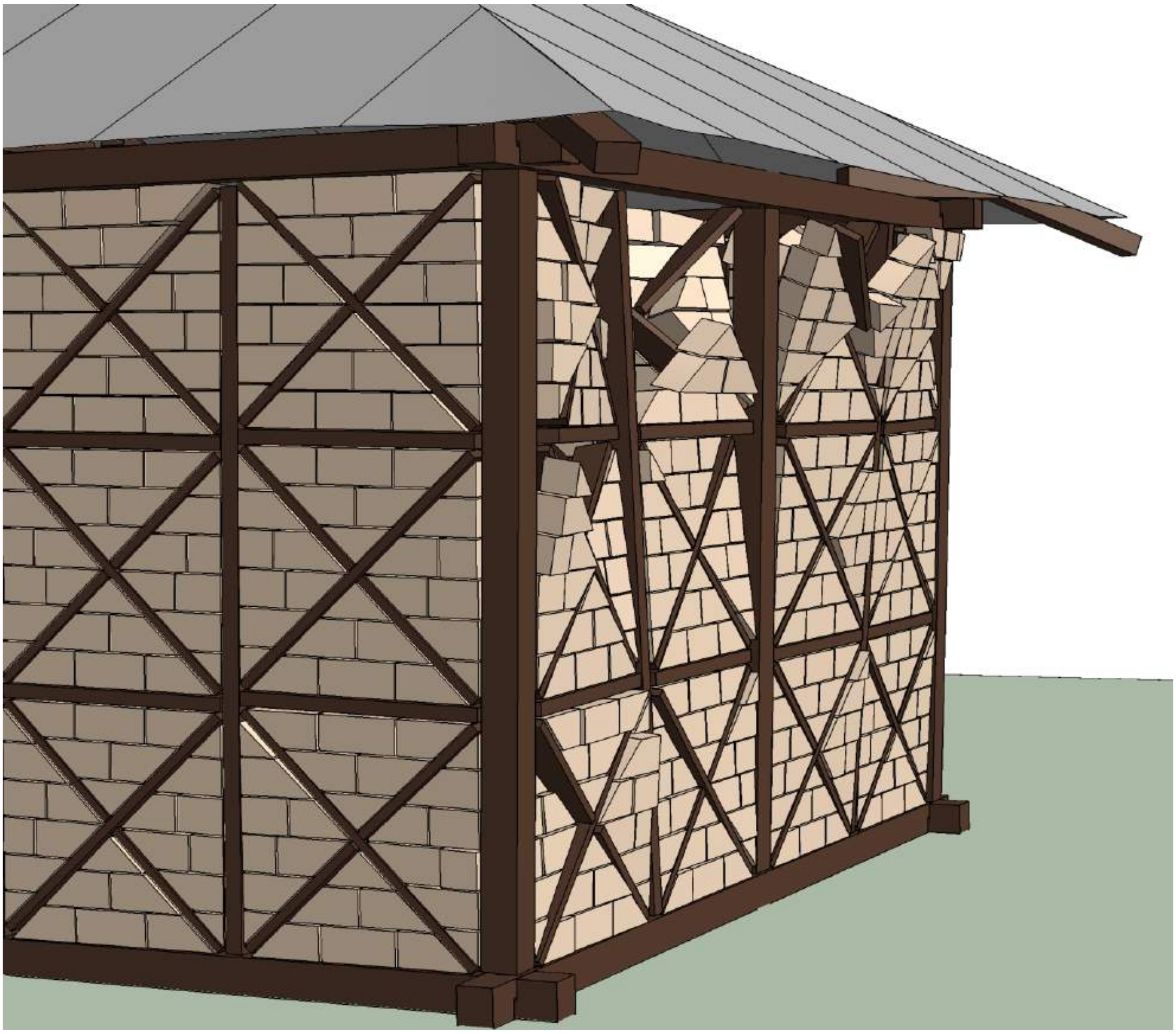
Dhajji Dewari

Affordable seismically resistant and sustainable housing

Rev C | September 2011



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Contents

Foreword

It is commonly cited that earthquakes don't kill people – buildings do. This was self-evident in the recent seismic disasters in Pakistan (2005) and Haiti (2010), particularly due to unreinforced masonry or poorly constructed reinforced concrete. Hence in post-disaster re-construction the notion of 'build back better' has been frequently interpreted as 'build back safer' so as to mitigate the risk from future seismic events. In this respect, identifying and promoting construction methods that are likely to result in safe housing that is affordable, culturally acceptable and can be built, maintained and adapted by local people, presents a major challenge.

Typically, the default is reinforced/confined masonry or reinforced concrete for which there are various codes, standards and guidelines that provide a mechanism to assure the quality of design, workmanship and materials. Yet, masonry and concrete are often considered to be complex or costly, and developing the necessary skills and institutionalising safe construction practice is not straightforward. In contrast vernacular building types, which use local materials and have performed well in earthquakes, are often over-looked by decision makers and donors since there is only empirical rather than scientific evidence to justify their performance. Investment is needed in research to better understand why such structures perform well in earthquakes, and to provide the evidence base that will enable wider acceptance and adoption.

In 2005-2006 Kubilay Hicyilmaz, and Associate at Arup, spent several months in Pakistan and witnessed this conundrum with respect to attitudes towards re-building using dhajji dewari. This form of timber and masonry infill construction has evolved over centuries, and similar forms of construction also exist in many other earthquake prone countries. As an experienced structural analyst with seismic expertise, he also recognised that advances in structural analysis software meant that it should be possible to analyse such structures. Hence, this research project was initiated and has been supported by Arup. The findings clearly demonstrate the appropriateness of this type of construction in seismic areas, and the opportunity it affords to provide a technology that is safe, sustainable and affordable. Our hope is that this initial research will encourage others to recognise the merits of this form of construction, and ultimately lead to the development of engineering standards, construction guidelines and training materials which will enable it to be more widely adopted.

Jo da Silva

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Director – International Development, Arup

Acknowledgments

This research has been carried out by Kubilay Hicyilmaz as a result of his curiosity and persistence to better understand the seismic behaviour of dhajji dewari construction. He has benefited from the support and expertise of a number of motivated colleagues at Arup, including: Tom Wilcock, Conrad Izatt, Damian Grant, Alan Cantos, Zsuzsanna Schreck, Jo da Silva, Ruth Kestermann, Rene Ciolo, Faustino Abbad, Nelson Soriano and Reynaldo de Guzman.

This report has relied on UN-Habitat, Pakistan for field information on local practices of dhajji dewari construction. In particular, Arup would like to thank Maggie Stephenson for her valuable advice and support; also Sheikh Ahsan Ahmed, S. Habib Mughal, Hamid Mumtaz and Babar Tanwir.

Further thanks are due to Dr. Ali Qaisir and his team from UET Peshawar for sharing valuable test data for bench marking purposes. Also, to Tom Schacher and Randolph Langenbach for their on-going support, discussions and inquisitive questioning of the work and for sharing their significant experience in traditional construction through the course of this work.

Finally, this work would not have been possible without the financial support from Arup and Randolph Langenbach and Frederick Charles Hertz through Kashmir Earthquake Relief (KER), an NGO established by Rafique Khan to support recovery and reconstruction after the 2005 Kashmir earthquake.

Executive Summary

The term dhajji dewari is derived from a Persian word meaning “patchwork quilt wall” and is a traditional building type found in the western Himalayas. Is a straightforward construction technology that can be easily built using local materials; timber and masonry infill. Similar forms of construction exist around the world (see Appendix A).

After the October 2005 Kashmir earthquake over 100,000 homes were reconstructed using this indigenous construction method. However, initially there was reluctance by donors and ERRA to promote or fund this type of construction in the absence of scientific, as opposed to empirical evidence of structural performance. There is very limited research to validate the performance of dhajji dewari construction (see Appendix B). A better understanding of the structural behaviour of dhajji dewari buildings is needed as a first step towards providing confidence in this technology, and to identify those aspects which are critical to the reliable performance of the building system.

Figure 0-1 Dhajji dewari under construction and image during a seismic analysis

This research has been carried out by Arup. Its purpose is to understand the structural behaviour of a typical dhajji dewari house, similar to those built after the 2005 Pakistan earthquake, using state of the art engineering analysis. Specifically, it sought to establish whether the building type could be accurately modelled, and in so doing determine how it theoretically performs when subjected to large earthquake loads. Hence, to establish what the critical engineering details are to help ensure reliable seismic performance and identify measures that might enhance performance.

The analysis was carried out by Arup’s Advanced Technology and Research Group using a non-linear finite element program (LS-DYNA). The analysis was based on a typical single storey dhajji dewari building constructed after the 2005 Pakistan earthquake. The timber elements, masonry infill pieces and roof have been modelled explicitly. Mortise and tenon as well as scarf joints have been explicitly modelled. Nailed connections have been idealised as discrete elements, and parallel analyses undertaken to reflect joints with and without nails. The limiting factor for the analysis has been the number of elements and associated processing time (approximately 68 hours).

Both ‘pushover’ and ‘time-history’ analyses of a whole house model were carried out to establish overall performance. In addition, initial sensitivity analyses were carried out on sub-frames to explore the impact of overburden on the walls due to additional storeys, and the effect of making the timber diagonal braces shorter to account for lack of fit or shrinkage over time and to mimic the situation of not having braces at all when sufficiently shortened were investigated. The analytical model was also verified by comparison with a physical model and tests undertaken by the University of Engineering Technology (UET) in Peshawar.

We have shown that it is possible to model the behaviour of traditional dhajji dewari buildings, and that this form of construction that can safely withstand forces associated with earthquakes in high seismic regions. The dhajji dewari construction behaviour is conceptually similar to 'confined masonry' construction although it is significantly more ductile as the mud mortar allows it to yield under relatively small lateral loads, and energy is dissipated in friction between the infill pieces as they slide across each other. Vertical pre-stress of the masonry (E.g. due to additional stories) increases friction and therefore the ability of the infill walls to absorb energy. The timber framing provides stable confinement to the masonry whilst it remains properly connected. The confinement also helps limit the out of plane demands on the infill masonry. Detailing of connections is important and is improved with strategic use of nails and possibly also straps.

This research is an important step in understanding the behaviour of dhajji dewari structures and creating a validated analytical model based on which further sensitivity analyses can be undertaken to test the performance of critical elements, also alternative configurations of dhajji dewari structures. If this form of construction is to become widely accepted by the general public, donors and governments, further investment and research is needed, ultimately leading to:

- An evidence based earthquake engineering building standard and construction guidelines for dhajji dewari buildings.
- An evidence based earthquake engineering building standard and construction guidelines for retro-fitting existing dhajji dewari buildings.
- Training materials aimed at self-builders, university students, architects and engineers and government.

1 Introduction

On the 8th of October 2005, a magnitude 7.6 earthquake struck Northern Pakistan at 8:50 in the morning. An estimated 74,000 people were killed and over 460,000 homes, over 5,000 schools and almost 800 health facilities were destroyed in a matter of seconds across 4000 villages leaving 3.5 million people homeless.

The challenges of reconstruction were compounded by the difficult mountain conditions at the foot hills of the Himalayas. Initially official guidelines were to rebuild using modern engineering methods which principally consisted of reinforced concrete construction. However the high cost of the materials, the difficulty of transporting the materials to remote locations and storing them safely made this form of construction too costly for many. In addition, reinforced concrete construction was a relatively sophisticated, unfamiliar construction method to those having to rebuild in the owner driven reconstruction programme guided by the Earthquake Rehabilitation and Reconstruction Authority (ERRA) of Pakistan.

Many locals started rebuilding their homes using the locally known traditional construction technique of dhajji dewari. These people had observed that many of the dhajji dewari houses had performed reasonably well during the earthquake by not collapsing to the extent that modern brittle masonry and reinforced concrete structures seemed to. People could afford to rebuild using dhajji dewari, they knew how to build it and it allowed them to recycle much of the building materials from their destroyed homes. Nevertheless, initially there was reluctance by donors and ERRA to promote or fund this form of construction[1]. This meant many people, often the poorest in society, were put at risk of not receiving financial and technical reconstruction support by ERRA.

