noted down in paragraph 5.4.1. In this paragraph it is also stated which adaptations need to be made to the model to most likely result in a better validation. Furthermore, a parameter study needs to be performed to research the sensitivity of the different parameters like:

- Dimensions of the wall
- Kodil connection capacity
- Maanwi connection capacity
- Distribution factor $\alpha$

<table>
<thead>
<tr>
<th></th>
<th>Compete wall</th>
<th>Missing kodil in 3rd layer</th>
<th>Only friction ($\mu = 0.35$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>42 kN</td>
<td>“</td>
<td>33 kN</td>
</tr>
<tr>
<td>Layer 2</td>
<td>46 kN</td>
<td>“</td>
<td>38 kN</td>
</tr>
<tr>
<td>Layer 3</td>
<td>51 kN</td>
<td>47 kN</td>
<td>42 kN</td>
</tr>
<tr>
<td>Layer 4</td>
<td>55 kN</td>
<td>“</td>
<td>46 kN</td>
</tr>
<tr>
<td>Layer 5</td>
<td>59 kN</td>
<td>“</td>
<td>50 kN</td>
</tr>
</tbody>
</table>

### 7.4. Qualitative results from other tests

Specimen IITR_PILOT_03 (February 2018) is a wall with the same materials and configuration as the assessed wall in this thesis (IITR_PILOT_02). The layers showed a deformation, which is increasing over the height of the structure. The top layer clearly showed to most significant failure. Some detailed photos are shown in figure 7.27.

![Figure 7.27: Final failure IITR_PILOT_03](image.png)
EXPERIMENTAL RESULTS

Specimen IITR_KK_01 (May 2018) is the only wall built with natural stone masonry. The wall is slightly lower than the IITR_PILOT_02 wall. The largest internal rotation failure happened in the first (top) layer (Figure 7.28 and Figure 7.29).

Important to notice is that the displacement capacity of the walls IITR_PILOT_03 and IITR_KK_01 are significantly higher than for IITR_PILOT_02. The walls showed no dip in the force-displacement curve and were not tested until their ultimate capacity, due to restrictions in the test setup. In both tests, the contribution in displacement is more distributed over the different layers of the wall than for IITR_PILOT_02. Because there is no significantly weaker layer, constant redistribution of forces is resulting in internal rotation in every layer. However, in both tests in can be observed that the top layer undergoes the most significant failure. Here the maanwi’s finally break.

In figure 7.27a and in figure 7.29, it can be seen that the longitudinal infill piece on the left side of the wall is rotated slightly, which is also seen in figure 5.14a.
During the testing of wall IITR_KK_01, in first instance it looked like layer number 2 would fail. However, at a certain point this failure shifted to the first layer, which is the layer expected to fail. This highlights that Kath-Kuni structures, are very prone for slight inconsistencies in either material or configuration. The wall is built by hand and no wall looks exactly the same. It is therefore likely to observe the consequences of local weaknesses in the structure.

<table>
<thead>
<tr>
<th></th>
<th>IITR_PILOT_02</th>
<th>IITR_PILOT_03</th>
<th>IITR_KK_01</th>
</tr>
</thead>
<tbody>
<tr>
<td>Which layer:</td>
<td>3rd</td>
<td>1st</td>
<td>1st</td>
</tr>
<tr>
<td>Type of failure:</td>
<td>Internal rotation</td>
<td>Internal rotation</td>
<td>Internal rotation</td>
</tr>
<tr>
<td>Similar computed mechanism:</td>
<td>Mechanism 1</td>
<td>Combination of mechanism 5 &amp; 7</td>
<td>Combination of mechanism 5 &amp; 7</td>
</tr>
</tbody>
</table>

7.5. Conclusion

The experimental work gave a good insight of the behaviour of the wall under lateral loading. Because of the IITR_PILOT_02 test, it became clear what the effect is of two missing kadils in the corner connections. The difference in possible failure mechanisms is highlighted in figure 7.7. The failure of the wall is governed by internal rotation in the layer. The perpendicular beams and infill pieces rotate which causes uplift in the layer. The maanwis ensure that the perpendicular beams need to rotate the same distance. However, at a certain displacement the maanwi breaks, giving the perpendicular beams space to rotate independently (which requires less energy for the structure). When the maanwi’s break, the forces redistribute to the kadil connections again and due to the large displacement, the embedment of the kadil connections break. The uplift of the wall reduces and this induces an increased friction capacity of the wall. Together with the enhanced kadil capacity, this causes regaining of the force capacity in the wall. The onset of the ultimate failure starts at the point where the vertical force cannot be transmitted through the perpendicular beams, due to large rotations. The force drops, and the wall loses its capacity. Due to setup restrictions, the test needed to be stopped, just at the moment when this force was expected to drop, and future additional tests need to be performed to be able to define the ultimate displacement capacity with more certainty.

Figure 7.30: Actual 3rd layer failure (a, b, d, e) versus the expected third layer failure (c, f)
The prediction of the global and the local analytical model, suggest that the wall will be prone to local layer failure above global sliding failure in the bottom layer. This is validated by the experimental results. However, the actual layer failure mechanism turns out to be different at the left side of the wall compared to the calculated mechanism in the validation model (Figure 7.30). The analytical model does not take large displacements into account and it is based on a lot of assumptions (paragraph 5.3).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Code input values</th>
<th>Revised values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T$</td>
<td>0.21 &amp; 0.38</td>
<td>0.48 0.58</td>
<td>s</td>
</tr>
<tr>
<td>$S_a/g$</td>
<td>2.5</td>
<td>2.1 1.7</td>
<td>-</td>
</tr>
<tr>
<td>$I$</td>
<td>1</td>
<td>1 1</td>
<td>-</td>
</tr>
<tr>
<td>$R$</td>
<td>2.5</td>
<td>5.7 5.7</td>
<td>-</td>
</tr>
<tr>
<td>$W$</td>
<td>4785.9</td>
<td>4785.9 4785.9</td>
<td>kN</td>
</tr>
<tr>
<td>$\mu$</td>
<td>0.35</td>
<td>0.35 0.35</td>
<td>-</td>
</tr>
</tbody>
</table>

**7.6. Revised equivalent lateral force analysis for full building**

The ELF assessment performed in paragraph 5.4 is revised in this paragraph. For the ELF method, the maximum lateral force capacity for the 7 layered full size walls is converted to the maximum allowable base shear force in the full building. By making use of the spectral acceleration coefficient (based on the time period $T$ of the building), the behaviour factor $R$ and the importance factor, a maximum allowable PGA is derived. In paragraph 7.2.2, a new value for the behaviour factor $R$ (ductility) is obtained, which is 2.3 times higher than the initial value obtained from IS 1893 (Part 1). The new value for the time period $T$ is calculated in paragraph 7.2.3. An increased time period means a reduction in the spectral acceleration coefficient. The derivation of these values is based on taking lower bound approximations in scaling the experimental output to the output of the total building. To be able to properly scale up the experimental results and derive the behaviour factor $R$ and time period $T$, either full scale experimental tests or an elaborated analytical model is needed that can evaluate the maximum force capacity, initial stiffness and ductility of the wall. To get a rough idea of the change in parameters and the consequence of this change has for the evaluated in-plane loaded wall capacity on full building scale, the input values shown in table 7.8 are used. The revised PGA values are shown in table 7.9.

<table>
<thead>
<tr>
<th>Wall no.</th>
<th>Wall length</th>
<th>Linear</th>
<th>Quadratic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$V_b$ building</td>
<td>$P_{GA}$ capacity</td>
</tr>
<tr>
<td>1</td>
<td>4.8 m</td>
<td>65 kN</td>
<td>0.06 g</td>
</tr>
<tr>
<td>2</td>
<td>5.4 m</td>
<td>71.5 kN</td>
<td>0.09 g</td>
</tr>
<tr>
<td>3</td>
<td>4.8 m</td>
<td>65 kN</td>
<td>0.09 g</td>
</tr>
<tr>
<td>4</td>
<td>4.6 m</td>
<td>63 kN</td>
<td>0.09 g</td>
</tr>
</tbody>
</table>

The PGA capacity results for the results are depicted in figure 7.31, where they are compared to the PGA demand values obtained from IS 1893 (Part 1) and literature (paragraph 1.1). The revised PGAs are a factor 2.5 - 3.2 times higher than the PGAs calculated in paragraph 5.4. It can be concluded, that the input parameters
obtained from the Indian Seismic Building code are insufficiently representing the seismic behaviour of Kath-Kuni structures. The value of 0.18 g is not representing the safety of the Kath-Kuni structures as it is lower than an expected demand PGA with a Return Period (RP) of 475 years. Because the determination of the revised behaviour factor $R$ and time period $T$ are still very rough lower bound estimations, more research to obtain the actual values is recommended.

Figure 7.31: Full building PGA demand versus capacity results revised
PART IV – CONCLUSIONS
Kath-Kuni is a centuries old Himalayan building style, whose knowledge has been passed on from generations and improved over time. Empirically, Kath-Kuni architecture shows good seismic resistance. The walls of the building are built up from horizontal timber beams interlocked in the corners and filled up with dry stone masonry in between.

The main objective of this research was to find an answer to the question; “which earthquake features and embedded traditional knowledge of Kath-Kuni walls are essential in generating adequate seismic performance?”. Linear static calculations of the laterally loaded out-of-plane walls show that the horizontal timber beams are generating box-action for the building by transferring the local lateral force to the in-plane loaded walls. Hence, no flexible floor diaphragms are required to generate box-action. The in-plane loaded walls are providing most of the stiffness of the building and determine the building’s total capacity. The linear static calculations use a behaviour factor R to take non-linear wall behaviour (ductility) into account. An in-plane pushover wall test is carried out which showed that the behaviour factor and the vibration time period of the building, obtained from standard formulas from IS 1893 (Part 1) (IS 1893 (Part 1), 2002), are too conservative for Kath-Kuni structures. An in-plane loaded wall model demonstrates the building’s high ductility, one of its most important earthquake resistant features. Other important earthquake resistant features found during this research are:

- The timber connections (kadil dowel connection and maanwi dovetail connection), which are allowing internal rotation in the layer and are acting in parallel and series with one another and generate robustness.
- The lack of vertical reinforcement is leaving the wall free to deform in vertical direction without damage.
- High contribution of friction between stone and stone-timber in the wall.

The secondary objective was to investigate why people, despite the proven seismic performance, are abandoning their traditional homes and replacing them with (poorly designed) concrete homes. Via a sustainability study, it has been investigated whether the traditional Kath-Kuni building technology can still contribute to generating environmentally and economically resilient communities in the mountains.

Elaborate conclusions are divided into paragraphs according to the flowchart depicted in figure 1.18.

8.1. Field research

Several field trips to various places in the Indian Himalayas showed the diversity in the construction configuration of Kath-Kuni buildings. Loss of traditional skills, because of the gradual reduction in the number of Kath-Kuni houses being built, enhances this diversity. However, all traditional Kath-Kuni houses are characterised by double horizontal timber beams in thick load bearing walls, which are connected to one another by timber dovetail connections called ‘maanwi’s’. The vertical space between the timber beams is filled
up by dry-stone masonry. The lateral strength of masonry without mortar is predominantly based on friction. In the corners, the beams are connected in horizontal direction by timber dowels called ‘kadils’ (Figure 8.1b). Flexible timber floors and the lack of floor-wall connections, ensure that these flexible floors diaphragms do not contribute to the box-action of the building. All calculations performed are based on a building located in Old-Jubbal, Himachal Pradesh, which is chosen as case study in this thesis (Figure 8.1a). The total building consists of a dry-stone masonry ground floor level, a Kath-Kuni part and a top floor level consisting of pure timber construction. In this thesis, only the capacity of the Kath-Kuni part of the building is assessed and this research therefore gives an indication of the total building seismic capacity.

![Figure 8.1: Overview of construction configuration of case study building](image)

### 8.2. Background information to seismic assessment

Different background information is required for the calculations performed in this thesis, such as information on the seismic building codes, the material characteristics and the demand peak ground acceleration values. The Indian seismic design code ‘IS 1893 (Part 1)’, is used as basis for the seismic design calculations. Since IS 1893 (Part 1) still uses the conservative working stress approach for the design of timber structures, Eurocode 8 ‘EN 1998-1’ (EN 1998-1, 2004) is consulted for guidance in using the Limit State approach.

Permissible timber material properties (used for working stress calculations) are obtained from the Indian timber design code ‘IS 883’ (IS 883, 1994), which are converted to mean and characteristic values. For design calculations, the characteristic material properties are used. A duration factor and a partial safety factor are applied to obtain the design timber properties, which vary for the different ductility classes of the walls in-plane (ductility class: high) and out-of-plane direction (ductility class: low). When an analytical model is compared to the experimental results, mean material properties are used.

Probabilistically and deterministically determined demand peak ground acceleration values are obtained from literature. Probabilistic values are ranging between 0.075 g and 0.732 g, with a return period of 2475 years and they are ranging between 0.0039 g and 0.289 g with a return period of 475 years. A zone factor $Z$ is used in IS 1893 (Part 1) for the design of structures for a maximum considered earthquake and a design basis earthquake. Because the code is still working with the working stress method for some materials, these values are lower and not representing the actual expected peak ground accelerations in the region. To convert the code values to realistic peak ground acceleration values, a factor of 1.5 is applied to the code values in this thesis (derived from the partial safety factor used for seismic concrete design). Finally, no partial safety factor is applied to all obtained (and converted) demand peak ground acceleration values, in accordance to the Limit State approach.
8.3. Seismic behaviour

The total Kath-Kuni building is evaluated for its seismic performance by making use of the linear static Equivalent Lateral Force method. The results are obtained as maximum allowable peak ground accelerations on the separate components of the building. Non-linear behaviour (ductility) can be taken into account by applying a behaviour factor $R$ to the force capacity to allow designing the structures for resistance to seismic forces smaller than those corresponding to a linear elastic response. The behaviour factor $R$ and the time period $T$ of the structure are input parameters obtained from IS 1893 (Part 1), valid for other more conventional construction typologies and their validity for Kath-Kuni structures is therefore questionable and hence investigated in this research. In order to assess the total building for the maximum peak ground acceleration which can be resisted during an earthquake, the in-plane loaded wall and the out-of-plane loaded wall are researched separately and for a single direction at the time. A wall has a significantly higher stiffness in the in-plane direction than in the out-of-plane direction, hence the in-plane loaded walls are evaluated for the global mode of vibration of the building. However, the walls are prone to vibrations in out-of-plane direction as well and because rigid floor diaphragms are lacking, other box-action generating features are required to prevent out-of-plane wall failure.