Dhajji dewari is a straightforward construction technology that can be easily built from local materials; timber and masonry. However, as found from the literature review and field evidence it has evolved based on past performance, and has a limited basis in formal engineering justification (see Appendix B). This research was proposed and has been carried out by Kubilay Hicyilmaz, an Associate at Arup, who spent several months in Pakistan in 2005-2006 providing advice on earthquake resistant construction. He recognised the need for a better understanding of the structural behaviour of dhajji dewari buildings, in order to identify those aspects which are critical to the reliable performance of the building system as a first step towards providing wider confidence in this technology.

The main body of this report briefly describes dhajji dewari construction (section 2), then presents the findings of a limited but detailed earthquake engineering assessment of the dhajji dewari construction form as typically found in Pakistan as shown in Figure 2-1 and Figure 2-2. It provides a summary of the methodology (section 3) and structural analysis (section 4), followed by key findings (section 5), conclusions and recommendations (section 6).

Examples of similar construction types from around the world, field evidence, and detailed descriptions of dhajji dewari are presented in Appendix A to Appendix C. Detailed descriptions of the project analysis models along with our engineering and analysis assumptions are included in Appendix D to enable others to build upon this work in the future.

[1] It was not until the spring of 2007, dhajji dewari was accepted by donors and ERRA as a reconstruction alternative to reinforced concrete construction.

2 Description

The term dhajji dewari is derived from a Persian word meaning “patchwork quilt wall” and is a traditional building type found in the western Himalayas. Examples from around the world are shown in Appendix A. Such houses are found in both the Pakistani and Indian administered sides of Kashmir. This form of construction is also referred to in the Indian literature as “brick nogged timber frame construction”.

Dhajji dewari consists of an extensively braced timber frame. The relatively small space left between the framing is filled with a thin wall of stone or brick masonry laid traditionally into mud mortar. They are typically founded on shallow foundations made from stone masonry.

It is estimated that over 100,000 houses have been built using dhajji dewari in northern Pakistan and Pakistani Administered Kashmir since the 2005 Earthquake [1]. This is due to the method’s affordability, perceived ability to withstand earthquakes and the fact that it is within the technological means of many who had to self build to reconstruct their destroyed homes.

Figure 2-1 Typical dhajji dewari building

Unfortunately, university degrees in engineering rarely touch upon such forms of construction. Research into dhajji dewari buildings is nearly non-existent, design guides are limited and what guidance there is, is often based on anecdotal evidence and apparent common sense principles, which have not been validated through rigorous engineering testing and analysis. The challenge is to understand the structural behaviour of dhajji dewari buildings, as a first step towards considering whether it is possible to develop safe construction guidelines, and provide confidence in this technology.

The purpose of this research project was to apply state of the art engineering analysis to a typical dhajji dewari house, similar to those built after the 2005 Pakistan earthquake. The project aimed to establish whether the building type could be accurately modelled and in so doing determine how it theoretically performs when subjected to large earthquake loads. More specifically, such analysis allows us to test the sensitivity of the dhajji dewari construction system to the effect of ground shaking. Also, to establish which the critical engineering details are that ensures reliable seismic performance. Based on this it is possible to identify effective measures that could be implemented to enhance the performance when constructing new buildings or retro-fitting existing dhajji dewari buildings.

Figure 2-2 Typical dhajji dewari building under construction

3 Methodology

Since dhajji dewari is a building form reliant on complex interaction between multiple loosely connected elements, sophisticated analytical software is needed to have a reasonable chance of accurately modelling the structure. This is especially true given that guidance for simplified engineering modelling based on good quality research of this construction form is not yet known to exist.

As illustrated in Appendix A2.1, dhajji dewari buildings vary considerably in form, materials, quality of construction and structural detailing. Since dhajji dewari is built in a variety of configurations, the analysis has focussed on establishing the overall performance of the structure, then exploring the relative importance of specific aspects of the construction form.

The output from the computer model has been compared with a physical model in order to verify the technique used.

3.1 Choosing the Analysis Model

Choosing a representative building to analyse was challenging for a dhajji dewari house. However, in collaboration with UN-Habitat Pakistan it was decided to base the engineering analysis as closely as possible on the building layout shown below. This is a house that UN-Habitat Pakistan felt was reasonably representative of what was being built in the region after the 2005 earthquake.

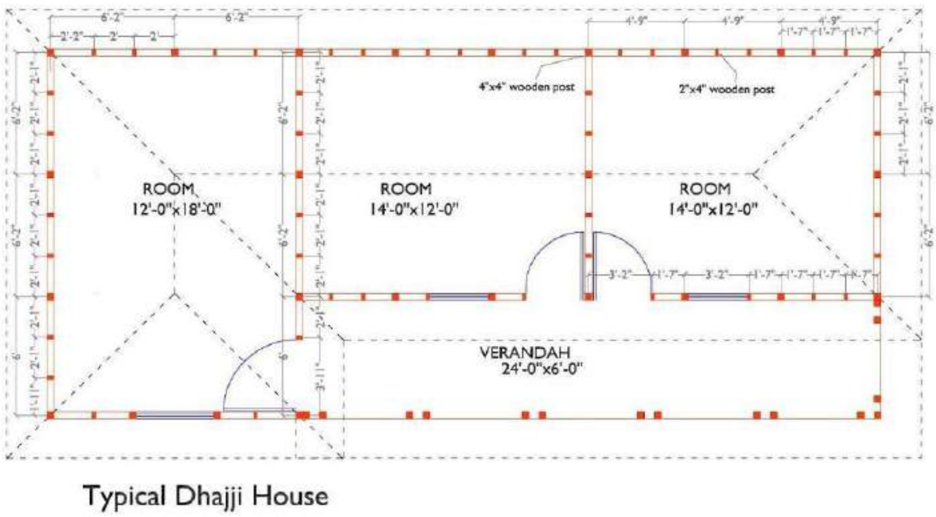


Figure 3-1 Typical plan view of an "engineered" dhajji dewari building after the 2005 Pakistan earthquake

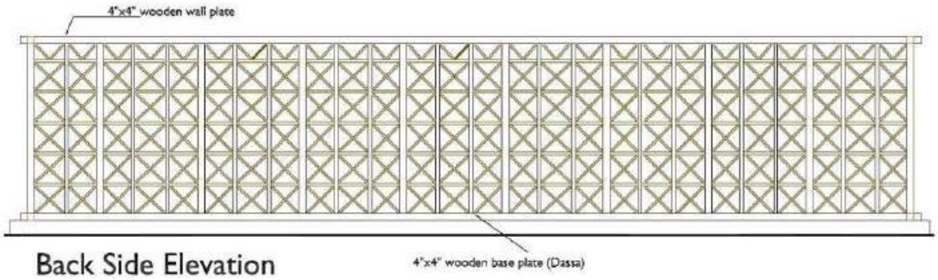


Figure 3-2 Typical rear elevation of an "engineered" dhajji dewari building after the 2005 Pakistan earthquake

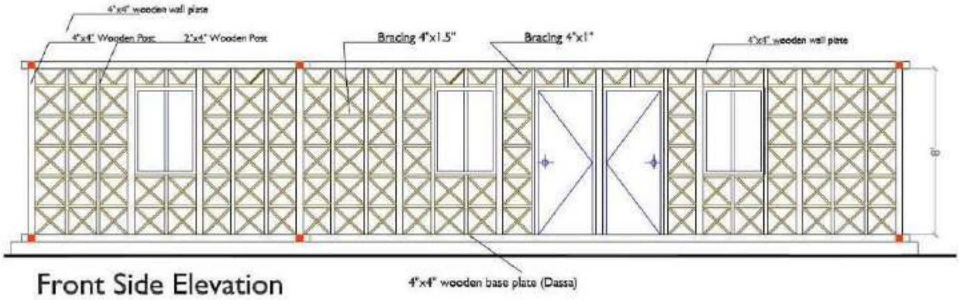


Figure 3-3 Typical front elevation of an "engineered" dhajji dewari building after the 2005 Pakistan earthquake

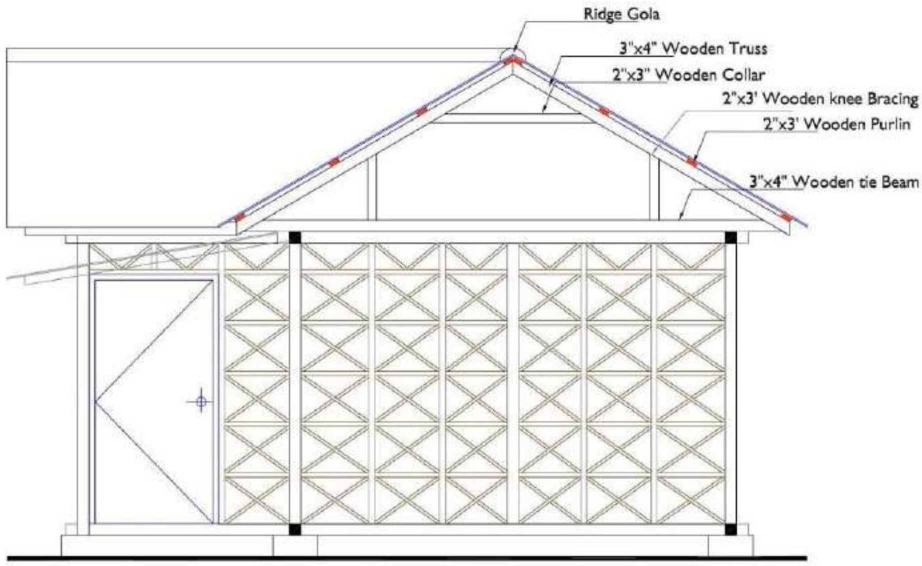


Figure 3-4 Typical side elevation of an "engineered" dhajji dewari building after the 2005 Pakistan earthquake

Note: The term "engineered" refers to the assumption that the building has been laid out and constructed using measuring equipment and that the main timber frame is aligned but no more. It is not thought that calculations and design checks have been undertaken to confirm the adequacy of the design.

3.2 Structural Analysis Software

The analytical element of this work has been undertaken by Arup's Advanced Technology and Research Group using LS-DYNA [30].

LS-DYNA is a multi-purpose explicit and implicit finite element program used to analyse the nonlinear response of structures. Its fully automated contact formulation and wide range of material models make it an attractive method for the solution of unusual problems.

Pre and post processing work was carried out using Arup's OASYS suite of programmes [31], [32], [33] in addition to Altair Hypermesh [29] for geometric construction of the model.

3.3 Structural idealisation

Design and analysis techniques for structures built from conventional modern materials (concrete, steel, glass etc) have been developed over many years. These allow engineers to predict with relative certainty a building's performance without having to explicitly model all the building elements. Dhajji dewari does not fall into this category, since it is constructed from non-engineered materials. By non-engineered it is meant that the components (timber section sizes and masonry shapes) are typically non uniform and that they are arranged differently from building to building. Its performance relies on the interaction of the various building elements and there is significant variation in building forms and quality of materials and workmanship between houses. In order to understand the structural behaviour and performance it is necessary to accurately and explicitly model the geometry, the various building elements, their material properties and the interaction between all the pieces.

The approach adopted was to model the timber frame and the masonry blocks as solid elements with contact surfaces between members to account for frictional behaviour.

The roof has been assumed to be clad in relatively lightweight CGI sheet. Historically timber and mud roofs were often built and more recently flat concrete roofs have sometime been adopted. The heavier roof structures (or floors) will have different dynamic characteristics which could induce larger lateral loads in the dhajji dewari walls. This is due to their large mass being excited by an earthquake which could generate larger horizontal inertia loads to be resisted by the structure.

However, the larger horizontal earthquake loads from the heavier roof structures might be offset by the vertical wall compression provided by the additional weight. This would increase the friction force between the masonry pieces. This hypothesis has been tested in sensitivity runs performed on the bench mark test frame.

The roof system (structure and cladding) used for this project is a conventional lightweight one and has been idealised as beam and shell elements to minimise computational demands.

A number of analytical assumptions were required as a result of computational demands, money and the available engineering time to develop the analysis models: Some of the key assumptions are:

- Friction between infill elements and at the interface of the infill and timber has been given a single coefficient. In reality, this value would vary with material but might also alter with time and duration of loading.

- Connections have been idealised, in most cases as discrete elements with non-linear material models. However, the approach of running parallel analyses, with and without ‘nailed’ joints has provided bounded results and demonstrated the importance of connection details; particularly for out-of-plane restraint.

- The flexible mud mortar has not been explicitly modelled

- The masonry infill has been assumed to be incompressible

- The timber has been modelled as an elastic material

The analysis to date has only considered a single storey dhajji dewari house. Different performance characteristics are expected from multi-storey structures. Detailed description of the LS-DYNA computer model is presented in Appendix D.

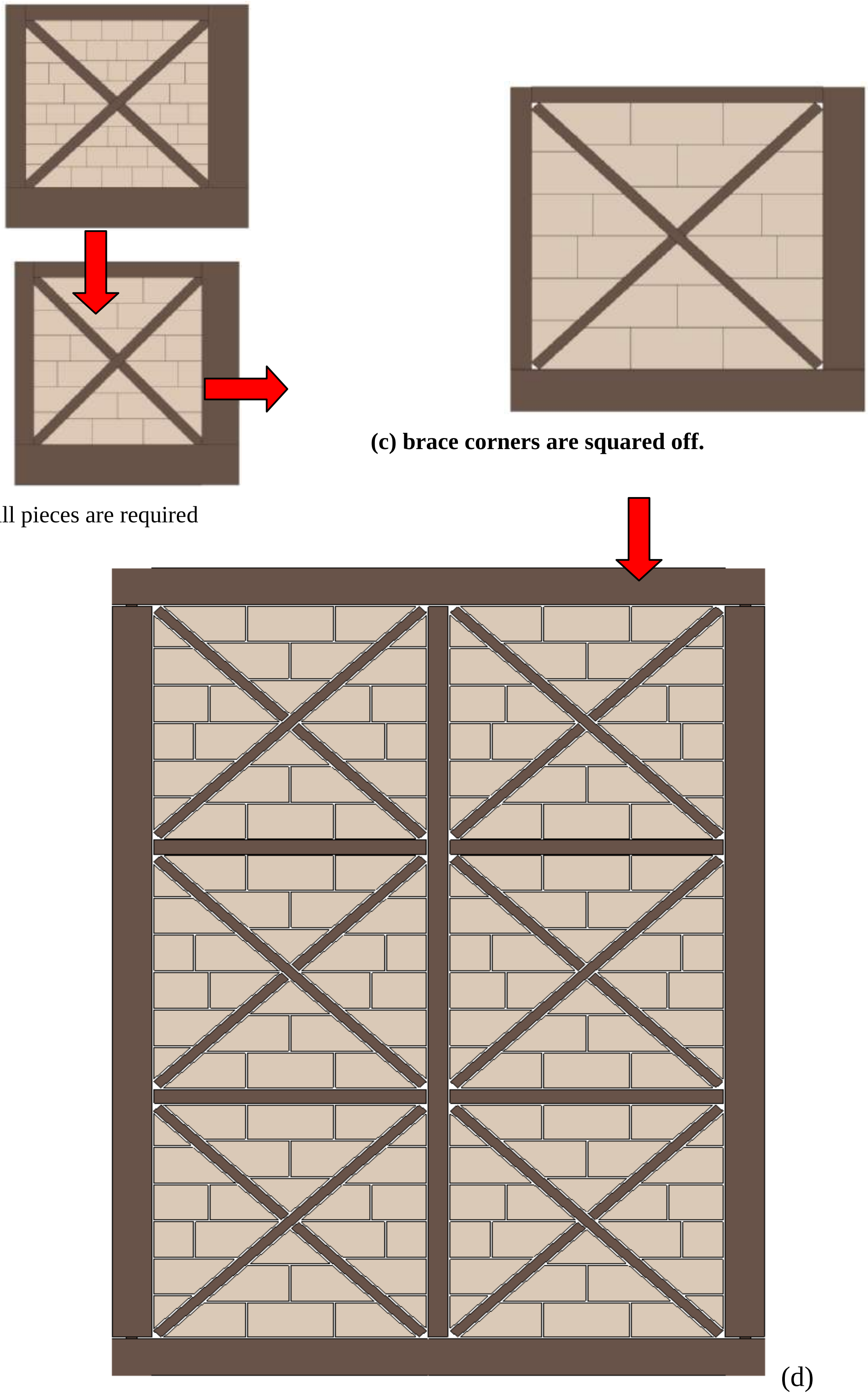
3.4 Model Evolution

The target of this work was to analyse the performance of an entire dhajji dewari building as shown in Figure 3-7. However, the complexity of the analysis dictated a gradual evolution of the model from a single panel (0.6 x 0.6m) (Figure 3-5), through a series of full height walls to a complete representation of the house including the roof structure and cladding (Figure 3-6 and Figure 3-7).

Numerical analyses must balance computational costs (run time, volume of data etc) with solution accuracy. Initial work on a single panel suggested a full scale analysis would be prohibitively long. Examinations of these early analyses indicated that the contact surface calculations dominated the computational time. To address this, the infill panel was remodelled with a coarser mesh, thus reducing the number of elements and hence reducing the contact calculation time. Yet, this only marginally improved the computational efficiency

As a further refinement of the model, the size of the masonry pieces were scaled up by 50 percent (note, timber section sizes were not altered). This allowed a full-scale dhajji dewari building to be constructed with half the number elements for analysis. Combined with economies of scale inherent in multi-processor finite element analyses, this reduced the run time substantially making the full building model analysis feasible. Even so, the processing time was still approximately 68 hours providing an indication of the complexity of the analysis.

(a) 52 infill pieces are required



(d)

Figure 3-5 Model evolution: a) small module/fine mesh b) small module/coarse mesh c) module scaled up 50% d) 2.24m high wall panel.

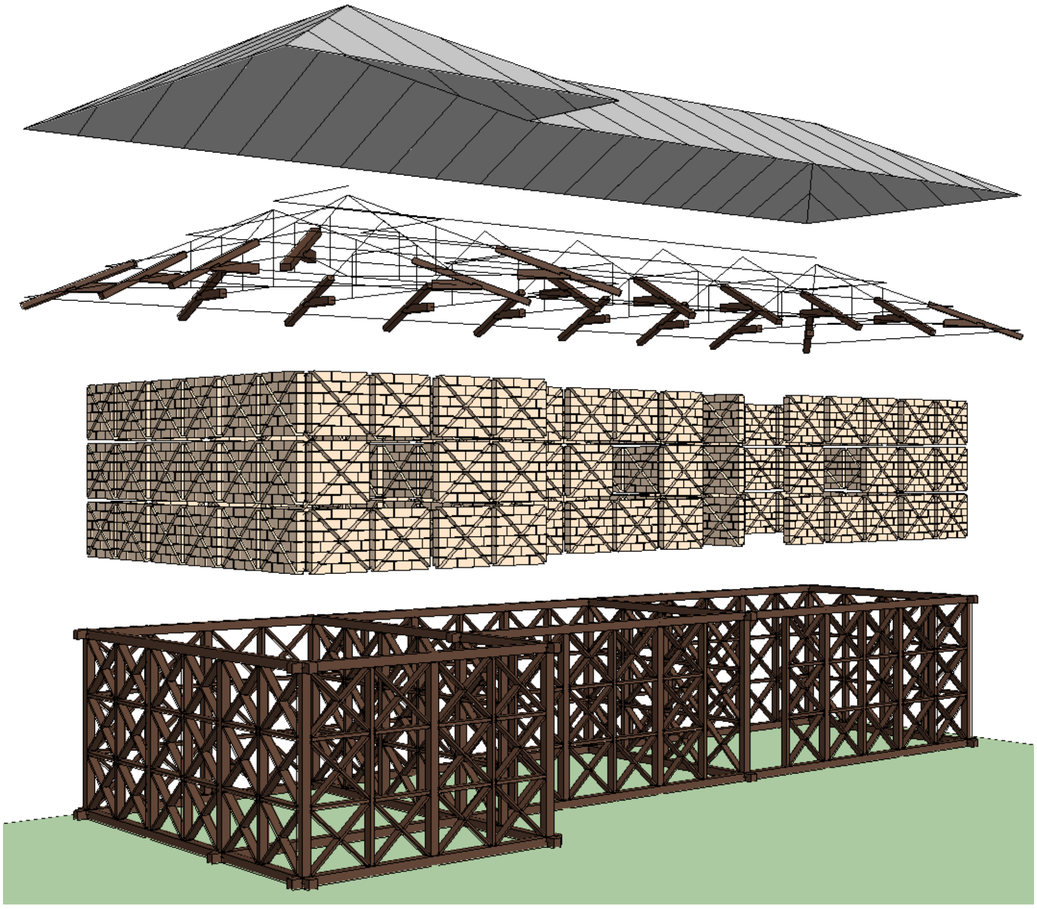


Figure 3-6 Exploded view of full dhajji dewari model

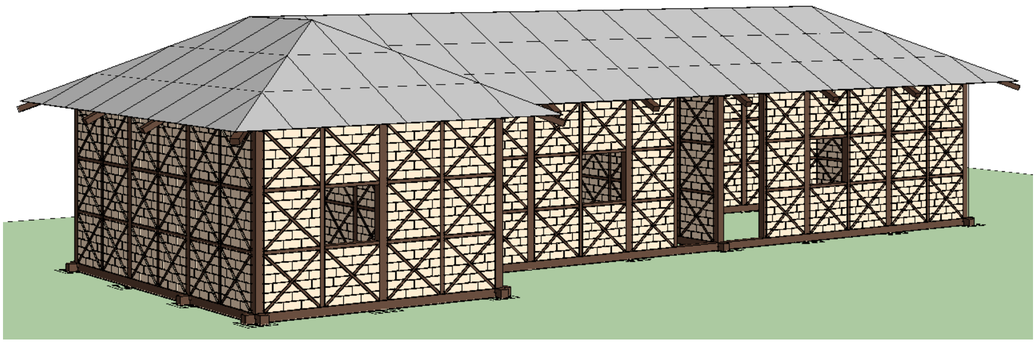


Figure 3-7 Full dhajji dewari model

In response to discussion on how these buildings behave it was felt to be useful to attempt to define what is meant by a wall, a wall panel, a wall panel segment and an infill segment. An attempt to do this is shown in Figure 3-8 and described below in words:

- A wall would be made up from one or more wall panels.
- A wall panel consist of a number of braced bay (wall panel segment) filled with infill masonry. For the purposes of defining the extent of the wall panel it is assumed to go between primary vertical timber posts.

- A wall panel segment consists of a braced bay and a number of these segments make up a wall panel. A wall panel segment will consist of a number of infill panels.
- An infill panel is a section of masonry wall that is surrounded by timber.