8.3.1. Out-of-plane loaded wall

The out-of-plane loaded walls are assessed for their local vibration mode. The existence of flexible floor diaphragms prevalent in Kath-Kuni construction highlights the importance of this assessment. Important box-action generating feature of the Kath-Kuni out-of-plane loaded walls is their robust connectivity with the in-plane loaded walls and the horizontal timber beams spanning in between the in-plane loaded walls. The maximum peak ground acceleration an out-of-plane loaded wall can resist is determined by making use of a modelling range, which accounts for the uncertainty in the connectivity and stiffness of the corner connections and the maanwi’s in between the beams. The time period $T$ is derived for the bounds of the stiffness range and since the ductility of walls in this direction is uncertain, a behaviour factor $R$ is assumed to be 1.0. Nevertheless, the linear out-of-plane wall model gives a maximum allowable lower bound peak ground acceleration of 0.53 g and the upper bound peak ground acceleration of 13.53 g. The lower bound value is higher than the design demand peak ground acceleration value obtained from IS 1893 (Part 1) for a design basis earthquake and similar to the design demand peak ground acceleration value of a maximum considered earthquake. The values are also exceeding the peak ground acceleration obtained from literature with a return period of 475 years (significant damage Limit State). The wall’s absolute lower bound capacity shows to be insufficient to withstand an earthquake with a peak ground acceleration with a return period of 2475 years (near collapse Limit State). However, this lower bound calculation is conservative since pinned supports (connection with the in-plane loaded wall) are assumed. Moreover, a behaviour factor larger than 1.0, reflecting non-linear behaviour, is not considered. Therefore, it can be concluded that an out-of-plane loaded wall is not critical for the seismic capacity of the total building.

The out-of-plane wall design improves with higher rigidity in the connections. Furthermore, when a connector in the middle of the wall is applied, a more robust out-of-plane loaded wall behaviour is expected. Instead of dominant one-way bending of the layers, the wall will also be able to transfer loads in vertical direction (two-
way bending. In many Kath-Kuni buildings in Uttarakhand, a shear key is applied at the walls prone to out-of-plane failure which distributes the lateral force over the total wall (Figure 8.2). It also connects the two parallel walls of the building.

8.3.2. Full building (in-plane loaded wall model)

The in-plane wall is more complicated to evaluate, because of the clear inability of the wall to resist significant lateral forces in the elastic range. First, a global wall capacity evaluation is performed, where the wall is considered as one solid element. However, internal layer failure is more likely than global failure and is studied making use of a analytical in-plane loaded wall layer model. Several failure mechanisms are assessed by making use of a virtual work approach. The kadi connections, maanwi connections, beam rotations and friction are the contributing elements to the total lateral force capacity of a layer. The global moment resisting capacity of the wall has not been taken into account in the current calculations. This needs to be implemented in the future improved models.

The maximum force capacity for the kadi and maanwi connection are derived from connection models. Since the models are simplified assumptions, they are contributing to the uncertainty in the model. This resulted in expected failure in the top layer with a maximum force capacity of 59 kN for a wall of 7 layers and 4800 mm long. This force is converted to a maximum allowable base shear force at ground floor level by making use of a linear and quadratic lateral force distribution valid for structures which are homogenous over the height of the building. Because of the different stiffnesses of the different construction methods in a Kath-Kuni building as the approximation of this multi-degree-of-freedom by a single-degree-of-freedom system is very rough, both these distributions are not applicable for Kath-Kuni structures but the actual deformed shape is expected to be somewhere in between (with kinks at the change of stiffness’s). Furthermore, the unknown displacement shape of the building also results in an unknown amount of activated seismic mass over the height of the building, hence the amount of activated seismic mass used in the equivalent lateral force calculation is most likely overestimated. Furthermore a larger part of the activated seismic mass might be directly transferred to the ground than now is assumed. The equivalent lateral force method is not able to take higher order degree of-freedom systems into account, however in this research this method is used to perform an initial assessment to this non-straightforward construction technique. The base shear force is converted to the maximum allowable peak ground acceleration on the structure by using a behaviour factor $R$ of 2.5 and a time period $T$ of 0.38 s. This leads to a maximum allowable peak ground acceleration of 0.06 g. This peak ground acceleration is not in accordance to the empirically proven seismic performance of a Kath-Kuni building. This led to setting up and performing a quasi-static in-plane wall experiment to investigate the validity of the standard behaviour factor $R$ and the time period $T$ values obtained from IS 1893 (Part 1).

8.3.3. Quasi-static in-plane loaded wall experiment

A Kath-Kuni wall specimen is built and tested in the laboratory of the Indian Institute of Technology Roorkee. Quasi-static in-plane loaded wall pushover tests are performed on a specimen with the same slenderness ratio as the actual walls. However, smaller dimensions had to be adopted, due to the restrictions of the test setup. There are several reasons why an experimental assessment is required:

1. To increase the engineering understanding of the wall behaviour; especially in the nonlinear range.
2. To list possible improvements to the analytical model, which could represent the wall behaviour in its elastic and non-linear range.
3. To give an initial insight into the validity of the behaviour factor $R$ and time period $T$; these values have a significant influence on the results and are expected to be significantly different from the values used for other construction typologies in the Indian Building code.
The failure of the wall is governed by internal rotation (Figure 8.3b-e) in the layer and the wall shows a high ductility. During the building of the wall, two of the twenty kadil connections in the third layer were accidentally left out. This led to a final failure in this layer, hence the missing kadils triggered other possible failure mechanisms.

![Force-displacement curve](image)

Figure 8.3: Force-displacement curve (a) and 3rd layer failure (b-e)

The analytical in-plane loaded wall model used is validated using the experimental results. The analytical model is scaled to correspond with the dimensions of the tested wall and, instead of earlier used design material properties, mean material properties are adopted. The in-plane loaded wall validation model gave a force capacity of 42.0 kN for a wall of 5 layers and 2995 mm long, which matches the experimental bi-linear maximum force capacity of 46.5 kN with an accuracy of approximately 10%, suggesting that the analytical in-plane wall model is reasonably able to approximate the actual wall behaviour. The difference in capacity among the different evaluated failure mechanisms is not more than 4 kN, which is approximately 10 % of the total wall capacity. The deviation of the material properties of timber is high, which makes the finally activated failure mechanism in the experimental wall highly dependent on local material imperfections and strength reductions.

![Expected failure](image)

Figure 8.4: Expected failure in 3rd layer calculated by analytical in-plane loaded wall model

Both experiments and validation model gave an insight into the important seismic features of Kath-Kuni buildings, which altogether generate high ductility and robustness:

1. Ductile kadil connections transfer shear force and allow for internal layer rotation
2. Maanwi’s keep the perpendicular beams together
3. Both connections are active in parallel and series, which makes the structure robust
4. Relative uplift between the layers is possible, without creating permanent damage
5. There is a high contribution of the frictional resistance in the wall. Important to notice is that for the analytical model, only the timber material properties are considered to be relevant and for the stone the friction is the only contribution characteristic. The timber strength is used to its full capacity as compression perpendicular to the grain in the *kadil* connections and for the rotating blocks due to the precompression force. For the *maanwi* connection the tensile strength of the timber perpendicular to the grain was essential (in the embedment of the connection). These mechanisms are all occurring in the corners and the beams in the span of the wall are not prone to failure. These beams are subjected to bending solely when the perpendicular beams in the corners start to rotate, and are restricted in the amount of deflection. It can be concluded that the bending behaviour of the long timber beams is mainly relevant in the out-of-plane loading direction.

After analysing the main force displacement diagram of the wall (Figure 8.3a), a behaviour factor $R$ is derived to be 5.7, which is 2.3 times higher than the value used in the initial assessment. The time period $T$ is established to be a minimum of 0.48 s for the most critical walls, which gives a value outside of the plateau of the elastic response spectrum. A revised maximum allowable peak ground acceleration of 0.19 g is found, which is a factor 3.2 higher than the initial established maximum allowable peak ground acceleration of 0.06 g. This is a significant improvement and proves that the standard values obtained from IS 1893 (Part 1) are not representative for Kath-Kuni structures. However, without the elaborate analytical model, the experimental wall results can actually not be scaled and the re-evaluated behaviour factor $R$ and time period $T$ are only rough estimations for the values that could be used for the total building. However, lower bound assumptions are taken to at least show the validity of the parameter and generate an incentive for further research to the actual values of these parameters. Hence, the equivalent lateral force method is used only as an initial seismic evaluation tool; it is recommended in future research to explore non-linear and dynamic analysis methods. Nevertheless, the calculated maximum allowable peak ground acceleration a conventional concrete structure can resist is just 0.03 g (Singh, Lang, & Narasimha, 2015), which is a factor 6 times lower than the resistance now calculated for Kath-Kuni structures. The concrete structure assessed is designed for purely vertical gravity loads as nowadays is often being built in the Himalayas. The final results can be found in figure 8.5.
8.4. Sustainability

Despite the proven seismic performance of Kath-Kuni structures, this building style seems to be no longer applied and is gradually being replaced with concrete structures. The benefits and disadvantages related to the sustainability of the Kath-Kuni building style are qualitatively investigated by means of literature survey and interviews with local people and categorised under the parameters of performance, service life and environmental impact of the building. The total sustainability of the Kath-Kuni building typology is qualitatively compared with concrete construction using formula (8.1). The indicator in 8.1 is including performance, service life and environmental impact (Mueller, Haist, Moffatt, & Vogel, 2017).

\[
\text{Building Material Sustainability Potential (BMSP)} \sim \frac{\text{Service Life} \cdot \text{Performance}}{\text{Environmental Impact}} \quad (8.1)
\]

Performance

It can be seen from empirical evidence that the seismic performance of a Kath-Kuni structure is significantly better than the current concrete and brick building practices frequently observed in the Himalayas. It should be noted that a well-designed Kath-Kuni structure is compared here to a badly designed concrete structure. However, this is unfortunately the reality in the choices people make and awareness should be created regarding the risks that they are taking. From interviews with local people there was an absolute consensus in the different regions that the Kath-Kuni buildings are much more comfortable in terms of indoor climate all year. However, local people feel that maintenance of Kath-Kuni buildings is a difficult task, as the building needs to be washed at least twice a year with water to keep the material from deteriorating. People would like a change in the general design of Kath-Kuni buildings, where the layout of the functional spaces is such that the bathrooms are separated (but still on the same floor level as the living area), the kitchen is on the ground floor to avoid carrying of water and firewood. Cleaning of the “modern” concrete buildings is considered to be easier, due to the smooth surfaces of the concrete floors. Despite the need for modernisation, local people show a general fondness of their vernacular architecture and the building style is important for the cultural identity of the community. Traditionally, Kath-Kuni used to strengthen the community bonds since the structures were built by local people together. The knowledge was passed from generation to generation, materials needed to be divided among families and construction work was heavy so workload needed to be shared. Local knowledge and skills also mean an enhanced local livelihood. Nowadays, well maintained traditional architecture is an important factor for enhancing tourism, which leads to an increase in the employment opportunities in the region.

However, the government of India now regulates tree felling and natural stone mining strictly, as a consequence of lots of illegal and non-regulated deforestation in the past. This resulted in a significant increase of the costs of the main materials used in the Kath-Kuni building style. Furthermore, the building speed and labour costs are disadvantageous over the relatively cheap and fast concrete construction.

Life time

Kath-Kuni: at least 200 years, evidence found for Kath-Kuni structures older than 800 years.
Concrete: designed for 50 years.

Environmental impact

The fact that Kath-Kuni is making use of completely local, reusable and biodegradable materials makes that the expected emissions of a Kath-Kuni building during the design, construction and end-of-life phase are very low. Especially compared to concrete buildings, where cement is obtained from the planes and known for its environmental impact. During the service life of the buildings, emissions due to improper insulation seem to be
more severe in concrete buildings, as the thermal comfort of the Kath-Kuni structures is experienced to be significantly better than the concrete counterpart.

The arguments above led to the conclusion that the Kath-Kuni building style scores well on sustainability. The modernisation and maintenance issues described above are relatively easy to solve with a professionally performed architectural redesign, which the local communities are so far not able to perform themselves. However, high construction costs, due to a scarcity of the main materials (and the restrictions set by the government in obtaining these materials), has rendered building of Kath-Kuni houses currently economically unviable.

All-in-all, the vernacular Kath-Kuni architecture shows great potential in the earthquake resistance, where it makes use of non-standard techniques, resulting in a high ductility and robustness of the building. Moreover, the buildings seem to score well on sustainability criteria and could be used in generating livelihood for the local community in terms of local craftsmanship and tourism. It is important that these traditional techniques do not get lost, despite the current lack of the mainly used Deodar timber. Recommendations on how to possibly revive Kath-Kuni construction in the Indian Himalaya can be found in the chapter 9.
9. RECOMMENDATIONS

Even though Kath-Kuni structures have been built over the last centuries, the technical knowledge is slowly vanishing. The field trips showed how local carpenters are incorporating techniques from the concrete construction sector, which are not always beneficial. It is recommended to continue the documentation of these traditional structures and their variation over different regions. These changes in configurations are recommended to be studied more intensively by use of experimental testing and analytical modelling.

The timber properties used as input values in the analytical models are obtained from the Indian timber building code IS 883:1994, but are very uncertain. Testing should be performed to the timber used for the experimental specimen.

As explained in paragraph 1.5, buildings can be calculated by making use of design calculations according to EN 1998-1, usually to calculate the seismic capacity of new buildings, and by assessment calculations according to 1998-3, usually to calculate the seismic capacity of existing buildings. In this thesis, it is chosen to perform the design calculations and compare those to demand peak ground accelerations. It is recommended to perform the calculations according to 1998-3 for existing buildings.

The out-of-plane loaded wall model can be improved by testing the *kadil* and *maanwi* connection stiffness and force capacity and implement the results as force-displacement graphs in the model. The wall is assessed for its capacity to resist forces induced by local natural frequency vibrations, but not for its capacity when subjected to forces induced by movement of the total building. For a full assessment, this aspect should be taken into consideration.

To improve the in-plane loaded wall model, similar to the out-of-plane analytical wall model, first of all the analytical models for the *kadil* and *maanwi* connection need to be improved and incorporated. These models should be able to give a full elastic and plastic strength and stiffness capacity and ductility of the connection. Ideally tests should be performed for these individual connections, to validate the models. The Johansen model now taken for the *kadil* connections is not representative for timber dowel connections subjected mainly to rotations. Friction must be incorporated as static and kinetic friction and the values should be obtained from tests. Moreover, the model should incorporate global wall behaviour and the virtual work has to be calculated for each incremental displacement step (possible improvements are elaborated in paragraph 5.3.8). An improved analytical model will allow for upscaling to full scale wall dimensions. A parameter study should be performed to analyse the influence of the different lateral force and stiffness contributing elements.

The perpendicular rotating beams in the layer failure are in fact the long beams in the out-of-plane loaded walls. In reality, the directions of the seismic accelerations are not restricted to one direction only. The walls are subjected to forces in both in-plane and out-of-plane direction at the same time. So far, only monotonically loaded quasi-static in-plane wall experiments are performed. Overall, full non-linear dynamic analysis is needed for the total building, to evaluate the capacity of Kath-Kuni structures to its full potential. Full shake table tests will show the contribution of box-action and connectivity of the out-of-plane and in-plane loaded walls and how
the walls are loaded in multiple directions at the same time. A three dimensional approach could clarify the amount of seismic mass now assumed needed to be resisted by the in-plane loaded walls. Now, all seismic mass is assumed to be resisted by the in-plane loaded walls, whereas for Kath-Kuni structures it is likely that the out-of-plane walls also contribute to the lateral resistance of the total building by transferring some of the forces directly to the ground by friction.

The different construction techniques of the different storeys (dry stone masonry and full timber construction) in the building should be taken into account. As the building is non-homogeneous, a multi-degree-of-freedom approach is required incorporating these different stiffnesses.

Timber and natural stone are nowadays becoming more scarce. This makes Kath-Kuni construction in its original form less feasible. However, the benefits of this traditional building technique deserve to be recognised and alternative innovative materials can be applied to continue this vernacular architecture in practice. Therefore, it is recommended to investigate possibilities in modernising and re-interpreting the Kath-Kuni building style using other sustainable and affordable materials, such as bamboo, stabilized earthen blocks or lime concrete. Not only the material properties of these materials are important to investigate, but also their durability and maintenance in order to immediately tackle the other disadvantageous performance criteria. Maintenance convenience also need to be taken into account in the architectural re-design and thought should be given to improve the construction process of this building style significantly. Even though earthquake resistant construction is also feasible using concrete as main material, it is the vernacular architecture and its embedded traditional knowledge that gives character to the Himalayas, which is beneficial for tourism and livelihood. Moreover, the concrete construction is a big contributor to the total global CO2 emissions; hence using local, sustainable and durable materials should be promoted. It is recommended in future research to perform a lifecycle assessment to compare the improved Kath-Kuni building technology with current concrete practices in terms of environmental impact.
References


Standards


EN 384 (2016). Structural timber - Determination of characteristic values of mechanical properties and density. Brussels. CEN.


APPENDICES
### A. IS 1893 (Part 1) vs. EN 1998-1

Table A.1: Comparison EN 1998-1 and IS 1893 (Part 1)

<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Demand</strong></td>
<td></td>
</tr>
</tbody>
</table>

Demand is given as zone factors \( Z \) in the following manner:
- **MCE** = Maximum Considered Earthquake
- **DBE** = Design Basis Earthquake

Where a factor 2 is used between DBE and MCE.

Zone factor \( Z \) is obtained from a seismic hazard map from IS 1893 (Part 1).

This approach received criticism (Jain, 2003), because the PGA given in the Indian Seismic design code are not related to certain probability RPs, which makes the \( Z \)-factor just a factor.

For Limit State load factors for load combinations for seismic design are assumed for concrete construction:

**5.3.3.1.2 Partial safety factors for limit state design of reinforced concrete and prestressed concrete structures**

In the limit state design of reinforced and prestressed concrete structures, the following load combinations shall be accounted for:

- a) \( 1.5 \ (DL \ + \ IL) \)
- b) \( 1.2 \ (DL + IL \ + \ EL) \)
- c) \( 1.5 \ (DL \ + \ EL) \)
- d) \( 0.9 \ DL \ + \ 1.5 \ EL \)

Where, \( DL \) is Dead Load, \( IL \) is Imposed Load and \( EL \) is the Earthquake Load.

For concrete characteristic material properties are used in the and multiplied with a partial factor of safety.

For timber working stress material properties are used and in a working stress approach no load factor is applied. To prevent overestimated design, the seismic demand values are reduced and proposed as being design zone factors \( Z \) instead of PGAs with a certain return period.

In this thesis the following is assumed*:

- Near Collapse Limit State \( PGA = 1.5 \times \) MCE Z-factor
- Significant Damage Limit State \( PGA = 1.5 \times \) DBE Z-factor

*The 1.5 is derived from the concrete partial safety factor applied on EL

The PGA \( (a_g) \) related to probability in following Limit States:
- **Near Collapse:** 2 percent probability of being exceeded in 50 years (2475 year RP)
- **Significant Damage:** 10 percent probability of being exceeded in 50 years (475 year RP)

\( a_g \) is obtained from seismic hazard maps from the respective National Annexes from EN 1998-1

The Limit State actions are computed as follows (EN 1990:2002):

**6.4.3.4 Combinations of actions for seismic design situations**

(1) The general format of effects of actions should be:

\[
\begin{align*}
F_d & = F \left[ \gamma_{2k} ; \gamma_{1k} , A_\text{Ed} , \psi_{2r} A_{Ak} \right], \\
& \quad f \geq 1; i \geq 1
\end{align*}
\]

(2) The combination of actions in brackets \{ \} can be expressed as:

\[
\sum \gamma_i G_i = \gamma_{1r} \gamma_{2r} A_{Ed} \psi_{2r} \sum \gamma_{2j} A_{Ak}
\]

Where, \( G \) is the permanent load, \( P \) the prestress (not applicable), \( A \) is the seismic loading and \( Q \) the accompanying variable actions and \( \psi_2 \) quasi-permanent value of a variable action

In this formula the seismic design action is defined as follows:

\[
A_{Ed} = \gamma_{1r} A_{Ek}
\]

Where the partial safety factor \( \gamma_1 = 1 \).

---

*The special publication no. 7 of the National Building Code of India is used as reference for the tables and pictures in Table A.1 (SP7, 2005)
Elastic spectrum and behaviour factor

Horizontal elastic design response spectrum

Curve consists of three components or ranges of \( T \) (time period)

Spectrum definition:

- **For Rocky, or hard soil sites**
  \[
  \frac{S}{g} = \begin{cases} 
  1+1.5 \frac{T}{T_0}; & 0.00 \leq T \leq 0.10 \\ 
  2.50; & 0.10 \leq T \leq 0.40 \\ 
  1.00/ \frac{T}{T_0}; & 0.40 \leq T \leq 4.00 
  \end{cases}
  \]

- **For Medium soil sites**
  \[
  \frac{S}{g} = \begin{cases} 
  1+1.5 \frac{T}{T_0}; & 0.00 \leq T \leq 0.10 \\ 
  2.50; & 0.10 \leq T \leq 0.55 \\ 
  1.36/ \frac{T}{T_0}; & 0.55 \leq T \leq 4.00 
  \end{cases}
  \]

- **For Soft soil sites**
  \[
  \frac{S}{g} = \begin{cases} 
  1+1.5 \frac{T}{T_0}; & 0.00 \leq T \leq 0.10 \\ 
  2.50; & 0.10 \leq T \leq 0.67 \\ 
  1.67/ \frac{T}{T_0}; & 0.67 \leq T \leq 4.00 
  \end{cases}
  \]

Design approach: spectrum can be reduced by using behaviour factor \( R \) (implemented in ELF method formula below)

Time period \( T \) can be estimated as follows:

\[
T_a = \frac{0.09}{\sqrt{d}} 
\]

where:

- \( h \) = Height of building, in metres, as defined in 5.4.6.1; and
- \( d \) = Base dimension of the building at the plinth level, in metres; and along the considered direction of the lateral force.

Multiplying factors for the damping are:

**Table 32 Multiplying Factors for Obtaining Values for Other Damping**

<table>
<thead>
<tr>
<th>Damping percent</th>
<th>Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>3.20</td>
</tr>
<tr>
<td>2</td>
<td>1.40</td>
</tr>
<tr>
<td>5</td>
<td>1.00</td>
</tr>
<tr>
<td>7</td>
<td>0.90</td>
</tr>
<tr>
<td>10</td>
<td>0.80</td>
</tr>
<tr>
<td>15</td>
<td>0.70</td>
</tr>
<tr>
<td>20</td>
<td>0.60</td>
</tr>
<tr>
<td>25</td>
<td>0.55</td>
</tr>
<tr>
<td>30</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Horizontal elastic response spectrum

Curve consists of four components or ranges of \( T \) (time period)

Spectrum definition:

\[
T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \\
T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \left[ \frac{2T}{T_C} \right] \\
T_D \leq T \leq 4S : S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5 \left[ \frac{T_C \cdot T_D}{T^2} \right]
\]

Where, \( a_g \) is the design ground acceleration on type A ground, \( S \) is the soil factor and \( \eta \) is the damping correction factor with a reference value of \( \eta = 1 \) for 5% viscous damping.

Here the PGA is incorporated in the \( S_e(T) \) value. In the Indian code, the PGA is incorporated as \( Z \)-value during the ELF method.

Design approach: spectrum can be reduced by using behaviour factor \( Q \).

Time period \( T \) can be estimated as follows:

\[
T_i = C_i \cdot H^{1.4} 
\]

where:

- \( C_i \) is 0.85 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.045 for all other structures;
- \( H \) is the height of the building, in m, from the foundation or from the top of a rigid basement.

The damping parameter in the response spectrum can be adapted as follows:

\[
\eta = \sqrt{10(5 + \xi)} \geq 0.55
\]

where \( \xi \) is the viscous damping ratio of the structure, expressed as a percentage.
ELF method

\[ F_b = \frac{Z}{R} \frac{I_S}{g} W \]

Where,
- \( S_a/g \) = spectral acceleration coefficient
- \( Z \) = Zone factor, is for the maximum considered earthquake (MCE) and service life of structure in a zone.
- \( I \) = importance factor
- \( R \) = behaviour factor
- \( W \) = seismic mass

Extrem short earthquake loading allows increase in permissible stress in materials of 1/3th!

Seismic mass

Seismic mass used to calculate the inertia effects on the structure is taken as:

\[ W = 100\% \times DL + 25\% \times IL \]

Seismic mass used to calculate the inertia effects on the structure is taken as:

Table 37: Percentage of Imposed Load to be Considered in Seismic Weight Calculation

<table>
<thead>
<tr>
<th>Imposed Uniformly Distributed Floor Loads (kN/m²)</th>
<th>Percentage of Imposed Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to and including 3.0</td>
<td>25</td>
</tr>
<tr>
<td>Above 3.0</td>
<td>50</td>
</tr>
</tbody>
</table>

3.2.4 Combinations of the seismic action with other actions

\[ \sum C_{k,j} \cdot \sum \psi_{E,i} \cdot Q_{k,i} \]

\[ \psi_{E,i} = \varphi \cdot \psi_{2i} \]

Table 42: Values of \( \varphi \) for calculating \( \psi_{2i} \)

<table>
<thead>
<tr>
<th>Type of variable action</th>
<th>Story</th>
<th>( \varphi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category A-C (^\dagger)</td>
<td>Roof</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Storeys with correlated occupancies</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>Independently occupied storeys</td>
<td>0.5</td>
</tr>
</tbody>
</table>

\( \psi_2 = 0.3 \) for category A buildings (domestic, residential areas) (EN 1990:2002, table A1.1)

Lateral force distribution

Based on a quadratic mode shape:

\[ Q_i = V_y \frac{W_i \cdot h_i^2}{\sum W_j \cdot h_j^2} \]

where
- \( Q_i \) = Design lateral force at floor \( i \)
- \( W_i \) = Seismic weight of floor \( i \)
- \( h_i \) = Height of floor \( i \) measured from base, and
- \( n \) = Number of storeys in the building is the number of levels at which the masses are located.

Based on the actual mode shape of the building:

\[ F_i = \frac{F_s \cdot \sum m_j}{\sum x_j \cdot m_j} \]

where
- \( F_i \) is the horizontal force acting on storey \( i \);
- \( F_s \) is the seismic base shear in accordance with expression (4.5);
- \( x_j, m_j \) are the displacements of masses \( m_n, m_i \) in the fundamental mode shape;
- \( m_n, m_i \) are the storey masses computed in accordance with 3.2.4\( \dagger \).