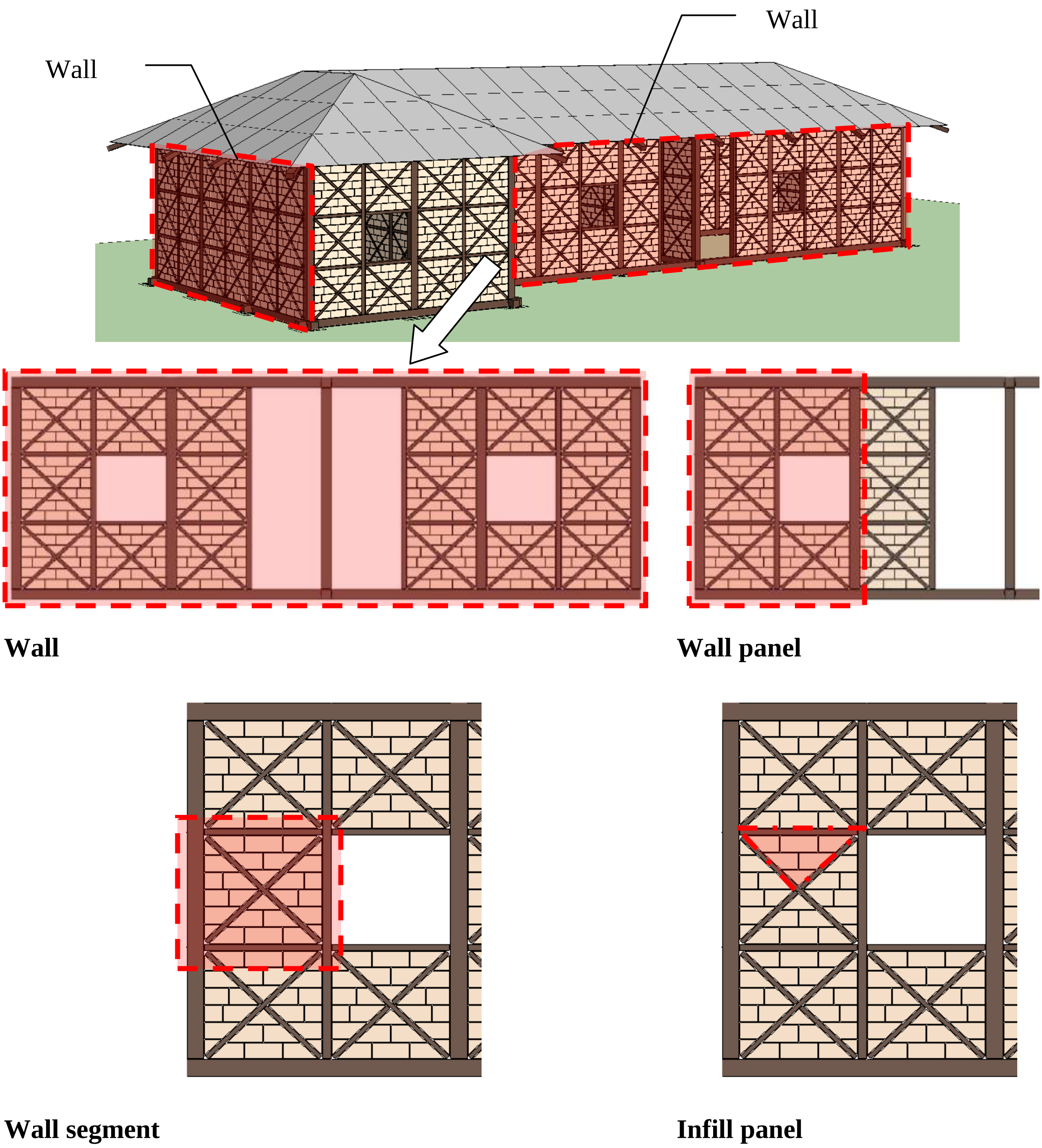


Figure 3-8 Terminology to help understand the wall build up

The above definitions need to be discussed and agreed so that there is clarity when describing the behaviour of dhajji dewari building. It is suggested that the following terminology is considered:

Wall	A wall is made up of multiple wall panels
Wall panel	A wall panel is a piece of wall between main vertical members and is made up of multiple wall segments.
Wall segment	A wall segment is made up of a complete brace unit and a number of wall segments will form to make a wall panel.
Infill panel	An infill panel is made up of masonry only and is bounded completely by timber

3.5 Model Verification

As with all numerical work, it is preferable to compare the computed results with data from physical tests. With the support of the Earthquake Engineering Centre in the Department of Civil Engineering at North West Frontier Province (NWFP) University of Engineering and Technology (UET) Peshawar, this has been made possible for the dhajji dewari analysis.

UET Peshawar carried out a series of quasi-static cyclic tests on two full scale wall specimens, loading them cyclically whilst recording the applied load and resulting displacements. The peak displacement was increased every three cycles to a maximum of 120mm. To simulate the load of the roof, a 200kg mass was placed at the top of each major column. Wall 1 was constructed well, with tightly packed infill material. The infill in wall 2 was loosely packed which required more mud mortar to fill the gaps.

To benchmark the analytical model, a cut down section of wall, 3.25m long, was produced. Whilst the overall dimensions vary slightly between the physical and analytical models, they were deemed sufficiently close to allow testing without building a model specifically for this purpose, see Figure 3-10 and Figure 3-11.

The difference in jointing techniques should also be noted. The analytical model assumes nailed connections for all secondary members (horizontal and diagonal braces). The physical model has mortise and tenon where main vertical posts connect to the bottom and top timber ring beams. Scarf joints are modelled where there are typically splices in long runs of timber. The roof trusses have joints between the inclined rafters and the horizontal truss beams.

The loading regime used by UET was reproduced in LS-DYNA (See Figure 3-9) and comparable displacement measurements taken. Lumped mass elements were also included to reproduce the 200kg masses applied to the test walls.

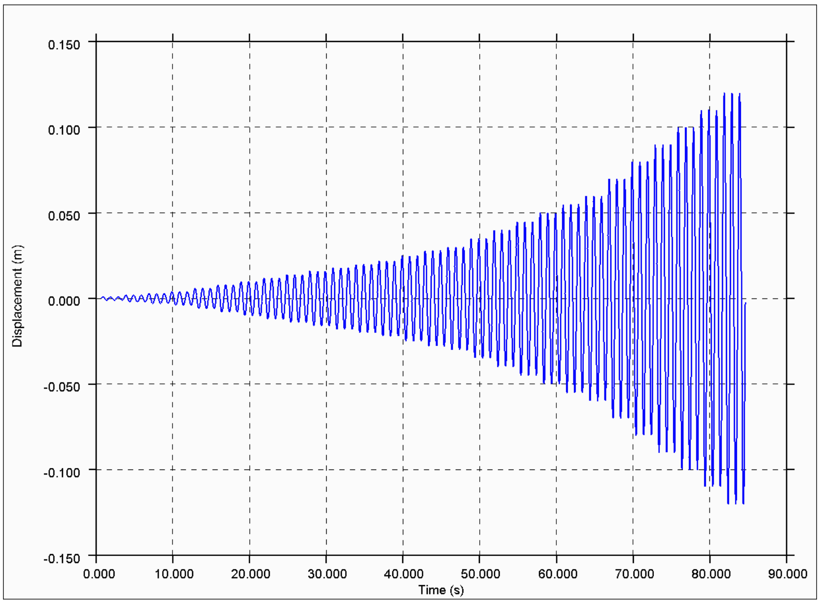


Figure 3-9 Cyclic push over analysis loading regime as used in the UET Peshawar tests.

It should be noted that we were unaware of the work at UET Peshawar until we had completed the computer analysis on the entire building.

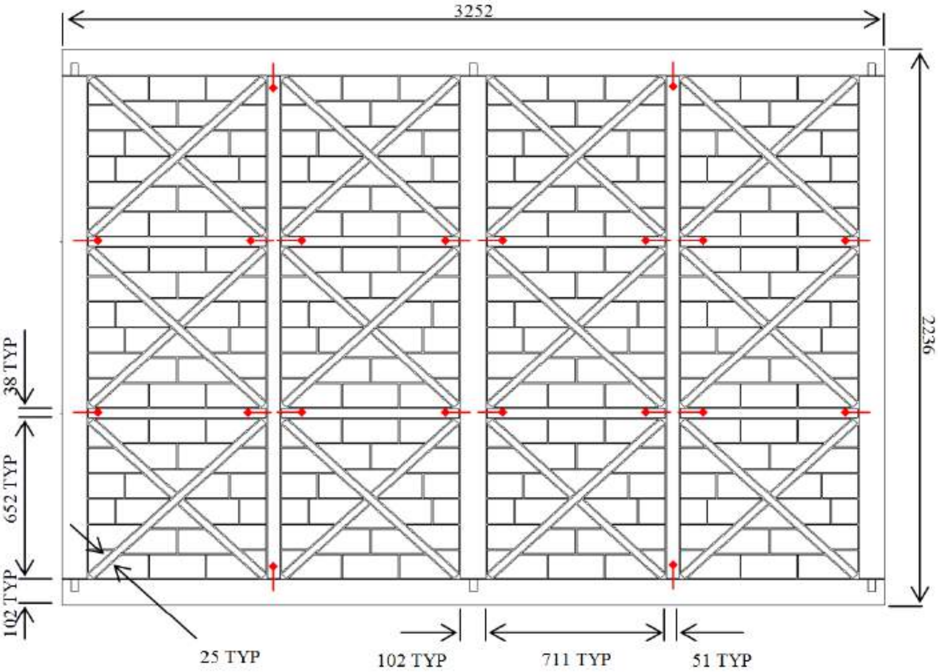


Figure 3-10 Analytical wall (measurements in mm)

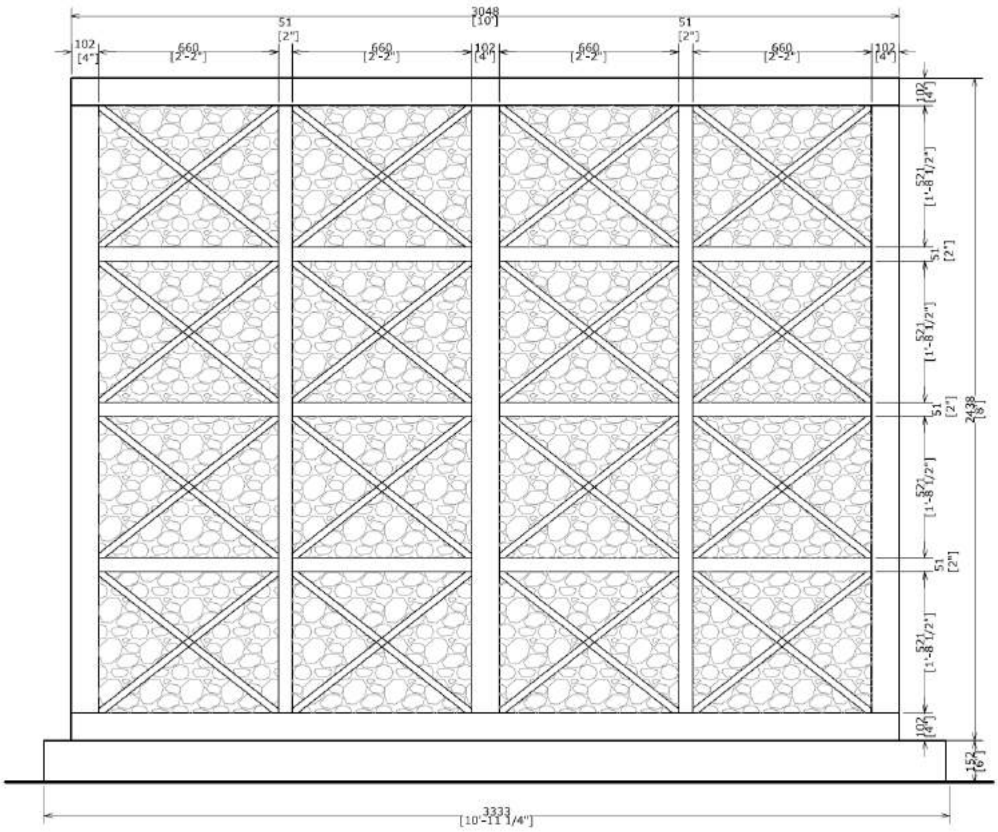


Figure 3-11 Physical test wall ©UET Peshawar [21]

4 Analysis

An earthquake is a sudden, rapid shaking of the earth caused by the release of elastic energy stored in rocks which has been accumulated over potentially hundreds of years. This release of energy is transmitted by seismic waves which spread and dissipate the released energy. Structures close to earthquakes are subjected to strong ground motions as they are hit by the seismic waves. Earthquake forces are generated by accelerations acting on a building's mass creating an inertia force. Generally a heavy building which has a large mass experiences larger seismic loads, than a lighter building.

During an earthquake structures are subjected to horizontal as well as vertical accelerations. The earthquake loads can be represented as equivalent lateral loads for static analysis, as a response spectrum to be used with modal analysis or an actual earthquake acceleration time history record for detailed analysis. One other method is to undertake a push over analyses whereby a building is pushed laterally to ever increasing levels of displacement until collapse. This 'pushover' method and the time history method were used to analyse the dhajji dewari construction in this project.