Or linear over the height of the building in case the displacements are increasing linearly over the height.
B. Sketch-Up comparison
C. Timber characteristics derivation

![Image of derivation diagram]

Figure C.1: Derivation of permissible stress according to the Handbook on Timber Engineering (SP33, 1986)

- Fundamental Stresses (Ultimate Stresses) (Derived from small-clear specimen on statistical consideration)
- Divided by factor of ignorance $F_a = \frac{1}{f_i \cdot f_r \cdot f_t}$
- Divided by total factor of safety $F = \frac{1}{f_i \cdot f_s \cdot f_t}$
- Divided by factor of grade and locations $F_b = \frac{1}{f_i \cdot f_t}$
- SAFE WORKING STRESSES (or simply 'working stresses' or permissible stresses as reported in IS : 883-1970)
- DIVIDE BY SPECIFIC DESIGN FACTORS $F_i = \frac{1}{f_i}$, etc.

where
- $f_i =$ factor due to inherent variability in the species
- $f_i =$ factor due to accidental loading
- $f_s =$ factor due to long time loading
- $f_r =$ factor for grade of the material
- $f_b =$ factor for location of use; and
- $f_e =$ factors for specific design, such as form factor, etc.
D. Weight calculation

Roof
The roof of the Kath-Kuni house is built in three sections. These three sections are the three different cuboids out of which the building is built up. The cuboids are consisting of thick Kath-Kuni load bearing walls. In the roof an additional Kath-Kuni wall is used as intermediate support. The slate roof is carried by horizontal timber beams (q_roof), which are spanning between the Kath-Kuni walls (F_kkwall). Because the beams are continuous beams, most of the weight will go to the intermediate supporting Kath-Kuni wall. Point loads that are bearing to the Kath-Kuni walls are considered to alter into distributed loads in the wall (q_kkwall). On the lower roof parts, which are protecting the balconies, the slate is similarly supported by timber beams (q_roof). It was not clear if those beams are continuous, so simply supported beams are assumed, although this might be too conservative.

4th storey (attic)
The fourth storey (the attic) consists of three floor beams spanning over the longitudinal Kath-Kuni walls, with two cantilevering parts on each side. The cantilever parts support the lower roof. The beams close to the walls carry half the weight of the beams in the middle of the floors. The load from the lower part of the roof results into four point loads (F_roof_beams) on the cantilever ends of the long beams. The intermediate roof Kath-Kuni walls are supported by the middle floor beam, which transfers the load to the longitudinal Kath-Kuni walls (q_floor + kk_separation_wall_roof). This beam is subjected to high loads as the weight of the intermediate Kath-Kuni wall is resting on it. Furthermore, due to the continuous beams, a big part of the load of the roof is resting on this beam. Similar to the floor beams, 4 point loads of the lower roof are bearing on the cantilever beams on the transverse side and on the diagonal beams (F_roof_beams). The support reactions of all beams resting on the Kath-Kuni walls (F_kkwall) are transferred into a distributed load (q_kkwall). Except for the corners, where the loads are considered as point loads (F_corner). Last, the distributed load from the roof is added to calculated distributed load (q_kkwall).
3\textsuperscript{rd} storey
The three floor beams are spanning between the two longitudinal Kath-Kuni beams. On these beams, a floor load is resting ($q_{\text{floor}}$) and half this floor load is resting on the side beams. The cantilever beams carry the load from the balcony floor ($q_{\text{balcony\_floor}}$) as well as the load from the timber balcony façade ($F_{\text{balcony\_wall}}$). The support reactions of all beams resting on the Kath-Khuni walls ($F_{kkwall}$) are altered into a distributed load ($q_{kkwall}$). Except for the corners, where the loads are considered as point loads ($F_{\text{corner}}$). Last, the distributed load from the storeys above, plus the self-weight of the wall is added to the calculated distributed load ($q_{kkwall}$).

![Diagram](image1.png)

2\textsuperscript{nd} storey
The second floor has again 3 simply supported timber beams spanning between the Kath-Kuni walls. On these beams, a floor load is resting ($q_{\text{floor}}$) and half this floor load is resting on the side beams. The cantilever beams are carrying the load from the balcony floor ($q_{\text{balcony\_floor}}$) and from the timber balcony façade ($F_{\text{balcony\_wall}}$). The support reactions of all beams resting on the Kath-Khuni walls ($F_{kkwall}$) are transferred into a distributed load ($q_{kkwall}$). Except for the corner, where the load is considered as point load ($F_{\text{corner}}$). Last, the distributed load from the storeys above, plus the self-weight of the wall is added to the calculated distributed load ($q_{kkwall}$).

![Diagram](image2.png)

1\textsuperscript{st} storey
On the first storey, the only load that is applied is that of the inside floors. The floors are resting on the three floor beams, similarly like the floors above. The support reactions of all beams resting on the Kath-Kuni walls ($F_{kkwall}$) are transferred into a distributed load ($q_{kkwall}$). Last, the distributed load from the storeys above, plus the self-weight of the wall is added to the calculated distributed load ($q_{kkwall}$).

![Diagram](image3.png)
Foundation

On foundation level the resulting forces are obtained and can be seen in the figure below. And a table is given with the final loads on the building at base level.

<table>
<thead>
<tr>
<th>нагрузка</th>
<th>значение</th>
<th>единица</th>
<th>значение</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL DEAD LOAD:</td>
<td>4605.3 kN</td>
<td></td>
<td>460.5 ton</td>
</tr>
<tr>
<td>length l [m]</td>
<td>14.8</td>
<td></td>
<td>14.8</td>
</tr>
<tr>
<td>weight w [m]</td>
<td>4.8</td>
<td></td>
<td>4.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>нагрузка</th>
<th>значение</th>
<th>единица</th>
<th>значение</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL LIVE LOAD: (WITH FLOOR REDUCTION)</td>
<td>854.4 kN</td>
<td></td>
<td>85.4 ton</td>
</tr>
<tr>
<td>length l [m]</td>
<td>14.8</td>
<td></td>
<td>14.8</td>
</tr>
<tr>
<td>weight w [m]</td>
<td>4.8</td>
<td></td>
<td>4.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>нагрузка</th>
<th>значение</th>
<th>единица</th>
<th>значение</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL LIVE LOAD SEISMIC CALCULATION: (WITHOUT FLOOR REDUCTION)</td>
<td>722.4 kN</td>
<td></td>
<td>72.2 ton</td>
</tr>
<tr>
<td>length l [m]</td>
<td>14.8</td>
<td></td>
<td>14.8</td>
</tr>
<tr>
<td>weight w [m]</td>
<td>4.8</td>
<td></td>
<td>4.8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>нагрузка</th>
<th>значение</th>
<th>единица</th>
<th>значение</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL COMBINED LOAD: (WITH FLOOR REDUCTION)</td>
<td>5459.7 kN</td>
<td></td>
<td>546.0 ton</td>
</tr>
<tr>
<td>( \gamma_P ) [ULS]</td>
<td>1</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>( \gamma_V ) [ULS]</td>
<td>0.25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>нагрузка</th>
<th>значение</th>
<th>единица</th>
<th>значение</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOTAL SEISMIC LOAD: (WITHOUT FLOOR REDUCTION)</td>
<td>4785.9 kN</td>
<td></td>
<td>478.6 ton</td>
</tr>
<tr>
<td>( \gamma_P ) [SEISMIC]</td>
<td>1</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>( \gamma_V ) [SEISMIC]</td>
<td>0.25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
E. Out-of-plane loaded wall

\[ y = \psi(x) = \text{shape function of mid displacement} \]
\[ u = \text{virtual deflection} \]
\[ \theta = \psi'(x) = \text{rotation} \]
\[ \kappa = \psi''(x) = \text{curvature} \]

**Moment:**
\[ M = EI \psi''(x) \]

**Internal virtual work:**
\[ \int EI \psi''(x) \psi''(x) \ du \ dx = \int \psi''(x)^2 \ dx \]

**External virtual work:**
\[ F \cdot \Delta u = k_{II} \cdot u \cdot \Delta u \]
(\( u = 1 \) at location of force, so \( F = k_{II} \))

**Stiffness:**
\[ k_{II} = \int EI (\psi''(x))^2 \ dx \]

Order of shape function to cover 4 degrees of freedom = 4.

\[ \psi(x) = ax^3 + bx^2 + cx + d \quad c, d = 0 \]
\[ \psi'(x) = 3ax^2 + 2bx \]
\[ \psi''(x) = 3ax + b \]
\[ \psi''(1/x) = 3a + b = 0 \]

Order of shape function to cover 4 degrees of freedom = 4.

\[ \psi(x) = \frac{1}{3} \delta x^3 - \frac{x}{16} a x^3 = 1 \]
\[ \frac{1}{16} a x^3 = 1 \]
\[ \delta = \frac{16}{x^3}, \quad b = \frac{12}{x^3} \]

\[ \psi'(x) = \frac{16}{x^2^2} + \frac{12}{x^2} x \]

\[ \psi''(x) = -\frac{6}{x^3^2} + \frac{24}{x^2} x \]

\[ \psi''''(x) = \left( \frac{6}{x} \right)^2 - \left( \frac{6}{x}^2 \right) \]

\[ k_{II} = 2EI \int \frac{926}{x^6} - \frac{46008}{x^5} + \frac{576}{x^4} \ dx = \frac{192EI}{x^2} \]
STIFFNESS PINNED

Order of shape function to cover 4 degrees of freedom = 4
\[ \psi(x) = ax^4 + bx^3 + cx^2 + dx + e \]
\[ a = 1 \quad b = 0 \quad c = 0 \quad d = 0 \quad E1 \psi''(x) = 0 \]
\[ \psi(x) = 2ax^2 + 2bx + c \]
\[ a = 0 \quad b = 0 \quad c = 0 \quad \psi(x) = 2x \]
\[ \psi''(x) = 6ax + 2b \]
\[ a = 0 \quad b = 0 \quad \psi''(x) = 0 \]
\[ \psi''(x) = 3a(4x^2 + c) = 0 \]
\[ a = \frac{3}{4} \quad c = \frac{3}{4} \quad \psi''(x) = \frac{3}{4} \quad x \]
\[ \psi(x) = 2a(\frac{2}{5}x^3) - \frac{3}{4}x^2 \quad \frac{2}{5}x^3 = 1 \]
\[ = \frac{1}{4}x^3 - \frac{3}{8}x^2 \quad \frac{1}{4}x^3 = 0 \]
\[ = \frac{1}{4}x^2 \quad x = \frac{3}{8} \quad a = -\frac{4}{5} \quad c = \frac{3}{4} \]
\[ \psi(x) = 4x^3 + \frac{2}{3}x \]
\[ \psi''(x) = -\frac{2a}{x^2} \quad \psi''(x) = \frac{24x^2}{x^2} \]
\[ k = 2E1 \int_{0}^{1} \frac{24x^2}{x^2} \, dx = \frac{10E1}{5} \]

CONSISTENT MASS
\[ \psi(x)^2 = 16x^4 - \frac{24x^4}{x^2} + \frac{9x^4}{x^2} \]
\[ \bar{m}(x) = 2 \int \frac{1}{2} \psi(x)^2 \, dx \approx 0.986 \bar{m} \]
**CONSISTENT MASS**

(3rd order shape function)

\[
\begin{align*}
\psi(x) \cdot u &= u(x) \\
\bar{f}(x) &= \bar{m}(x) \ddot{u}(x) \\
\bar{m}(x) &= \bar{m}(x) \psi(x) \ddot{u} \\
\delta u &= \text{virtual deflection} \\
\end{align*}
\]

Internal virtual work:
\[
\int \bar{m}(x) \psi(x) \ddot{u} \cdot \psi(x) \delta u \, dx
\]

External virtual work:
\[
F \cdot \delta u = m_u \ddot{u} \delta u
\]

\[
\begin{align*}
\psi(x) &= -\frac{15}{8} x^2 + \frac{12}{8} x^2 \\
\psi(x)^2 &= \frac{255 x^6}{8^6} - \frac{324 x^5}{8^5} + \frac{144 x^4}{8^4} \\
m_u &= 2 \int \bar{m}(x) \psi(x)^2 \, dx = 2 \bar{m} \left[ \frac{255 x^6}{8^6} - \frac{324 x^5}{8^5} + \frac{144 x^4}{8^4} \right]_{x=L}^{x=0}
\end{align*}
\]

\[
m_u = \frac{13 \bar{m}^2}{85} \approx 0.37 \bar{m} L
\]

**LUMPED MASS**

(1st order shape function)

\[
m = 0.5 \bar{m} L
\]

\[\rho_{\text{timber}} = 5.6 \text{ kN/m}^2, \quad \rho_{\text{stone}} = 25.9 \text{ kN/m}^2, \quad \rho_{\text{rubble}} = 20.4 \text{ kN/m}^2\]

\[
\bar{m} = 2.100 \cdot 175 \cdot 5.6 \cdot 10^{-6} + (460 - 2) \cdot 175 \cdot 294 \cdot 10^{-6} + 200 \cdot 460 \cdot 25 \cdot 10^{-6}
\]

\[
= 4.6 \text{ N/mm}
\]
F. Global in-plane loaded wall calculation

Full scale wall with 7 layers:
\[ \sigma_{c,90,d} = 2.2 \text{ N/mm}^2 \]
\[ F = 144000 \text{ N} \]
\[ W = 123493 \text{ N} \]
\[ l_{1,3} = 4800 \text{ mm} \]
\[ l_2 = 5400 \text{ mm} \]
\[ l_4 = 4600 \text{ mm} \]
\[ h = 2953 \text{ mm} \]
\[ h_l = 0 \text{ mm (point of load application)} \]
\[ w = 460 \text{ mm} \]
\[ \mu = 0.35 \text{ mm} \]

Experimental wall with 5 layers:
\[ \sigma_{c,90,m} = 2.8 \text{ N/mm}^2 \]
\[ F = 92306 \text{ N} \]
\[ W = 55056 \text{ N} \]
\[ l = 2995 \text{ mm} \]
\[ h = 2110 \text{ mm} \]
\[ h_l = 220 \text{ mm (point of load application)} \]
\[ w = 460 \text{ mm} \]
\[ \mu = 0.35 \text{ mm} \]

**Bottom sliding:**
\[ V_{st} = \mu \cdot (F + W) \]
7 layers: 93.6 kN 51.6 kN

**Rocking:**
\[ V_r = \frac{1}{2} l \cdot (F + W) \]
\[ \frac{h + h_l}{h + h_l} \]
217.4 kN 94.7 kN

**Toe crushing:**
\[ x = \frac{2 \cdot (F + W)}{\sigma_{c,90} \cdot w} \]
528.6 mm 228.8 mm
\[ V_t = \frac{\left( \frac{1}{2} l - \frac{1}{3} x_{\min} \right) \cdot (F + W)}{h + h_l} \]
201.4 kN 89.9 kN
The in-plane wall is assumed to be the main lateral resisting element of the building. In this paragraph the maximum internal lateral force capacity of a Kath-Kuni in-plane loaded wall is calculated. In other words: from what lateral force does the wall turn into a mechanism? In order to assess this, different possible failure mechanisms for an in-plane wall layer are modelled, where the mechanism with the lowest capacity is the expected failure mechanism. The terminology of the components of an in-plane wall layer are given in figure 5.12. This in-plane loaded wall model is used to calculate the capacity of a full scale wall with 7 layers and an experimental wall with 5 layers. The material input parameters and geometrical details differ between the two cases.