Brittle or very stiff structures do not have the ability to keep resisting seismic and gravity loads beyond their elastic capacity. They therefore tend to perform very poorly in earthquakes. Examples of brittle construction includes unreinforced masonry or poorly detailed reinforced concrete structures. Such structures are unable to absorb the seismic energy they are subjected to and collapse.

Ductile structures are able to keep resisting loads safely even after they have passed their elastic capacity. They are able to respond to earthquake forces through stable plastic yielding of key components which absorb the seismic energy. Such structures are inherently earthquake resistant.

Dhajji dewari buildings are typically relatively weak and flexible. This means that they will deform in a non linear manner even under low earthquake shaking intensities. As part of this work we want to determine if dhajji dewari is also a ductile structure that is able to resist many cycles of loading in a stable manner and at the same time de-tune its self away from the dominant earthquake frequencies by being so flexible.

Finally and importantly is a building's designed level of seismic performance. New homes built to appropriate seismic codes are designed to achieve the basic objective of "Life Safety". "Life Safety" is the post earthquake damage state which allows some damage to structural components but retains a margin against the onset of partial or total collapse of the structure. There is a lower level of performance called "Collapse Prevention" which is the post earthquake damage state that allows damage to structural components such that the structure continues to support gravity loads only but retains limited margin against collapse. A structure pushed to the performance point of "Collapse Prevention" is vulnerable to collapse due to aftershocks.

The purpose of the detailed analysis is to determine how dhajji dewari structures perform in strong earthquakes. The findings from this are presented in the next sections of this report.

4.1 Benchmark Analysis Results

The following figures compare the results of the LS-DYNA benchmark analysis model with photographs kindly made available by UET Peshawar of their test results. These illustrate that the model is broadly able to recreate the behaviour of the physical wall.

Figure 4-1 Deformation at 40mm top displacement

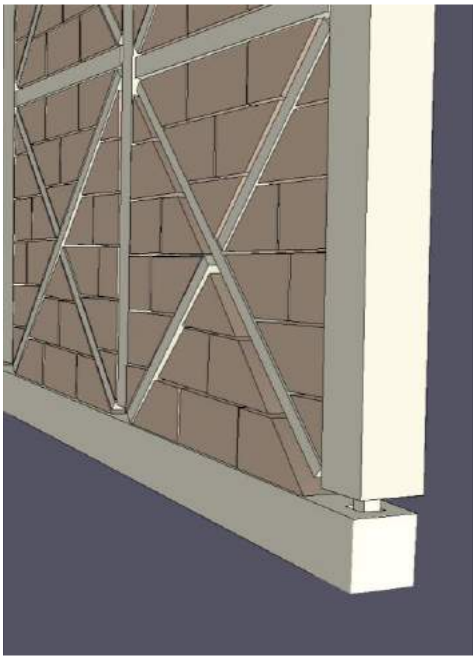


Figure 4-2 Deformation at 120mm top displacement

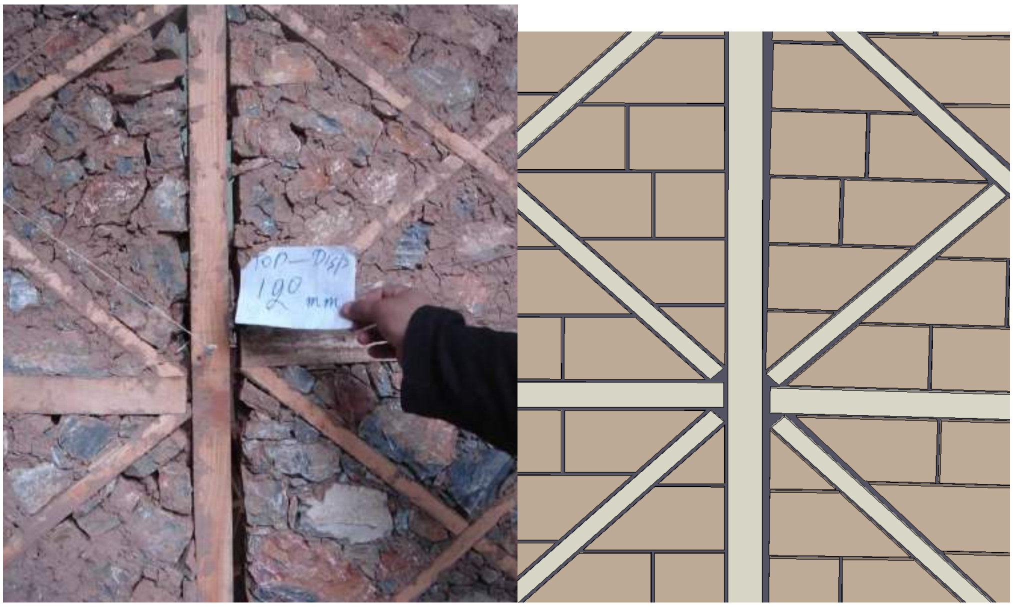


Figure 4-3 Secondary column at 120mm top displacement

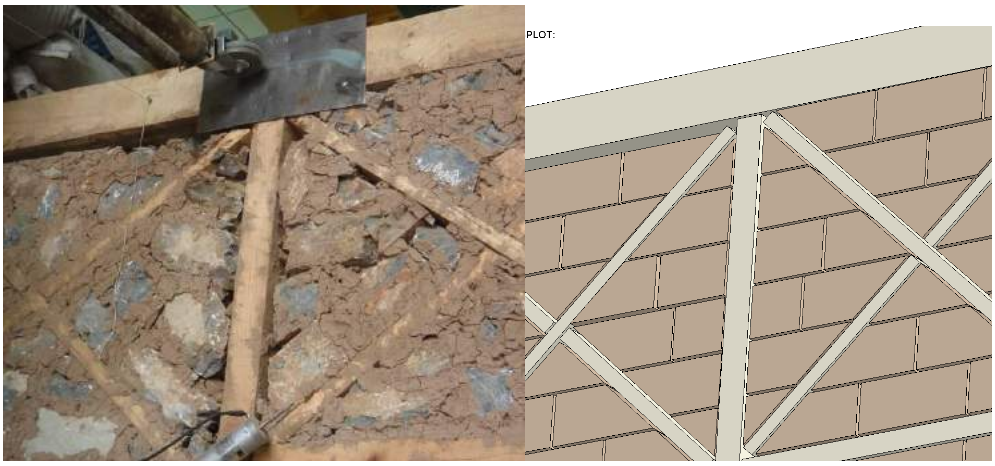


Figure 4-4Top of secondary column at 120mm top displacement

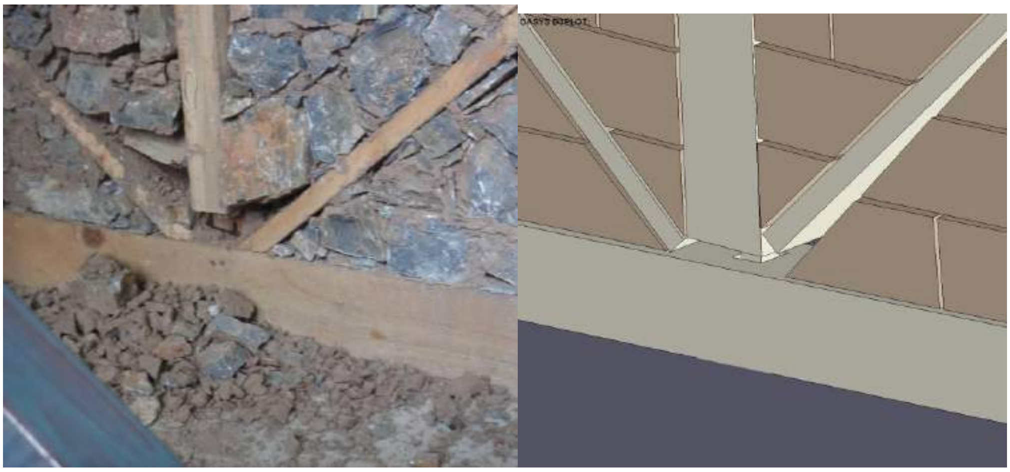
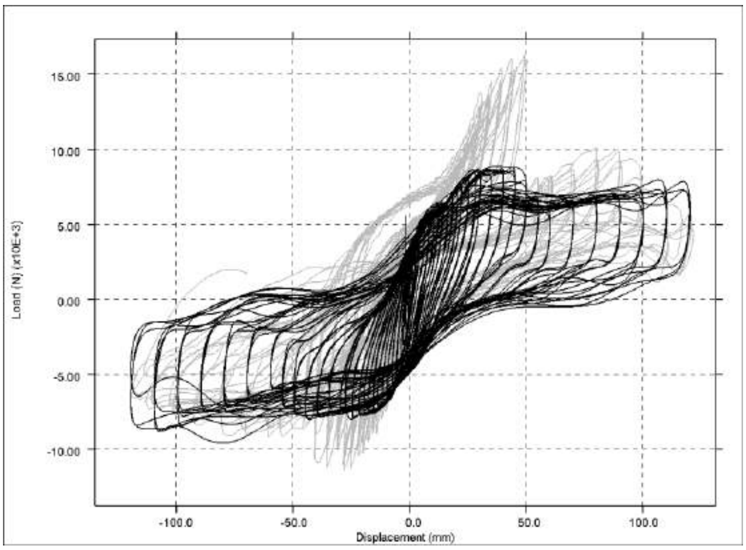


Figure 4-5 Base of middle column at 120mm top displacement

During the cyclic analysis, the horizontal base reaction was measured at the base of the frame and has been plotted against the applied displacement profile to produce hysteresis curves from the analytical wall model. These have been overlaid with the physical tests as shown in Figure 4-6 and Figure 4-7. The comparisons show that the LS-DYNA analysis model is able to reasonably predict the overall behaviour of the dhajji dewari frame made of timber, stone, mud and a few nails. Given that these models were conducted without prior knowledge of each other’s work the level correlation is considered good. It needs to be mentioned that the LS-DYNA model is not identical in terms of layout or the amount of nailing to the UET Peshawar test model. The UET test is initially stronger (by up to 50%) to start with. Post yielding, their physical test results and the analytical LS-DYNA model hysteresis curves are more closely matched. The stability of the system is better defined by the post elastic behaviour where there is broad agreement.

The UET Peshawar tests were conducted for two types of infill construction. One was tightly packed which means that the amount of mud mortar was minimised. In their other model the infill is loosely packed thereby requiring much more generous use of mud mortar between the stone pieces and between the stones and the timber frame.



LS-DYNA

Physical

Figure 4-6 Bench Mark Test: Wall 1 (tightly packed)

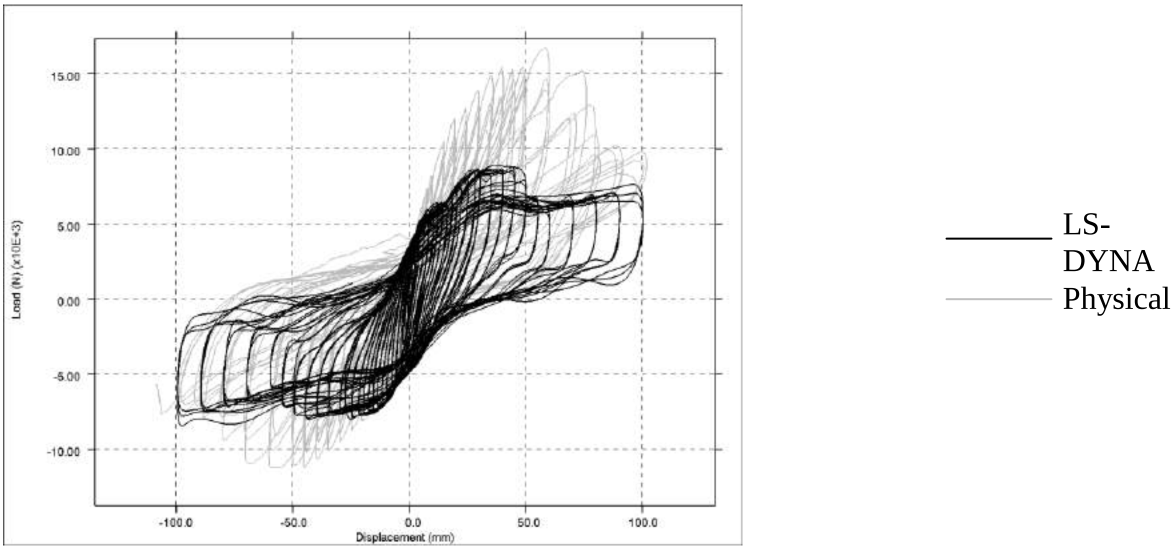


Figure 4-7 Bench Mark Test: Wall 2 (loosely packed)

4.2 Full House: Quasi-Static Nonlinear Pushover

Two quasi-static nonlinear pushover tests have been conducted in the long and short directions for the entire building. In each case, the pushover was carried out by pushing one end of the top ring beam at a constant rate horizontally and at the same time recording forces in the ground anchors to generate characteristic force-displacement curves for the building (see Figure 4-8). The buildings were displaced at a constant rate until they collapsed (seen by the drop off in the lateral resistance of the building). As can be seen from Figure 4-9 and Figure 4-10 the buildings were displaced laterally by over 1.0m over the one storey height of the building. The imposed lateral movement corresponds to a high storey drift. Such a drift level is very high and is probably more than what would be expected from the largest earthquake specified in the Uniform Building Code 1997 (UBC97, Zone 4 peak ground displacement at a building period of 1 second is approximately 200mm)

The quasi-static nonlinear pushover analysis allows a close examination of the building's failure mechanism under controlled conditions as shown in Figure 4-9 and Figure 4-10. The weakest parts of the structure yield and fail first. When this occurs load paths are constantly readjusted and the seismic loads are redistributed to other parts of the structure. Brittle structures will reach their collapse point quickly after the initial onset of yielding whereas well detailed and ductile structures exhibit significant post yield load resisting capacity.

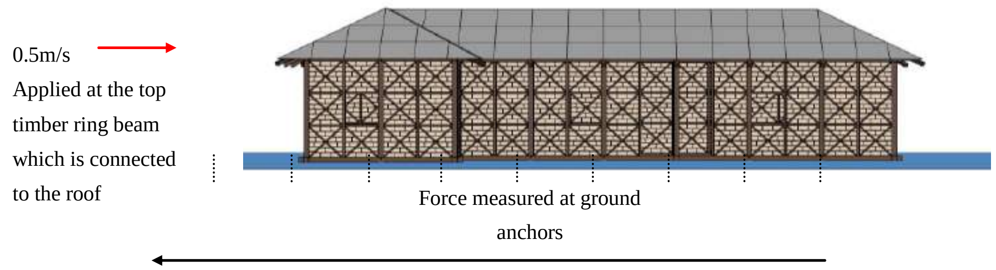


Figure 4-8. Configuration for pushover analysis

In both pushover directions the building behaviour shows the timber members becoming disconnected. This results in the confinement to the infill masonry being lost thereby precipitating the collapse of the building.

4.3 Quasi- Static Pushover Analysis Results

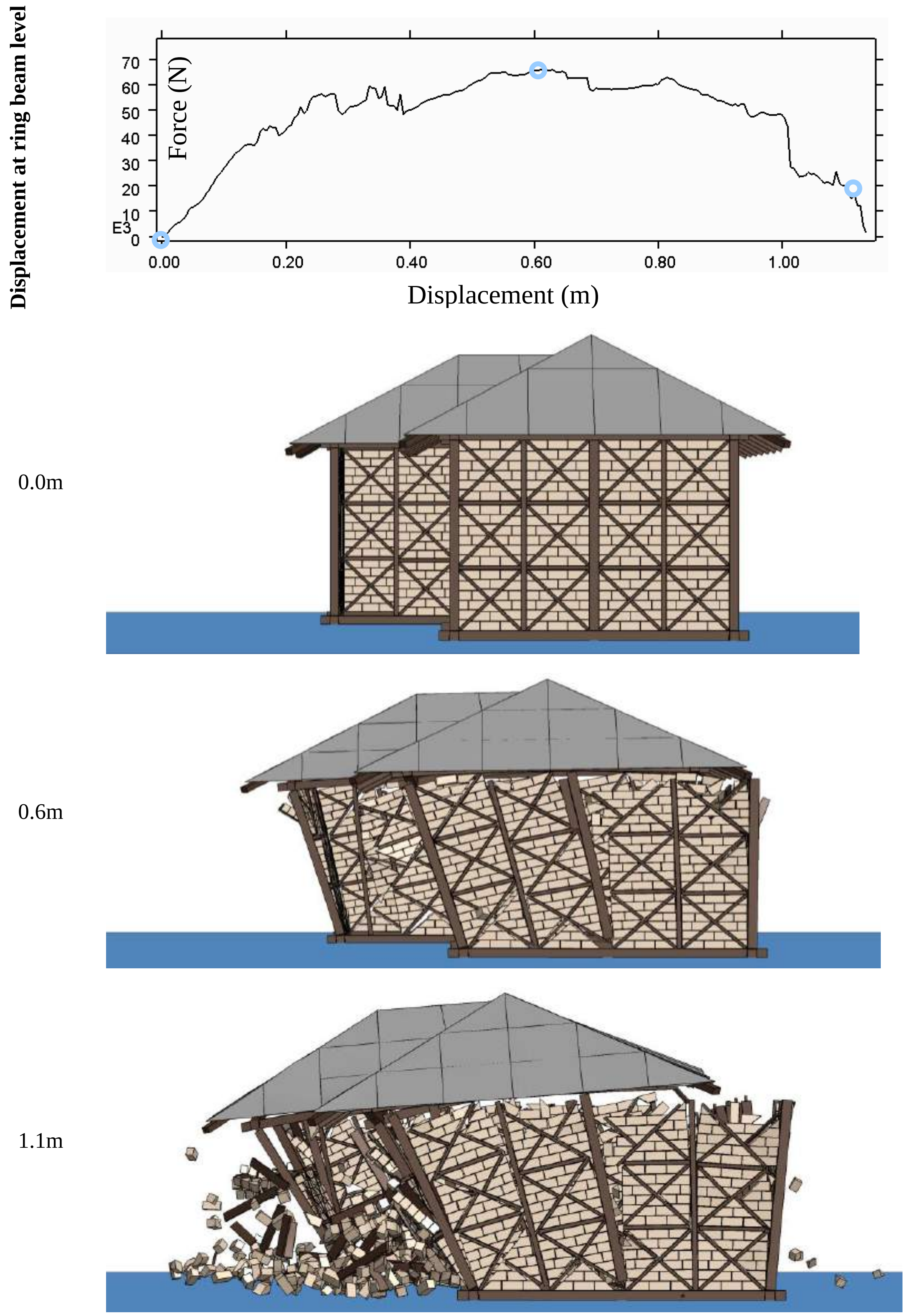


Figure 4-9 Pushover analysis: short direction

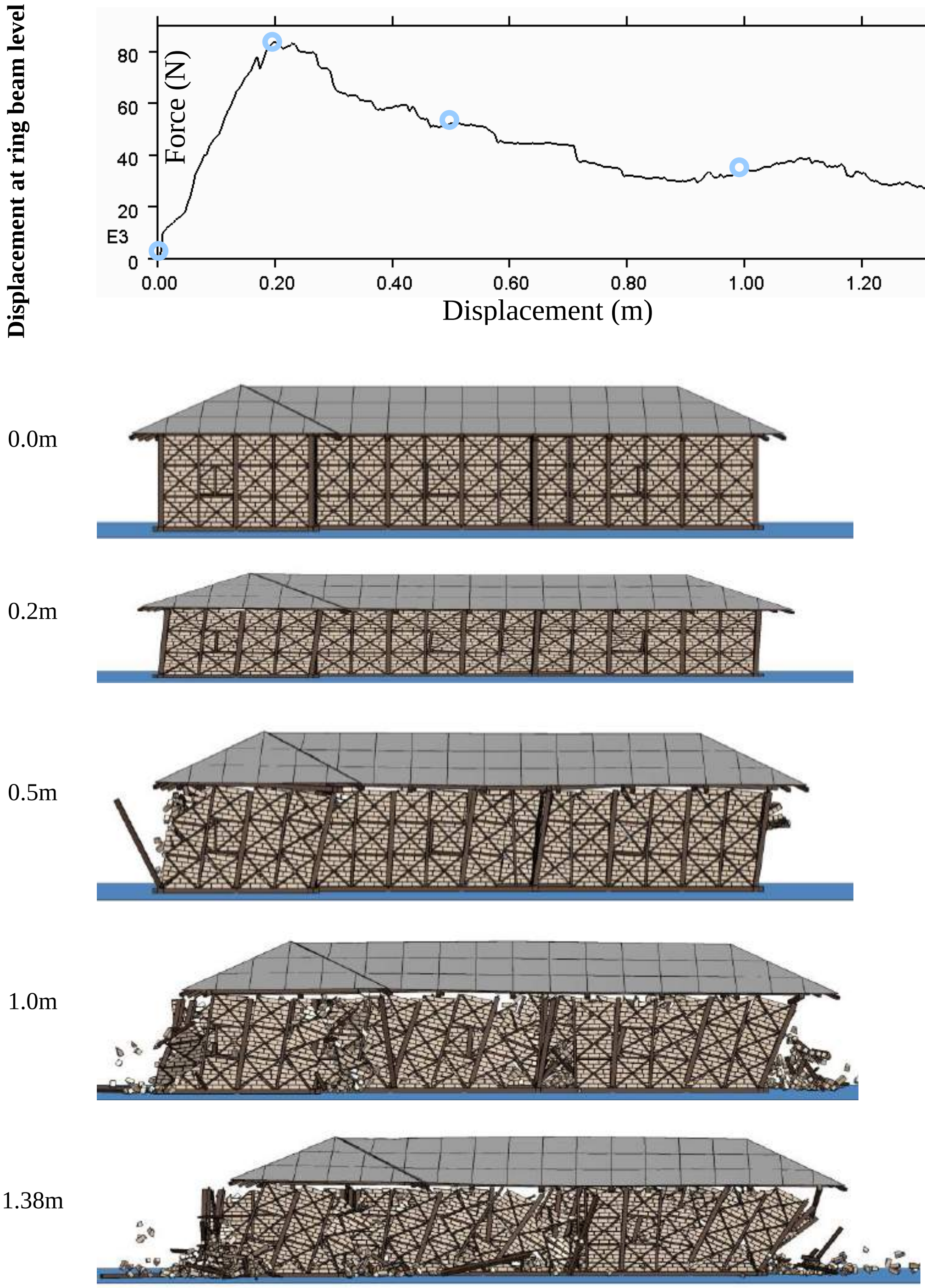


Figure 4-10 Pushover analysis: long direction

4.4 Full House: Time History Analysis

Lateral and vertical seismic actions were applied explicitly to the model as ground motions in the time domain to model the behaviour of the building under real earthquakes.

A series of spectrally matched time histories were produced using RSPMatch2005, which modifies real recorded ground motions in the time domain to match a target response spectrum. This can provide a match of a code spectrum, without removing the characteristic features of the original ground motion such as energy and phase content which were recorded during real earthquakes. Further details on the selection of the earthquake time histories are presented in Appendix D7.

4.4.1 Analysis cases

Two sets of time-histories have been analysed; PEER 1161 (without near source) and record PEER 828 (with near source). The near source record accounts for exceptionally large earthquake loads due to very close proximity to the faults in the ground. For each record two cases were considered. One with nailed connections between the horizontal and vertical members (as per section D1.3.3) and the other with the nails removed. This was intended to provide bounding information regarding the significance of some nailed connections in dhajji dewari construction.

4.4.2 Results – PEER 1161

For the time-history without near source effects neither the dhajji dewari model with nails, nor the one without, exhibited significant deformation (see Figure 4-11 and Figure 4-12). After the first 13 seconds, the building had deformed into a certain position and pretty much remained there. There was no recovery towards the original position due to gravity or further earthquake motions as most of the energy in the earthquake had passed and therefore very little additional local deformation occurred beyond this point (see Figure 4-13 and Figure 4-14).