Below an overview of the possible activated forces by the different mechanisms is shown:

Assumed is that the failure of the wall is caused by mechanisms in series, the maximum force is reached in stage 4 and this stage is assessed in this analysis.

<table>
<thead>
<tr>
<th>Stage 1</th>
<th>Stage 2</th>
<th>Stage 3</th>
<th>Stage 4</th>
<th>Stage 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static friction capacity</td>
<td>Static friction capacity</td>
<td>$\alpha$ * static friction capacity</td>
<td>$\alpha$ * static friction capacity</td>
<td>Post failure behaviour</td>
</tr>
<tr>
<td>Elastic kadil connection capacity</td>
<td>Plastic kadil connection capacity</td>
<td>Plastic kadil connection capacity</td>
<td>Plastic kadil connection capacity</td>
<td></td>
</tr>
<tr>
<td>Perpendicular beam plastic rotational resistance</td>
<td></td>
<td>Perpendicular beam plastic rotational resistance</td>
<td>Maanwi elastic tensile capacity (brittle)</td>
<td></td>
</tr>
</tbody>
</table>

First the wall will have a linear elastic behaviour and the static friction capacity is not exceeded. The wall will deform elastically. Small plastic deformations will start and the perpendicular beams in the layer (and the longitudinal infill piece) start to rotate. It is assumed that the kadils and perpendicular beam rotational resistance first reach their plastic capacity. The already large deformation in stage 4 activates the maanwi’s. The maanwi’s contributes with its elastic virtual work, but only if the connected beams are both rotating with a different angle. After the maximum capacity is reached the maanwi’s will fail brittle and post-failure behaviour
will generate more deformation in the wall, which is not evaluated in this thesis. This model does calculate the deformations in the wall and is designed purely to calculate the maximum force capacity of an in-plane loaded wall layer.

**Symbols:**

\( \delta u \) = virtual displacement of layer relative to layer below  
\( h_s \) = height stone/concrete layer  
\( h_b \) = height beams  
\( h_i \) = height infill piece  
\( h_m \) = height maanwi location from bottom longitudinal beam  
\( x_n \) = maximum eccentricity of pre-compression force on beam no. \( n \)  
\( x_{n,l} \) = maximum eccentricity of pre-compression force on long infill piece at location \( n \)  
\( F_{v,n,a} \) = vertical design force on the beams and infill pieces  
\( M_{k,a,d} \) = kadil design moment capacity for \( a \) is option 1, 2 and 3  
\( F_{m,t,d} \) = design tensile capacity of the maanwi at the height of the wall  
\( F_{fr} \) = lateral friction force capacity  
\( V_h \) = horizontal lateral force capacity of an in-plane loaded wall layer  
\( \theta_1 \) = rotation over height of stone/concrete layer  
\( \theta_2 \) = rotation over height of timber beam  
\( \theta_3 \) = rotation over height of infill piece

**Layer A: complete layer**

The failure mechanisms modelled below, are estimations of possible failure mechanism. The actual mechanism can be a combination of multiple modelled failure mechanisms. Note: \( M_{k,a,d} \) is different for the different kadil connections.

**Mechanism A1:**  
Internal rotation of the perpendicular beams, where the perpendicular beams and the infill pieces on the left-hand side rotate together. Assuming small angle rotations, the maanwi does not elongate due to these rotations, hence the amount of rotation between the two rotating beams is similar. On the right-hand side, the longitudinal infill piece will not rotate along with the outer perpendicular beam and both perpendicular beams will undergo different rotations. This has as consequence that the maanwi breaks. It is expected that after breaking of the maanwi the force will drop.

![Figure G.2: Failure mechanism A1](image-url)
Mechanism A2:
The left-hand side failure mechanism is the same as described in failure mechanism A1. At the right-hand side, both the infill pieces will not rotate along with the perpendicular beams and both perpendicular beams undergo the same rotation. When this failure mechanism occurs, none of the *maalwi*’s will be breaking.

![Figure G.3: Failure mechanism A2](image)

\[
(F_{v,4,d}x_4 + 2 M_{k,1,d} + M_{k,3,d} + F_{m,t,d} h_m) \frac{\delta u}{h_s} + (F_{v,3,d} x_3 + 4 M_{k,1,d} - F_{m,t,d} h_m) \frac{\delta u}{h_s} \\
+ (F_{v,2,d} x_2 + 4 M_{k,1,d} - F_{m,t,d} h_m) \frac{\delta u}{h_b} + (F_{v,1,d} x_1 + 3 M_{k,1,d} + F_{m,t,d} h_m) \frac{\delta u}{h_b} + F_{fr} \delta u = V_h \delta u
\]

Mechanism A3:
In mechanism 7, the perpendicular beams on the left-hand side will rotate, without rotation of the infill pieces. Consequently the *maalwi*’s are not breaking. The right hand side failure is the same as described in failure mechanism A2.

![Figure G.4: Failure mechanism A3](image)

\[
(F_{v,4,d} x_4 + 3 M_{k,1,d} + F_{m,t,d} h_m) \frac{\delta u}{h_b} + (F_{v,3,d} x_3 + 2 M_{k,1,d} + 2 M_{k,2,d} - F_{m,t,d} h_m) \frac{\delta u}{h_b} \\
+ (F_{v,2,d} x_2 + 4 M_{k,1,d} - F_{m,t,d} h_m) \frac{\delta u}{h_s} + (F_{v,1,d} x_1 + 3 M_{k,1,d} + F_{m,t,d} h_m) \frac{\delta u}{h_b} + F_{fr} \delta u = V_h \delta u
\]
9.1.1. Layer B: missing kadil in 3rd layer

Layer B described the behaviour in the layer with the missing connection, which is the 3rd layer in case of the experimental wall. The consequence of this missing connection is the ability of sliding to happen between the longitudinal beam and the longitudinal infill piece.

**Mechanism B1:**
The left hand side failure mechanism is based on the rotation of the perpendicular infill piece only. This infill piece can rotate as a consequence of the missing *kadils* in this layer. Because the rotation will only occur in the perpendicular infill piece a lot of the vertical force needs to be carried by this infill piece and rotation will cost more energy than when the longitudinal infill piece is also rotating. The rotations of the perpendicular beams on the right-hand side differ from each other, as a consequence of the different heights of the rotating beams.

\[
(F_v,3dx_3 + 2M_{k,1d} + 2M_{k,2d}) \frac{\delta u}{h_i} + (F_v,2dx_2 + 4M_{k,1d} - F_{m,t,d}h_m) \frac{\delta u}{h_s}
\]

\[
+ (F_v,1dx_1 + 3M_{k,1d} + F_{m,t,d}h_m) \frac{\delta u}{h_b} + F_{fr} \delta u = V_h \delta u
\]

**Mechanism B2:**
The failure mechanism on the left-hand side is the same as in failure mechanism B1. The rotation of the perpendicular beams at the right side of the layer is equal and the *maonwi* connection does not fail.

\[
(F_v,3dx_3 + 2M_{k,1d} + 2M_{k,2d}) \frac{\delta u}{h_i} + (F_v,2dx_2 - 4M_{k,1d} - F_{m,t,d}h_m) \frac{\delta u}{h_b}
\]

\[
+ (F_v,1dx_1 + 3M_{k,a,d} + F_{m,t,d}h_m) \frac{\delta u}{h_b} + F_{fr} \delta u = V_h \delta u
\]
Mechanism B3:
The *maanwi* on the left-hand side will pull the outer beam along with inner beam, which ensures that the rotation between the two left hand side perpendicular beams is equal. Sliding can happen between the longitudinal infill piece and the longitudinal beam, because the *kodil* connection between the two is missing. This means that the longitudinal infill piece does not have to rotate along with the perpendicular beam. The right-hand side beams rotate as described in failure mechanism B1.

![Figure G.7: Failure mechanism B3](image)

\[
(F_{v,4,a}x_4 + 3 M_{k,1,a}F_{m,t,a}h_m) \frac{\delta u}{c} + \left( F_{v,3,a}x_3 + 4 M_{k,a,d}F_{m,t,a}h_m \right) \frac{\delta u}{c} \\
+ \left( F_{v,2,a}x_2 + 4 M_{k,1,d}F_{m,t,a}h_m \right) \frac{\delta u}{c} + \left( F_{v,1,a}x_1 + 3 M_{k,1,a}F_{m,t,a}h_m \right) \frac{\delta u}{b} + F_{fr} \delta u = V_h \delta u
\]

Mechanism B4:
The left-hand side perpendicular beams rotate as described for mechanism B3. The right-hand side perpendicular beams rotate with the same angle and the longitudinal infill piece rotates along with the outer perpendicular beam. No virtual work contribution of the *maanwi*, hence the beams have same rotation.

![Figure G.8: Failure mechanism B4](image)

\[
(F_{v,4,u}x_4 + 3 M_{k,u}F_{m,t,u}h_m) \frac{\delta u}{c} + \left( F_{v,3,u}x_3 + 4 M_{k,u}F_{m,t,u}h_m \right) \frac{\delta u}{c} \\
+ \left( F_{v,2,u}x_2 + 4 M_{k,u}F_{m,t,u}h_m \right) \frac{\delta u}{c} + \left( F_{v,1,u}x_1 + 2 M_{k,u}F_{m,t,u}h_m \right) \frac{\delta u}{c} + F_{fr,static} \delta u = V_h \delta u
\]
Lateral layer force capacity results

The results are obtained by making use of MATLAB and displayed in the graphs in figure g.9, figure g.10 and figure g.11.

The MATLAB scripts can be found on the next pages.
Matlab file for full scale 7-layered wall

clear all; close all;

%% specify general input parameters

% input parameters timber strength
sigma_comp_perp = 2.2; % design compression strength perpendicular to the grain
sigma_comp_par = 15.7; % design compression strength parallel to the grain
sigma_tens_perp = 1.5; % design tension strength perpendicular to the grain
sigma_sh_roll = 1.6; % design rolling shear
E_wood = 13035; % youngs modulus of wood in N/mm^2

% input parameters friction
mu_st = 0.35; % static friction coefficient
alfa = 0.8; % distribution coefficient vertical force

% input parameters wall global
lw=4800; % length wall [mm]
vb=100; % width beams [mm]
v1=60; % length of longitudinal infill piece
vw=460; % width wall [mm]
h_in=lv-2*vw; % internal length [mm]
rho=18.94*10^-6; % density wall total [N/mm3]
c=0.06621739; % precompression [N/mm2]
number_layers_wall=7; % number of layers in the wall [#]
force_range_low=30000; % from which force you want to get results? [N]
force_range_high=15000; % until what force you want to get results? [N]
force_steps=100; % what is the magnitude of the force step you want to take? [N]

% input parameters layer
height_long_beam=175; % height of the longitudinal beams [mm]
height_perp_beam=175; % height of the perpendicular beams [mm]
height_infill=113; % height of the infill pieces [mm]
a = height_infill;
b = height_perp_beam;
c = (height_infill+height_perp_beam);

% input parameters kadil
wk=45; % width of kadil
wh=50; % height of kadil

% input parameters maanvi
hm = 175; % height of maanvi location
tm = 45; % thickness maanvi
vm = 120; % width maanvi
lm = 360; % length maanvi
lm_taper = 76; % length of maanvi taper end
vm_taper_end = 85; % width of largest part taper
vm_taper_small = 65; % width of smallest part taper
dpl = 69; % distance of maanvi to the end of the perpendicular beam
dp2 = 55; % distance of maanvi to the end of the perpendicular beam
sm = 125; % distance of maanvi from wall

%% start model
Fm_real = sigma_tens_perp*dpl*tm+sigma_sh_roll*0.5*(dpl+dp2)*tm; % actual maanvi force
capacity on distance from the wall
For which layer = 1:7

%alfa = 0.8+0.03*which_layer; % distribution coefficient vertical force

% first calculations
range=force_range_high-force_range_low; % force range
h1=height_long_beam+height_perp_beam+height_infill; % height of layer
h_eval=which_layer-1)*h1+height_long_beam; % height of wall on top of the evaluated layer
h_tot=(number_layers_wall-1)*h1+height_long_beam; % total height of the wall (excl. bottom stone/concrete layer)
W=(q-h_eval*rho)*1*v+w); % precompression including self weight
Mk = sigma_comp_perp*0.25*wk*vh*(1/3)*vh; % kadil elastic moment capacity (based on perpendicular embedment capacity)
Fr_stl = mu_st*M; % static friction coefficient full weight
Fr_stl = alfa*mu_st*M; % static friction coefficient part of the weight

force_check=zeros((range/force_steps),7); % final force matrix overview

% specific input parameters
F=force_range_low; % first force value
kl3=[3,4,4]; % amount of kadils in the layers
mm3 = [1, -1, 0]; % maanui application