Examining the energy content of the applied time histories (using $\frac{1}{2} mv^2$) and checking this against the kinetic energy output from the LS-DYNA computer model (Figure 4-15), it is clear that beyond 13 to 14 seconds hardly any additional energy was put into the system. The result is a plateau in the sliding interface energy, indicating that the contact surfaces (representing the mud mortar, timber and masonry interfaces) dissipate energy through friction losses during the earthquake (see Figure 4-16 and Figure 4-17).

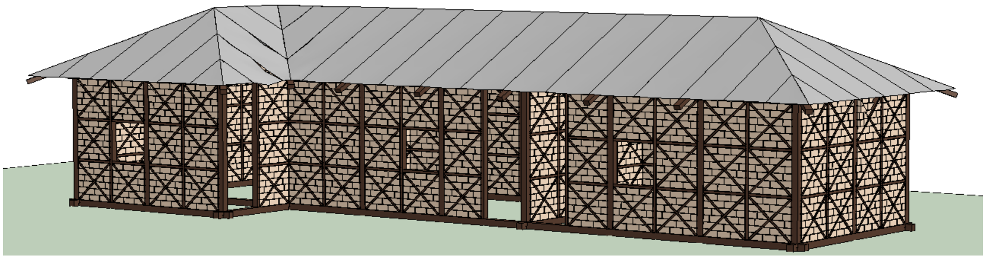


Figure 4-11. Condition of building after PEER 1161 analysis (with nailed connections)

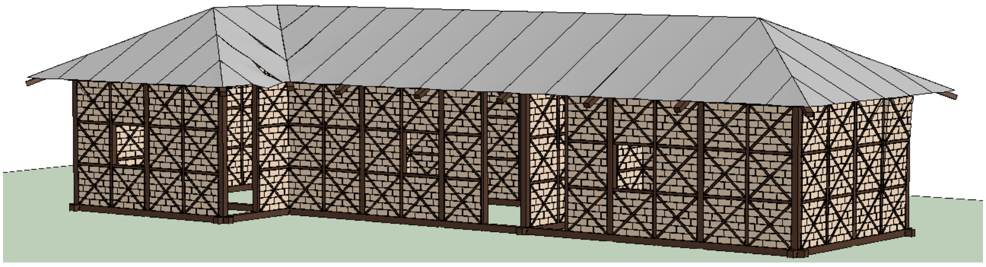
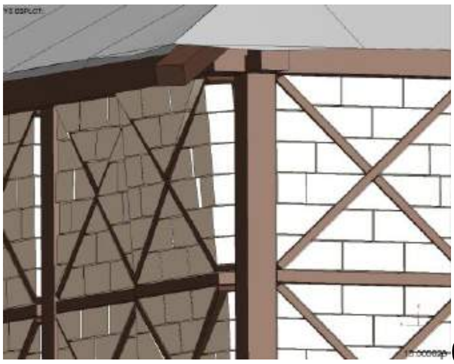
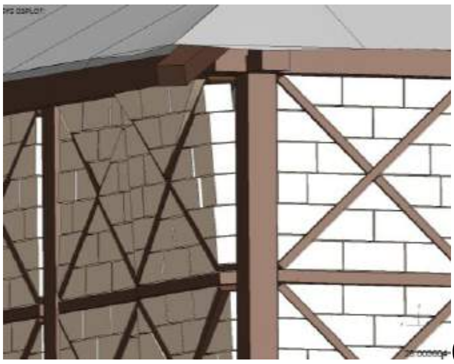


Figure 4-12. Condition of building after PEER 1161 analysis (without nailed connections)

In both the nailed and without nails models, mortise and tenon and scarf joints are present.



(a)



(b)

Figure 4-13 Local condition of the building after 13 seconds

Figure 4-14 Local condition of the building at the end of analysis (28seconds)

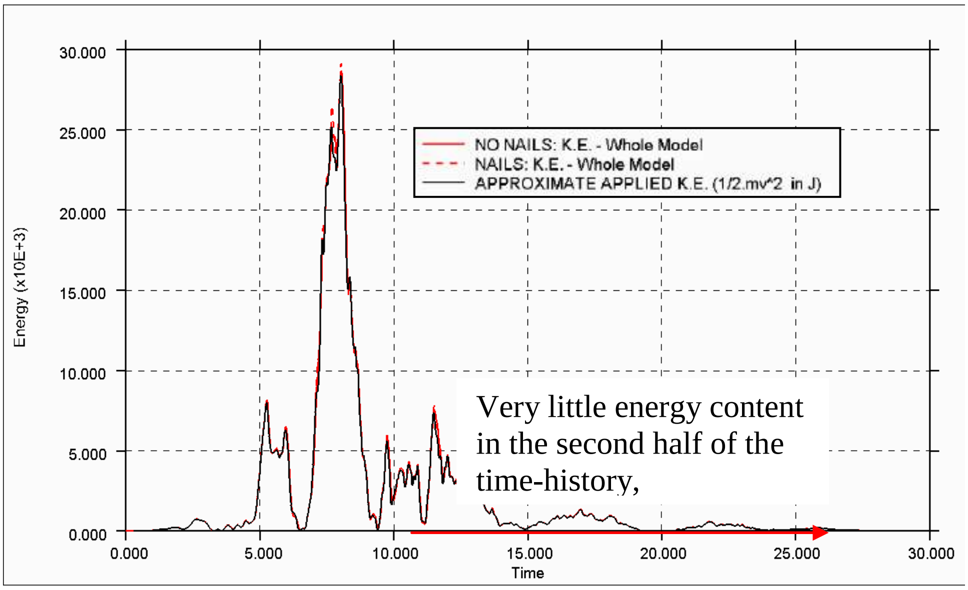


Figure 4-15 Comparison of kinetic energies

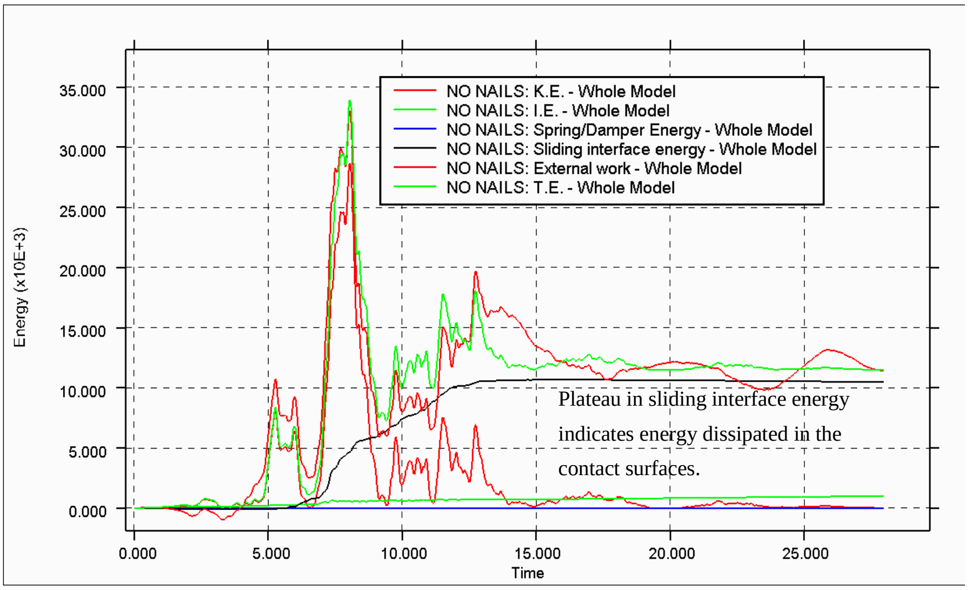


Figure 4-16 Energy plot for Time history analysis (PEER1161) without nailed joints

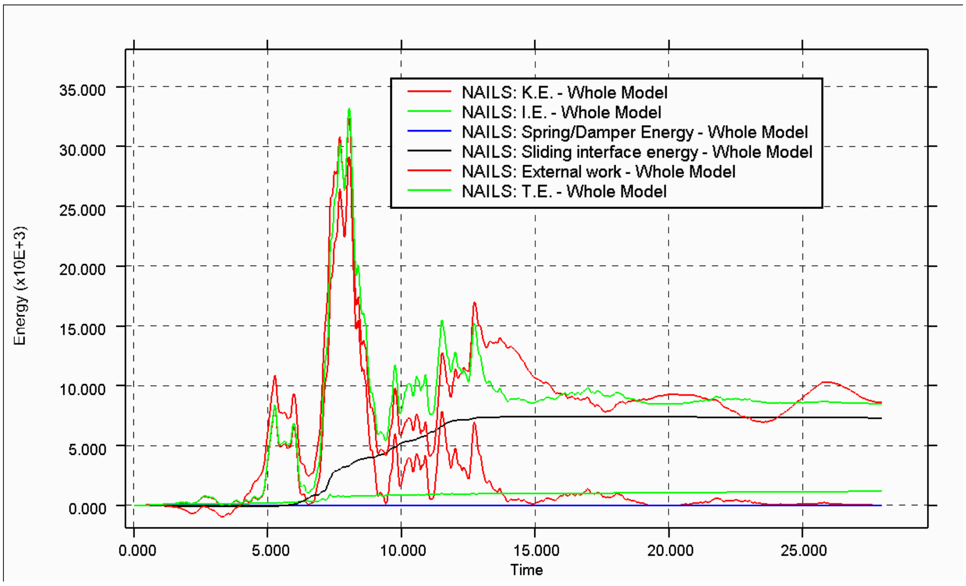
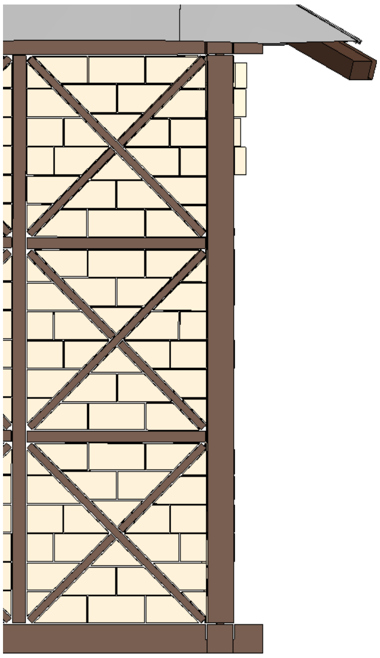


Figure 4-17 Energy plot for time history analysis (PEER1161) with nailed joints

Whilst neither case sustained catastrophic damage, the presence of nailed connections had a significant beneficial impact on the building performance. The most notable aspect of this was the out of plane behaviour of the dhajji dewari walls. In the un-nailed case, one of the end walls is on the brink of collapses and would be vulnerable to aftershocks. The additional deformation can also be seen in the energy plots; in the case without nails the sliding interface energy increased by approximately 40% in comparison with the nailed version (see Figure 4-16 and Figure 4-17). The ductility and strength provided by the nails ensured the timber frames remained around the masonry thereby ensuring good confinement to the unreinforced masonry.



a) With Nails **b) Without Nails**

Figure 4-18 Comparison of out of plane deformation (end wall)

4.4.3 Results – PEER 828 (including near source effects)

The near source effects make this time-history the more onerous of the two; this can be seen clearly in the peak base forces which are developed during the analyses (Figure 4-19). The peak forces from each time history are lower in the case without nails. The structure without nails is more flexible and therefore has a longer time period thus attracting lower accelerations from the earthquake records as well as having a lower strength capacity.