% empty matrix formation
F_scl_check=zeros((range/force_steps),1); % horizontal force matrix
F_scl_check=zeros((range/force_steps),1); % vertical force check
F_scl3=zeros((range/force_steps),3); % reaction forces at the perpendicular beams
F_scl3=zeros((range/force_steps),3); % point of vertical force application on rotating beams
F_scl3=zeros((range/force_steps),3); % point of vertical force application on rotating longitudinal infill piece
FB1=zeros((range/force_steps),1); % mechanism 1, missing kadil, right side 288
FB2=zeros((range/force_steps),1); % mechanism 2, missing kadil, right side 175

% kath-kuni model

for k = 1:(range/force_steps)
    Fscl(k,1) = (1/3)^(1-alfa)*W;
    Fscl(k,2) = (1/3)^(1-alfa)*W;
    Fscl(k,3) = (1/3)^(1-alfa)*W;

% static friction in layer exceeded?
if F > Fr_stl
    force_check(k,1) = 1;
    force_check(k,2) = 1;
% rotational capacity point maximum point of force application
for n = 1:4
    x3max(k,n) = v1 - (Fsol3(k,n))/(2*vb*sigma_comp_perp));
    if x3max(k,n)<0
        x3max(k,n) = 0;
    end
end

for n = 1:4
    x3max1(k,n) = v1 - (Fsol3(k,n))/(2*vb*sigma_comp_perp));
    if x3max1(k,n)<0
        x3max1(k,n) = 0;
    end
end

% virtual work
F81(k,1) = ((Fsol3(k,3).*x3max(k,3)+2*Mk1+2*Mk2+mm2(1,3)*Fm.*hm)/a)+(((Fsol3(k,2).*x3max(k,2)+4*Mk1+mm3(1,2)*Fm.*hm)/c)+((Fsol3(k,1).*x3max(k,1)+3*Mk1+mm3(1,1)*Fm.*hm)/b)+(Fx_st2);
F82(k,1) = ((Fsol3(k,3).*x3max(k,3)+2*Mk1+2*Mk2+mm2(1,3)*Fm.*hm)/a)+(((Fsol3(k,2).*x3max(k,2)+4*Mk1+mm3(1,2)*Fm.*hm)/c)+((Fsol3(k,1).*x3max(k,1)+3*Mk1+mm3(1,1)*Fm.*hm)/b)+(Fx_st2);
if F81(k,1) < F
    force_check(k,1) = 2;
end
if F82(k,1) < F
    force_check(k,2) = 2;
end
end
Fsol_check(k,1) = Fsol3(k,1)+Fsol3(k,2)+Fsol3(k,3);
F_hox(k,1) = F;
F=F+force_steps;
end

% failure mechanism: 4 beams rotating

clear F1 F2 F3 F4 k n

% specific input parameters
F=force_range_low; % first force value
wl4=[3,4,4,3]; % amount of Cadis in the layers
mm4 = [1, -1, -1, 1]; % location and direction of functional meanvi

% empty matrix formation
F_hxo=zeros(((range/force_steps),1)); % horizontal force matrix
Fsol_check=zeros(((range/force_steps),4)); % vertical force check
Fsol4=zeros(((range/force_steps),4)); % reaction forces at the perpendicular beams
x3max=zeros(((range/force_steps),4)); % point of vertical force application on rotating beams
\( \text{xmaxl} = \text{zeros}((\text{range/force_steps}, 4)); \) % point of vertical force application on rotating longitudinal infill piece

FB3 = \text{zeros}((\text{range/force_steps}, 1)); \% mechanism 3, missing kadi, 4 rotating blocks
FB4 = \text{zeros}((\text{range/force_steps}, 1)); \% mechanism 4, left side long. infill rotates
FB1 = \text{zeros}((\text{range/force_steps}, 1)); \% mechanism 4, all kadils, both 288
FA2 = \text{zeros}((\text{range/force_steps}, 1)); \% mechanism 5, all kadils, right 175
FA3 = \text{zeros}((\text{range/force_steps}, 1)); \% mechanism 6, all kadi, left 175

\% distribution of vertical force due to moment
for \( k = 1:\) (\text{range/force_steps})
    FSol4(k,1) = (1/4) * (1-alfa) * W;
    FSol4(k,2) = (1/4) * (1-alfa) * W;
    FSol4(k,3) = (1/4) * (1-alfa) * W;
    FSol4(k,4) = (1/4) * (1-alfa) * W;
end

\% static friction in layer exceeded?
if \( F > \text{Fr_st1} \)
    force_check(k,1) = 1;
    force_check(k,4) = 1;
    force_check(k,5) = 1;
    force_check(k,6) = 1;
    force_check(k,7) = 1;
end

\% rotational capacity point maximum point of force application
for \( n = 1:4 \)
    xmaxn(k,n) = \text{ub} - ((FSol4(k,n))/(2*\text{ub}^*\text{sigma_comp_perp}));
    if xmaxn(k,n) < 0
        xmaxn(k,n) = 0;
    end
end

for \( n = 1:4 \)
    xmaxn(k,n) = \text{ub} - ((FSol4(k,n))/(2*\text{ub}^*\text{sigma_comp_perp}));
    if xmaxn(k,n) < 0
        xmaxn(k,n) = 0;
    end
end

\% virtual work
FS3(k,1) = ((FSol4(k,4) * x4max(k,4) + \text{Mkl1} * \text{mm4}(1,4) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,3) * x4max(k,3) + \text{Mkl1} * \text{mm4}(1,3) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,2) * x4max(k,2) + \text{Mkl1} * \text{mm4}(1,2) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,1) * x4max(k,1) + \text{Mkl1} * \text{mm4}(1,1) * \text{Fm} * \text{hm}) / c) + (\text{Fr_st2};

FA1(k,1) = ((FSol4(k,4) * x4max(k,4) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,4) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,3) * x4max(k,3) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,3) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,2) * x4max(k,2) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,2) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,1) * x4max(k,1) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,1) * \text{Fm} * \text{hm}) / c) + (\text{Fr_st2};

FA2(k,1) = ((FSol4(k,4) * x4max(k,4) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,4) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,3) * x4max(k,3) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,3) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,2) * x4max(k,2) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,2) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,1) * x4max(k,1) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,1) * \text{Fm} * \text{hm}) / c) + (\text{Fr_st2};

FA3(k,1) = ((FSol4(k,4) * x4max(k,4) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,4) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,3) * x4max(k,3) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,3) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,2) * x4max(k,2) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,2) * \text{Fm} * \text{hm}) / c) + ((FSol4(k,1) * x4max(k,1) + \text{Mkl1} + \text{Mkl3} + \text{mm4}(1,1) * \text{Fm} * \text{hm}) / c) + (\text{Fr_st2);
if $F_{B3}(k,1) < F$
    
    force_check(k,3) = 2;
end

if $F_{B4}(k,1) < F$
    
    force_check(k,4) = 2;
end

if $F_{A1}(k,1) < F$
    
    force_check(k,5) = 2;
end

if $F_{A2}(k,1) < F$
    
    force_check(k,6) = 2;
end

if $F_{A3}(k,1) < F$
    
    force_check(k,7) = 2;
end
end

$F_{s1} = F_{s14}(k,1) + F_{s14}(k,2) + F_{s14}(k,3) + F_{s14}(k,4)$;
$F_{hor}(k,1) = F/1000$;
$F=F+force_{steps}$;
end
Matlab file for full scale 5-layered wall

clear all; close all;

%% specify general input parameters

% input parameters timber strenght
sigma_comp_perp = 2.8; % compression strength perpendicular to the grain
sigma_comp_par = 24.5; % compression strength parallel to the grain
sigma_tens_perp = 2.0; % tension strength perpendicular to the grain
sigma_sh_rol = 2.0; % rolling shear
E_wood=5450; % youngs modulus of wood in N/mm^2

%% input parameters friction
mu_st = 0.35; % static friction coefficient
alfa = 0.7; % distribution coefficient vertical force

%% input parameters wall global
lw=2995; % length wall [mm]
w=100; % width beams [mm]
w1=360; % length of longitudinal infill piece
w=460; % width wall [mm]
h_in=1*w-2*ww; % internal length [mm]
rho=18.94*10^-6; % density wall total [N/mm3]
q=0.067; % precompression [N/mm2]
number_layers_wall=5; % number of layers in the wall [#]
force_range_low=30000; % from which force you want to get results? [N]
force_range_high=150000; % until what force you want to get results? [N]
force_steps=1000; % what is de magnitude of the force step you want to take? [N]

%% input parameters layer
height_long_beam=175; % height of the longitudinal beams [mm]
height_perp_beam=175; % height of the perpendicular beams [mm]
height_infill=113; % height of the infill pieces [mm]

%% input parameters kadil
w=45; % width of kadil
w=90; % height of kadil

%% input parameters maanwi
h_m = 175; % height of maanwi location
t_m = 45; % thickness maanwi
w_m = 120; % width maanwi
l_m = 380; % length maanwi
l_m_taper = 76; % length of maanwi taper end
w_m_taper_end = 25; % width of largest part taper
w_m_taper_small = 65; % width of smallest part taper
dp1 = 80; % distance of maanwi to the end of the perpendicular beam
dp2 = 95; % distance of maanwi to the end of the perpendicular beam
am = 125; % distance of maanwi from wall

%% start model
P_m_real = sigma_tens_perp*dp1*tm+sigma_sh_rol*0.5*(dp1+dp2)*tm; % actual maanwi force.
capacity on distance from the wall
Fx = (Fm_real*2*am+ww)/(ww+am))/2; % maanwi force at the wall central axis elastic!
Mk1 = 18422.3; % kadil capacity option 1
Mk2 = 3321.5; % kadil capacity option 2
Mk3 = 29418.2; % kadil capacity option 3
delt_m = Fm_real*(ln-2*lm_taper)/(tm*wm*E_wood); % maanwi deflection

for which_layer = 1:5

% first calculations
range=force_range_high-force_range_low; % force range
h1=height_long_beam+height_perp_beam-height_infill; % height of layer
h_eval=(which_layer-1)*h1+height_long_beam; % height of wall on top of the evaluated layer
h_tct=(number_layers_wall-1)*h1+height_long_beam; % total height of the wall (excl. bottom stone/concrete layer)
W=([q_h_eval+cho]*1*ww); % precompression including selfweight
Mk = sigma_comp_perp*0.25*ww+wh*(1/3)*wh; % kadil elastic moment capacity (based on perpendicular embedment capacity)
Fr_stl = nu_st*W; % static friction coefficient full weight
Fr_st2 = alpha*nu_st*W; % static friction coefficient part of the weight

force_check=zeros((range/force_steps),6); % final force matrix overview

%% failure mechanisms: 4 beams rotating

% specific input parameters
F=force_range_low; % first force value
kk3=[0,4,4]; % amount of kadils in the layers
mx3 = [1, -1, 0]; % maanwi application

% empty matrix formation
F_hx=zeros((range/force_steps),1); % horizontal force matrix
Fv01=zeros((range/force_steps),1); % vertical force check
Fv03=zeros((range/force_steps),3); % reaction forces at the perpendicular beams
xmax=zeros((range/force_steps),3); % point of vertical force application on rotating beams
xmaxi=zeros((range/force_steps),3); % point of vertical force application on rotating longitudinal infill piece
FB1=zeros((range/force_steps),1); % mechanism 1, missing kadil, right side 288
FB2=zeros((range/force_steps),1); % mechanism 2, missing kadil, right side 175

%% keth-khuni model

for k = 1:(range/force_steps)
    Fv03(k,1)=1/3*(1-alpha)*W;
    Fv03(k,2)=1/3*(1-alpha)*W;
    Fv03(k,3)=1/3*(1-alpha)*W;

    % static friction in layer exceeded?
    if F > Fr_stl
        force_check(k,1) = 1;
        force_check(k,2) = 1;

        % rotational capacity point maximum point of force application

end
for n = 1:3
    x3max(k,n)=wb-(Fsol3(k,n)/(2*wb*sigma_comp_perp));
    if x3max(k,n)<0
        x3max(k,n) = 0;
    end
end

for n = 1:3
    x3maxl(k,n)=wl-(Fsol3(k,n)/(2*wb*sigma_comp_perp));
    if x3maxl(k,n)<0
        x3maxl(k,n) = 0;
    end
end

% virtual work
FB1(k,1) = ((Fsol3(k,3).*x3max(x,k,3)+2*Mkl2*HM2+mm3(l,1,3)*Fm.*hm)/a)+(Fsol3(k,2).*
    x3max(x,k,2)+4*Mkl1*mm3(l,2)*Fm.*hm)/c+(Fsol3(k,1).*x3max(x,k,1)+3*Mkl1*mm3(l,1,1)*Fm.*
    hm)/b)+[Fz.sc2];
FB2(k,1) = ((Fsol3(k,3).*x3max(x,k,3)+2*Mkl2*HM2+mm3(l,1,3)*Fm.*hm)/a)+(Fsol3(k,2).*
    x3max(x,k,2)+4*Mkl1*mm3(l,2)*Fm.*hm)/c+(Fsol3(k,1).*x3max(x,k,1)+3*Mkl1*mm3(l,1,1)*Fm.*
    hm)/b)+[Fz.sc2];

if FB1(k,1) < F
    force_check(k,1) = 2;
end
if FB2(k,1) < F
    force_check(k,2) = 2;
end
end

Fsol1_check(k,1)=Fsol3(k,1)+Fsol3(k,2)+Fsol3(k,3);
F_hor(k,1) = F;
F=F-force_steps;
end

%% failure mechanisms: 4 beams rotating

clear F1 F2 F3 F4 k n

% specific input parameters
F=force_range_low;  % first force value
kkl=[3,4,4,3];    % amount of kklis in the layers
mm4 = [1, -1, -1, 1]; % location and direction of functional meanwi

% empty matrix formation
F_hor=zeros((range/force_steps),1);  % horizontal force matrix
Fsol1_check=zeros((range/force_steps),1);  % vertical force check
Fsol4=zeros((range/force_steps),4);    % reaction forces at the perpendicular beams
x3max=zeros((range/force_steps),4);    % point of vertical force application on rotating beams
x3maxl=zeros((range/force_steps),4);   % point of vertical force application on rotating beams
longitudinal infill piece
FB3 \= zeros((ranges/force_steps),1); % mechanism 3, missing kada, 4 rotating blocks
FB4 \= zeros((ranges/force_steps),1); % mechanism 4, left side long, infill rotates
FA1 \= zeros((ranges/force_steps),1); % mechanism 4, all kada, both 288
FA2 \= zeros((ranges/force_steps),1); % mechanism 5, all kada, right 175
FA3 \= zeros((ranges/force_steps),1); % mechanism 6, all kada, left 175

% distribution of vertical force due to moment
for k = 1:(ranges/force_steps)
    Fso14(k,1)\= (1/4) \* (1-alfa) \* W;
    Fso14(k,2)\= (1/4) \* (1+alfa) \* W;
    Fso14(k,3)\= (1/4) \* (1+alfa) \* W;
    Fso14(k,4)\= (1/4) \* (1-alfa) \* W;
end

% static friction in layer exceeded?
if F > Fr_st1
    force_check(k,3) = 1;
    force_check(k,4) = 1;
    force_check(k,5) = 1;
    force_check(k,6) = 1;
    force_check(k,7) = 1;
end

% rotational capacity point maximum point of force application
for n = 1:4
    x4max(k,n)\= \text{whb}-(Fso14(k,n)/(2*whb*sigma_comp_perp));
    if x4max(k,n)\< 0
        x4max(k,n) = 0;
    end
end

% virtual work
FB3(k,1)\= (Fso14(k,4) \* x4max(k,4) + 3*Wh1+mm4(1,4)*Fm.*hm)./c)+(Fso14(k,3) \* x4max(k,3) + 3*Wh1+mm4(1,3)*Fm.*hm)./c)+(Fso14(k,2) \* x4max(k,2) + 4*Wh1+mm4(1,2)*Fm.*hm)./c)+(Fso14(k,1) \* x4max(k,1) + 5*Wh1+mm4(1,1)*Fm.*hm)./c)+(Fso14(k,6) \* x4max(k,6) + 2*Wh1+mm4(1,6)*Fm.*hm)./c)+(Fso14(k,5) \* x4max(k,5) + Wh1+mm4(1,5)*Fm.*hm)./c)+(Fso14(k,7) \* x4max(k,7) + Wh1+mm4(1,7)*Fm.*hm)./c)+(Fso14(k,8) \* x4max(k,8) + 2*Wh1+mm4(1,8)*Fm.*hm)./c)+(Fso14(k,9) \* x4max(k,9) + Wh1+mm4(1,9)*Fm.*hm)./c)+(Fso14(k,10) \* x4max(k,10) + Wh1+mm4(1,10)*Fm.*hm)./c)+(Fso14(k,11) \* x4max(k,11) + Wh1+mm4(1,11)*Fm.*hm)./c)+(Fso14(k,12) \* x4max(k,12) + Wh1+mm4(1,12)*Fm.*hm)./c)+(Fso14(k,13) \* x4max(k,13) + Wh1+mm4(1,13)*Fm.*hm)./c)+(Fso14(k,14) \* x4max(k,14) + Wh1+mm4(1,14)*Fm.*hm)./c)+(Fso14(k,15) \* x4max(k,15) + Wh1+mm4(1,15)*Fm.*hm)./c);
end

if FB3(k,1) < F
force_check(k,3) = 2;
end

if FB4(k,1) < F
   force_check(k,4) = 2;
end

if FA1(k,1) < F
   force_check(k,5) = 2;
end

if FA2(k,1) < F
   force_check(k,6) = 2;
end

if FA3(k,1) < F
   force_check(k,7) = 2;
end
end

Fsol_check(k,1)=Fsol4(k,1)+Fsol4(k,2)+Fsol4(k,3)+Fsol4(k,4);
F_hor(k,1) = F/1000;
F=F+force_steps;
end
H. Experiment: test setup

In figure 6.6 the component names of the experimental setup can be found. The concrete test pit consists of a reaction wall and floor and four concrete bases, from which steel bolt rods are sticking out. C-profiles (ISMC400) are attached to two of these steel bolt rods. On these C-profiles, an ISBM450 is attached on which consequently a hinge is places aligned to the centre of the Kath-Kuni wall. Two ISMB250 (width: 125 mm) columns are loaded in tension and are attached to these hinges (height of hinges: 250 mm). In between these columns a reaction beam is attached (two ISBM450 girders attached with twelve 16 mm bolts (10.9). On top of the load-distributing beam the vertical hydraulic jack is placed in a stiff clamped steel base frame. The pin of the actuator will fall into a hole in an 8 mm thick plate (locally increased height of 16 mm), which is attached to the bottom of the reaction beam.

A stabilizing frame is setup to ensure that the lateral load on the vertical jack will be mineralized. It consists of four vertical angles ISA75.75.8 welded together with steel strips. This is attached to the steel beam by four bolts. A hinged ISA75.75.8 angle is connecting the bottom of the reaction beam and the angle. When the wall moves in horizontal direction, the load-distributing beam, the spreader beam and the vertical jack will move along with the wall and move with the same angle as the wall. The tip of the jack falls in the plate with a hole, where friction ensures that the reaction frame will move along with the vertical actuator in case horizontal displacement of the wall is relatively small. However, with large displacements, the rotation of the reaction frame and the wall will be different. Therefore, the pre-compressed vertical force applied by the vertical jack will get a horizontal component. This force will be taken by the stabilizing frame, which has a free degree of freedom in the vertical direction, but is restrained in the horizontal direction. This ensures that the system is able to elongate and shorten in vertical direction, but will have a negligible local horizontal displacement.

An out-of-plane resisting frame is fabricated to ensure that out-of-plane movement is minimalized at the height of the main load distributing beam. For every specimen this out-of-plane system needs to be adapted to the appropriate height and designed for the expected forces that come on the system.

The vertical hydraulic jack has a capacity of 500 kN and is a manual operated system. The stabilizing system, together with the base frame, has to ensure that vertical hydraulic jack is not disturbed, due to the lateral load. The vertical jack is calibrated before use.

The horizontal hydraulic actuator has a capacity of 500 kN and has an inbuilt load cell and Linear Variable Differential Transformers (LVDTs). The LVDT measurements will not be used as final displacements of the wall, as the stiffness of the reaction wall and actuator base plate might not be infinite. The hydraulic actuator is fixed to the reaction wall with 6 Ø24mm bolts. The test setup is a cantilever system. In between the actuator and the specimen wall, rollers are applied to make sure vertical movement of the wall is not restricted. The rollers (diameter 100 mm) are attached to a 35 mm plate.

With every specimen tested in the experimental setup, a horizontal component of the vertical force (due to rotation of the system) has to be distracted of the measured force. This horizontal component is calculated by a geometrical calculation of the rotating test setup (Figure H.2) and distracted from the horizontal force. This resulting horizontal force component can be observed in Figure H.1 as the resulting negative horizontal force in the wall after the horizontal actuator force is removed. The vertical force that has a resulting horizontal negative force component on the wall even after unloading of the horizontal actuator. The wall itself is bringing this force to the ground.
Figure H.1: The results of the horizontal actuator LVDT

Figure H.2: Force - displacement of the horizontal component of the vertical jack force
I. Experiment: loading scheme

The weight and density of the experimental Kath-Kuni wall is determined in Table I.1 and Table I.2.

<table>
<thead>
<tr>
<th>Material</th>
<th>Component</th>
<th>Dimensions [mm]</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>l</td>
<td>w</td>
</tr>
<tr>
<td>Timber</td>
<td>Longitudinal beams</td>
<td>2995</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Perpendicular beams</td>
<td>960</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Infill pieces</td>
<td>460</td>
<td>100</td>
</tr>
<tr>
<td>Concrete</td>
<td>Block</td>
<td>500</td>
<td>230</td>
</tr>
<tr>
<td></td>
<td>Wall layer</td>
<td>2075</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>Bottom glued layer</td>
<td>2995</td>
<td>460</td>
</tr>
<tr>
<td></td>
<td>Corners</td>
<td>360</td>
<td>360</td>
</tr>
<tr>
<td>Rubble</td>
<td>In between longitudinal timber beam</td>
<td>2995</td>
<td>260</td>
</tr>
<tr>
<td></td>
<td>In between perpendicular timber beam</td>
<td>460</td>
<td>260</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material</th>
<th>Density [kN/m$^3$]</th>
<th>Volume [m$^3$]</th>
<th>Weight [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber</td>
<td>5.4</td>
<td>10*0.0524</td>
<td>4.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16*0.0168</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>16*0.0052</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>25</td>
<td>4*0.2749</td>
<td>30.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8*0.0146</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.22</td>
<td></td>
</tr>
<tr>
<td>Rubble</td>
<td>20.4</td>
<td>5*0.1363</td>
<td>17.34</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8*0.0209</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.85</td>
<td></td>
</tr>
<tr>
<td>TOTAL</td>
<td></td>
<td>52.59</td>
<td></td>
</tr>
</tbody>
</table>

The design of the setup guarantees cantilever boundary conditions for the wall. The vertical and rotational degree of the wall is not restrained on the top of the wall. However, they are restrained at the bottom of the wall, where the bottom concrete block row is glued to the concrete reaction floor.

During the in-plane testing, the wall is subjected to a vertical pre-compression (overburden) force, which is applied by one vertical actuator that is placed between the load spreading beam and the reaction beam. The vertical hydraulic jack has a capacity of 500 kN and jack is a manual force controlled system. The force on the wall is kept constant throughout the testing by keeping the force in the external load cell constant (the vertical load range is between 82 and 88 kN. This results in an average pre-compression force 0.067 MPa (30.8 kN/m).

The horizontal hydraulic actuator is a displacement controlled system and has a capacity of 500 kN and a sensitivity of 0.01 mm/second. The horizontal displacement controlled loading is applied in steps of either 5 kN or 10 mm, whichever of those is smaller. The speed of the actuator will be set to 5 mm/second and has a reach of 300 mm (85 mm is the approximate distance from the actuator to the wall, so the total displacement is restricted by approximately 215 mm). In between the steps, the dial gauges are read. A quasi-static force is applied, until failure. The test will be stopped when failure will generate safety issues for the test setup, or at a maximum displacement of 210 mm in Linear Variable Differential Transformer (LVDT) 2, restricted by the capacity of the catching frame. The maximum out-of-plane displacement of the masonry wall is set to 1/12$^{th}$ of the width of the wall (= 38 mm).

The vertical applied force will be brought to the foundation by two columns that will be loaded in tension. The tension force on the foundation is equal to the applied load of the actuator, minus the self-weight $W_F$ of the reaction beam, columns, lower beams and the C-profiles.
The compression force on the wall has the magnitude of the applied force in the actuator, plus the own weight of the spreader beam, the load distribution beam and the stabilizing angle $W_T$. This results in an average overburden on the wall of 0.067 MPa. The contribution of the eccentricity of the weight of the stabilizing system is neglected, as this is only 0.2% of the total vertical force applied in the vertical jack. The following table gives the own weights of the main parts of the setup:

<table>
<thead>
<tr>
<th>Self-weights</th>
<th>kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reaction beam (2x ISBM450)</td>
<td>4.1  kN</td>
</tr>
<tr>
<td>Columns (2x ISBM250)</td>
<td>2.2 kN</td>
</tr>
<tr>
<td>Lower beams (2x ISBM450)</td>
<td>3.6  kN</td>
</tr>
<tr>
<td>C-profile (4x ISMC400)</td>
<td>0.8  kN</td>
</tr>
<tr>
<td>Steel spreader beam (h: 385 mm, w: 150 mm, adapted from ISBM45)</td>
<td>1.3  kN</td>
</tr>
<tr>
<td>Steel load distribution beam (4x ISBM250 + plates)</td>
<td>5.3  kN</td>
</tr>
<tr>
<td>Stabilizing frame angle (4x ISA75.75.8)</td>
<td>0.5  kN</td>
</tr>
</tbody>
</table>

The load on the wall can be formulated as:

$$ P = W_T + F_V $$

Where $P$ is the vertical force on the wall, $W_T$ is the self-weight of the part of the setup in between the vertical jack and the wall and $F_V$ is the force in the vertical actuator.

The loading is performed according to the following steps:

- A strain measurement is done on the day before to obtain the strains in the different parts of the frame without loading, in order to do a proper temperature calibration.
- A vertical loading test will be performed before the actual push-over test starts to a maximum of 250 kN. The vertical load will be applied and the out-of-plane measurements will be taken close to the out-of-plane rollers. In case the amount of displacement is below 2 mm, the vertical load is placed with minimum eccentricity and the vertical jack does not have to be changed in position.
- The concrete blocks are tested for their compression strength.
- First the vertical pre-compression load is applied, which is kept constant throughout the complete test. The vertical load $W_T$ is taken into account. The load cell placed under the vertical jack will give the amount of load and has to be kept constant by adapting the force in the vertical jack.
- Secondly, the horizontal load is applied in a displacement controlled manner, with a speed of 0.05 mm/s.
- Every 5 kN or every 10 mm the actuator is pauzed to take the dial guage readings, whichever of both is reached first.
- The test is continued until failure. When failure is creating a safety risk, the test is stopped. The following check points are used during the test.
  - If the displacement of the wall including the load distributing beam is more than 210 mm (LVDT2).
  - If the out-of-plane displacement reaches 38 mm (1/12 * with of wall).
  - In case of a sudden drop in force > 0.8 * Fmax, in case continuing loading will not generate additional value (mostly with sliding)
  - If the force in the stabilizing frame exceeds 40 kN ( = 140 microstrain).
  - In case the vertical displacement is more than 100 mm, the horizontal actuator rollers will not be in contact with the steel anymore. Furthermore, the total uplift in the middle of the wall cannot exceed 80 mm, as this is the distance that the vertical actuator is extended. If the wall reached its 200 mm displacement it can undergo a maximum uplift in LVDT 5 of 100 mm.
J. Experiment: instrumentation

Instrumentation is applied to the IITR_PILOT_02 experimental Kath-Kuni wall. Displacements of the specimen are measured during the test and strains are measured in the experimental setup to ensure safety of the setup. In total there are 13 LVDT’s applied with a stroke of 50 mm. And 24 dial guages, from which 21 with a stroke of 25 mm and 3 with a stroke of 50 mm. The full instrumentation scheme can be found in figure j.1 and table j.1.

![Instrumentation scheme of IITR_PILOT_02](image)

**Figure J.1: Instrumentation scheme of IITR_PILOT_02**

**Table J.1: Instrumentation scheme of IITR_PILOT_02**

<table>
<thead>
<tr>
<th>No.</th>
<th>Description [all applied in center of width of wall unless described differently]</th>
<th>Direction</th>
<th>Sensor Type</th>
<th>Stroke (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Displacement left top side of the wall</td>
<td>Horizontal</td>
<td>LVDT</td>
<td>50</td>
</tr>
<tr>
<td>2</td>
<td>Displacement left top side of the wall</td>
<td>Horizontal</td>
<td>Dial guage</td>
<td>50</td>
</tr>
<tr>
<td>3</td>
<td>Displacement right top side of the wall</td>
<td>Horizontal</td>
<td>LVDT</td>
<td>50</td>
</tr>
<tr>
<td>4</td>
<td>Displacement right top side of the steel beam</td>
<td>Horizontal</td>
<td>LVDT</td>
<td>50</td>
</tr>
<tr>
<td>5</td>
<td>Displacement left top side of the steel beam</td>
<td>Vertical</td>
<td>LVDT</td>
<td>±25 (=50)</td>
</tr>
<tr>
<td>6</td>
<td>Displacement right top side of the steel beam</td>
<td>Vertical</td>
<td>LVDT</td>
<td>±25 (=50)</td>
</tr>
<tr>
<td>7a</td>
<td>Displacement left bottom front side of the wall</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>7b</td>
<td>Displacement left bottom back side of the wall</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>8a</td>
<td>Displacement right bottom front side of the wall</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>8b</td>
<td>Displacement right bottom back side of the wall</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>Displacement of the 1st timber beam from the bottom on the right side</td>
<td>Horizontal</td>
<td>LVDT</td>
<td>50</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>10</td>
<td>Displacement of the 2nd timber beam from the bottom on the right side</td>
<td>Horizontal</td>
<td>LVDT</td>
<td>50</td>
</tr>
<tr>
<td>11a</td>
<td>Displacement of the 3rd timber beam from the bottom on the right side (front side)</td>
<td>Horizontal</td>
<td>LVDT</td>
<td>50</td>
</tr>
<tr>
<td>11a</td>
<td>Displacement of the 3rd timber beam from the bottom on the right side (front side)</td>
<td>Horizontal</td>
<td>Dial guage</td>
<td>50</td>
</tr>
<tr>
<td>11b</td>
<td>Displacement of the 3rd timber beam from the bottom on the right side (back side)</td>
<td>Horizontal</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>12</td>
<td>Displacement of the 4th timber beam from the bottom on the right side</td>
<td>Horizontal</td>
<td>LVDT</td>
<td>50</td>
</tr>
<tr>
<td>13a</td>
<td>Relative displacement between the 1st and 2nd beam on the front side</td>
<td>Vertical</td>
<td>LVDT</td>
<td>50</td>
</tr>
<tr>
<td>13b</td>
<td>Relative displacement between the 1st and 2nd beam on the back side</td>
<td>Vertical</td>
<td>LVDT</td>
<td>50</td>
</tr>
<tr>
<td>14a</td>
<td>Relative displacement between the 2nd and 3rd beam on the front side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>14b</td>
<td>Relative displacement between the 2nd and 3rd beam on the back side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>15a</td>
<td>Relative displacement between the 3rd and 4th beam on the front side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>15b</td>
<td>Relative displacement between the 3rd and 4th beam on the back side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>16a</td>
<td>Relative displacement between the 4th and 5th beam on the front side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>16b</td>
<td>Relative displacement between the 4th and 5th beam on the back side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>17a</td>
<td>Relative displacement between the 1st and 2nd beam on the front side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>17b</td>
<td>Relative displacement between the 1st and 2nd beam on the back side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>18a</td>
<td>Relative displacement between the 2nd and 3rd beam on the front side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>18b</td>
<td>Relative displacement between the 2nd and 3rd beam on the back side</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>19</td>
<td>Displacement of the top of the wall in out-of-plane direction</td>
<td>Horizontal</td>
<td>Dial guage</td>
<td>±12,5 (=25)</td>
</tr>
<tr>
<td>20</td>
<td>Displacement of the middle of the wall in out-of-plane direction</td>
<td>Horizontal</td>
<td>LVDT</td>
<td>±25 (=50)</td>
</tr>
<tr>
<td>21</td>
<td>Displacement of the bottom of the wall in out-of-plane direction</td>
<td>Horizontal</td>
<td>Dial guage</td>
<td>±12,5 (=25)</td>
</tr>
<tr>
<td>22</td>
<td>Relative displacement between steel beam and top timber beam (front side)</td>
<td>Horizontal</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>23</td>
<td>Relative displacement between the first masonry row and the floor (front side)</td>
<td>Horizontal</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>24</td>
<td>Displacement of the first masonry row relative to the floor</td>
<td>Vertical</td>
<td>Dial guage</td>
<td>25</td>
</tr>
<tr>
<td>25</td>
<td>Displacement of the bottom of the steel beam in out-of-plane direction (right side of wall)</td>
<td>Horizontal</td>
<td>Dial guage</td>
<td>±12,5 (=25)</td>
</tr>
<tr>
<td>FR1a</td>
<td>Strain in the stabilizing frame</td>
<td>-</td>
<td>Strain gauge</td>
<td>-</td>
</tr>
<tr>
<td>FR1b</td>
<td>Strain in the stabilizing frame</td>
<td>-</td>
<td>Strain gauge</td>
<td>-</td>
</tr>
<tr>
<td>FR2a</td>
<td>Strain in left column</td>
<td>-</td>
<td>Strain gauge</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Description</td>
<td></td>
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<tr>
<td>-----</td>
<td>-----------------------------------------------------------------------------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FR2b</td>
<td>Strain in left column</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FR3a</td>
<td>Strain in the right column</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FR3b</td>
<td>Strain in the right column</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FR4a</td>
<td>Spreader beam strain, bottom upper flange</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FR4b</td>
<td>Spreader beam strain, bottom lower flange</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F1</td>
<td>Vertical hydraulic jack load cell (max. 500 kN)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F2</td>
<td>Spreader beam strain, bottom lower flange</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F2</td>
<td>Displacement in the horizontal hydraulic actuator (build in)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Vertical Load cell

Horizontal Load cell

LVDT ±300

Some example pictures of the dial gauges and LVDTs applied on the IITR_PILOT_02 specimen can be found in Figure J.2.

(a)  (b)  (c)

(d)  (e)  (f)

Figure J.2: Photos of some of the applied dial gauges
K. Experiment: load steps

The following table gives an overview of the load steps taken during the testing of Kath-Kuni wall IITR_PILOT_02.

Table K.1: Time steps taken during the testing of IITR_PILOT_02

<table>
<thead>
<tr>
<th>Step</th>
<th>Start time</th>
<th>Starting load [kN]</th>
<th>Pauze time</th>
<th>End load [kN]</th>
<th>Displacement LVDT 1 [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>09:39:48</td>
<td>-1.2</td>
<td>09:43:43</td>
<td>-1.2</td>
<td>-0.06</td>
</tr>
<tr>
<td>2</td>
<td>09:57:39</td>
<td>-1.4</td>
<td>10:03:11</td>
<td>5.6</td>
<td>0.32</td>
</tr>
<tr>
<td>3</td>
<td>10:14:12</td>
<td>3.4</td>
<td>10:14:28</td>
<td>8.8</td>
<td>0.64</td>
</tr>
<tr>
<td>4</td>
<td>10:34:57</td>
<td>7.8</td>
<td>10:35:08</td>
<td>10.4</td>
<td>0.98</td>
</tr>
<tr>
<td>5</td>
<td>10:44:50</td>
<td>12.5</td>
<td>10:45:11</td>
<td>23.6</td>
<td>2.20</td>
</tr>
<tr>
<td>7</td>
<td>11:05:44</td>
<td>26.4</td>
<td>11:06:43</td>
<td>33.9</td>
<td>6.34</td>
</tr>
<tr>
<td>8</td>
<td>11:21:47</td>
<td>31.6</td>
<td>11:23:21</td>
<td>37.4</td>
<td>11.08</td>
</tr>
<tr>
<td>9</td>
<td>11:33:55</td>
<td>34.3</td>
<td>11:35:27</td>
<td>39.8</td>
<td>15.41</td>
</tr>
<tr>
<td>10</td>
<td>11:46:36</td>
<td>35.4</td>
<td>11:48:17</td>
<td>42.2</td>
<td>20.43</td>
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<tr>
<td>11</td>
<td>11:57:34</td>
<td>37.5</td>
<td>11:59:12</td>
<td>44.1</td>
<td>25.07</td>
</tr>
<tr>
<td>12</td>
<td>12:09:08</td>
<td>40.5</td>
<td>12:10:46</td>
<td>45.8</td>
<td>29.72</td>
</tr>
<tr>
<td>13</td>
<td>12:22:15</td>
<td>40.5</td>
<td>12:23:53</td>
<td>46.5</td>
<td>34.36</td>
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<tr>
<td>14</td>
<td>12:41:07</td>
<td>42.0</td>
<td>12:42:45</td>
<td>47.8</td>
<td>38.88</td>
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<tr>
<td>15</td>
<td>13:11:45</td>
<td>42.7</td>
<td>13:16:44</td>
<td>47.8</td>
<td>48.52</td>
</tr>
<tr>
<td>16</td>
<td>13:26:45</td>
<td>45.1</td>
<td>13:32:07</td>
<td>50.0</td>
<td>57.40</td>
</tr>
<tr>
<td>17</td>
<td>14:01:07</td>
<td>44.0</td>
<td>14:06:16</td>
<td>48.5</td>
<td>66.75</td>
</tr>
<tr>
<td>18</td>
<td>14:56:45</td>
<td>45.6</td>
<td>15:01:46</td>
<td>50.2</td>
<td>76.04</td>
</tr>
<tr>
<td>19</td>
<td>15:31:36</td>
<td>44.4</td>
<td>15:36:47</td>
<td>49.5</td>
<td>85.89</td>
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<td>45.3</td>
<td>15:49:50</td>
<td>50.2</td>
<td>95.28</td>
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<tr>
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<td>16:00:54</td>
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<td>16:04:39</td>
<td>49.5</td>
<td>104.44</td>
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<td>16:22:40</td>
<td>42.7</td>
<td>16:27:18</td>
<td>42.8</td>
<td>114.27</td>
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<td>39.4</td>
<td>17:13:09</td>
<td>42.9</td>
<td>123.55</td>
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<tr>
<td>24</td>
<td>17:24:12</td>
<td>37.9</td>
<td>17:29:08</td>
<td>38.8</td>
<td>133.33</td>
</tr>
<tr>
<td>25</td>
<td>17:46:35</td>
<td>35.4</td>
<td>17:50:56</td>
<td>37.1</td>
<td>143.00</td>
</tr>
<tr>
<td>26</td>
<td>18:02:37</td>
<td>35.1</td>
<td>18:08:36</td>
<td>39.6</td>
<td>152.35</td>
</tr>
<tr>
<td>27</td>
<td>18:55:31</td>
<td>32.3</td>
<td>19:00:00</td>
<td>42.4</td>
<td>161.64</td>
</tr>
<tr>
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<td>19:14:42</td>
<td>35.6</td>
<td>19:18:55</td>
<td>41.6</td>
<td>171.15</td>
</tr>
<tr>
<td>29</td>
<td>19:39:10</td>
<td>36.0</td>
<td>19:43:10</td>
<td>44.5</td>
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<tr>
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<td>19:54:42</td>
<td>36.4</td>
<td>19:58:14</td>
<td>42.1</td>
<td>189.12</td>
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<td>20:34:39</td>
<td>35.4</td>
<td>20:38:11</td>
<td>37.8</td>
<td>198.78</td>
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<td>32</td>
<td>20:58:13</td>
<td>Unloading horizontal actuator</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
This appendix contains the full experimental photo analysis of Kath-Kuni specimen IITR_PILOT_02.

Figure L.1: OVERVIEW DAMAGE WALL OVER TIME (LS = Load step)

LS 2 – Left back side 3rd layer: cracks in longitudinal beams
LS 3 – Left upper front side: bend in the upper part of the wall
LS 7 – Left upper front side: bending
LS 7 – Left back side 3/4th layer
LS 8 – Upper layer right back side: displacement of concrete blocks
LS 9 – Right front side 4th layer: crushing of infill piece
Figure L.2: OVERVIEW INFILL PIECES OVER TIME (LS = Load step)
Figure L.3: OVERVIEW MAANWI’S OVER TIME