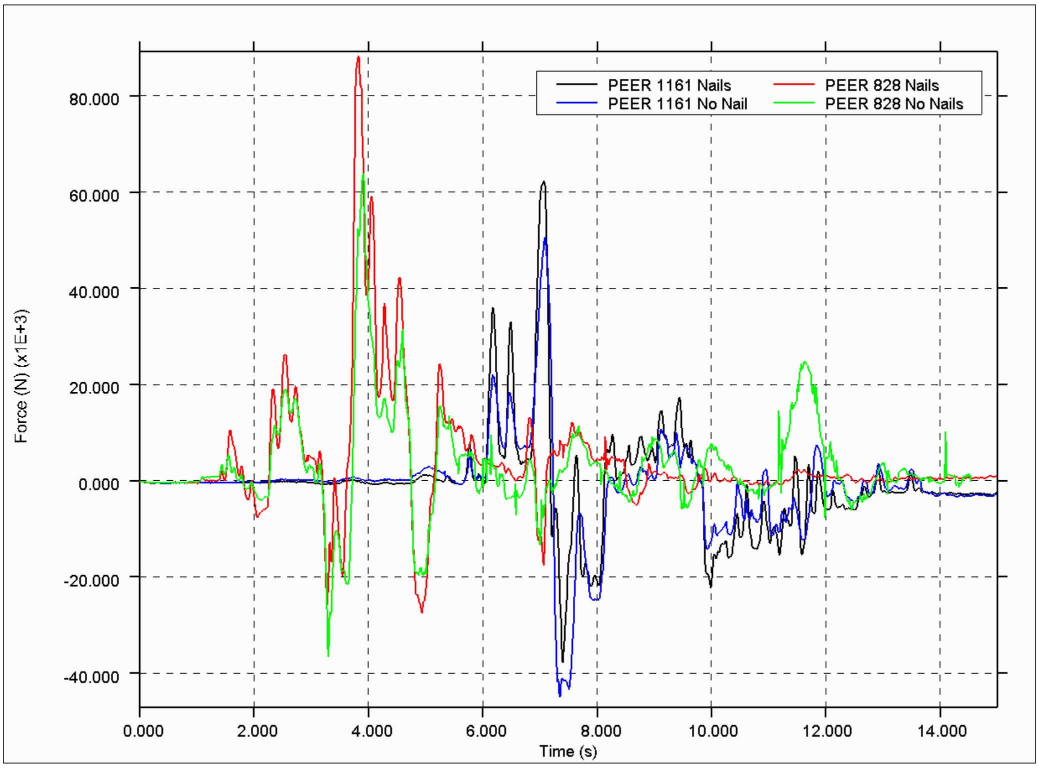


Figure 4-19. Resultant forces at the base of the model

As anticipated, the PEER 828 analyses produce higher levels of damage; see Figure 4-20 to Figure 4-23. There is a distinct difference between the performance of the models with and without nails, largely resulting from out of plane failure of the short walls in the case without nails. In these locations, failure is initiated when infill at the top of the wall is dislodged (Figure 4-24 and Figure 4-26); precipitating complete failure of one of the end walls (Figure 4-25). The infill is only able to fall out when the timber pieces confining the infill pulls away from the rest of the frame because of the out of plane inertia force exerted on the timber by the infill. Nailed connections help keep the timber frame together which enables greater levels of confinement to be maintained on the infill material. Hence greater over all structural stability is ensured.

The model with nailed connections survives the earthquake with only minor local damage (Figure 4-21) because it is able to maintain confinement to the infill. The confinement given by the timber frame to the infill is reduced as the timber frame comes apart in the model without the nails.

Nailed Connections **Without Nailed Connections**

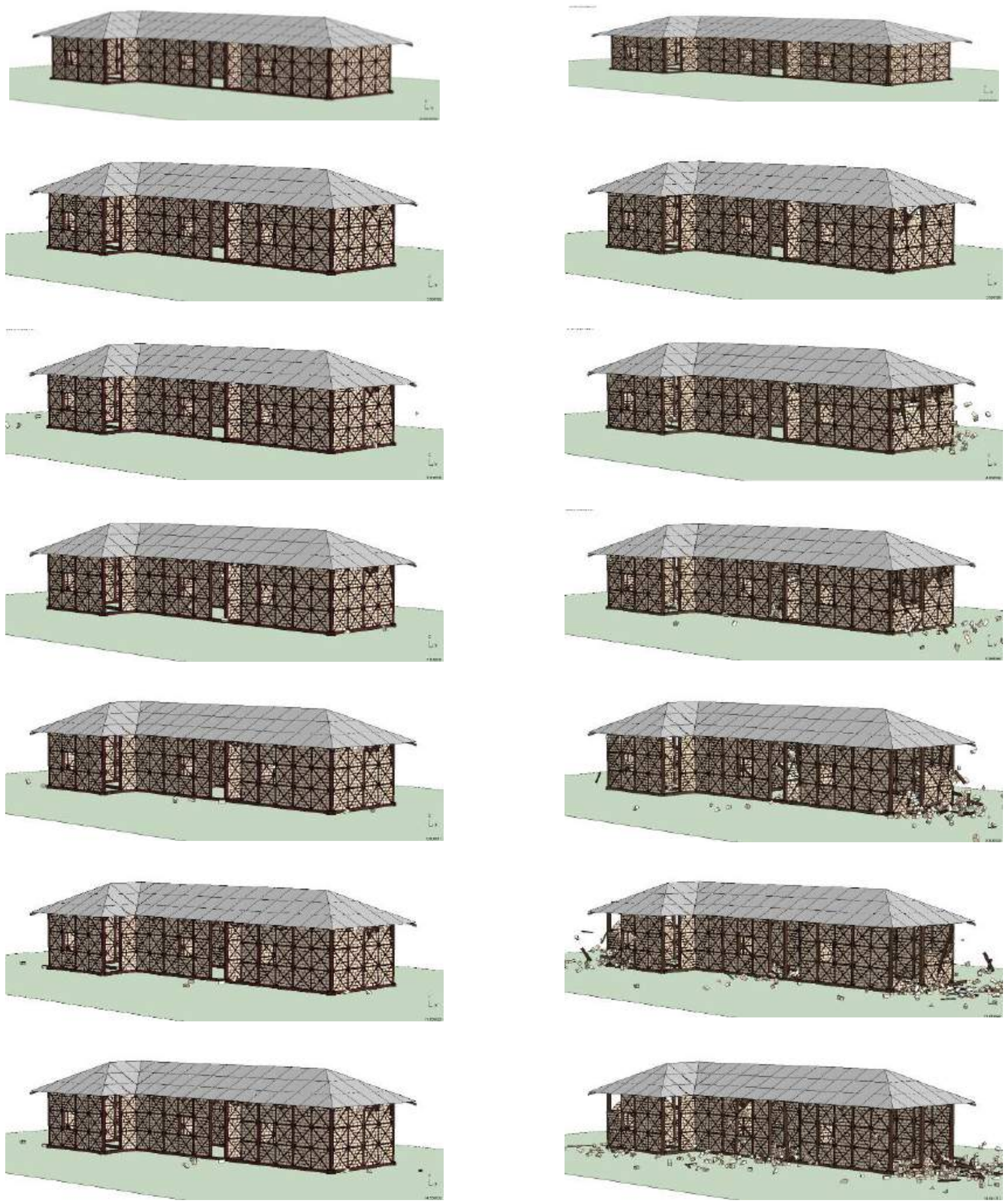


Figure 4-20. PEER 828 time-history analyses (with and without Nails)

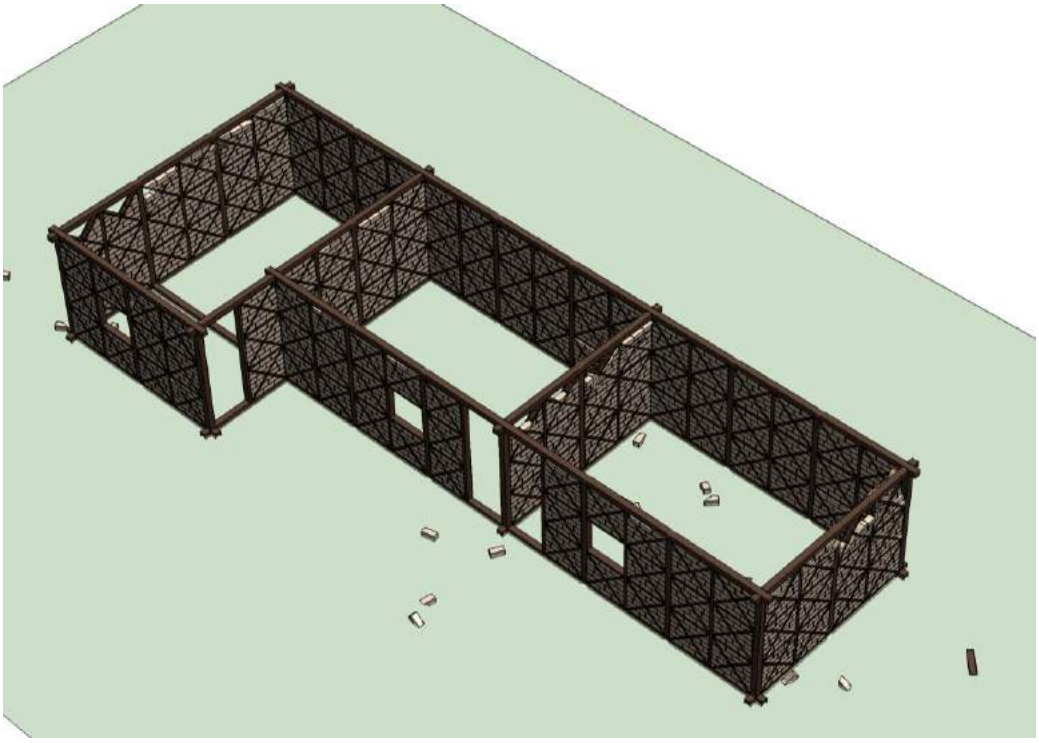


Figure 4-21. Damage to walls of Dhajji with nailed connections (PEER 828, tri-directional earthquake, See Appendix D7). (Roof not shown for clarity)

Figure 4-22. Damage to walls of Dhajji without nailed connections (PEER 828, tri-directional earthquake, See Appendix D7). (Roof not shown for clarity).

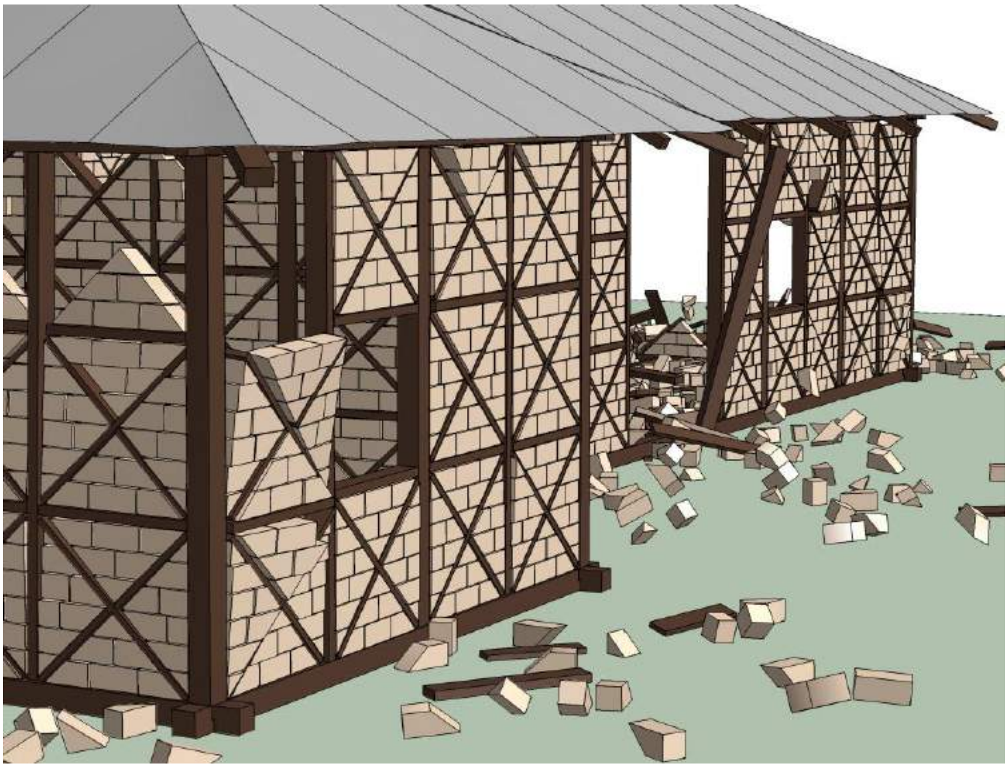


Figure 4-23. Significant damage sustained during PEER 828 analysis (without nails)

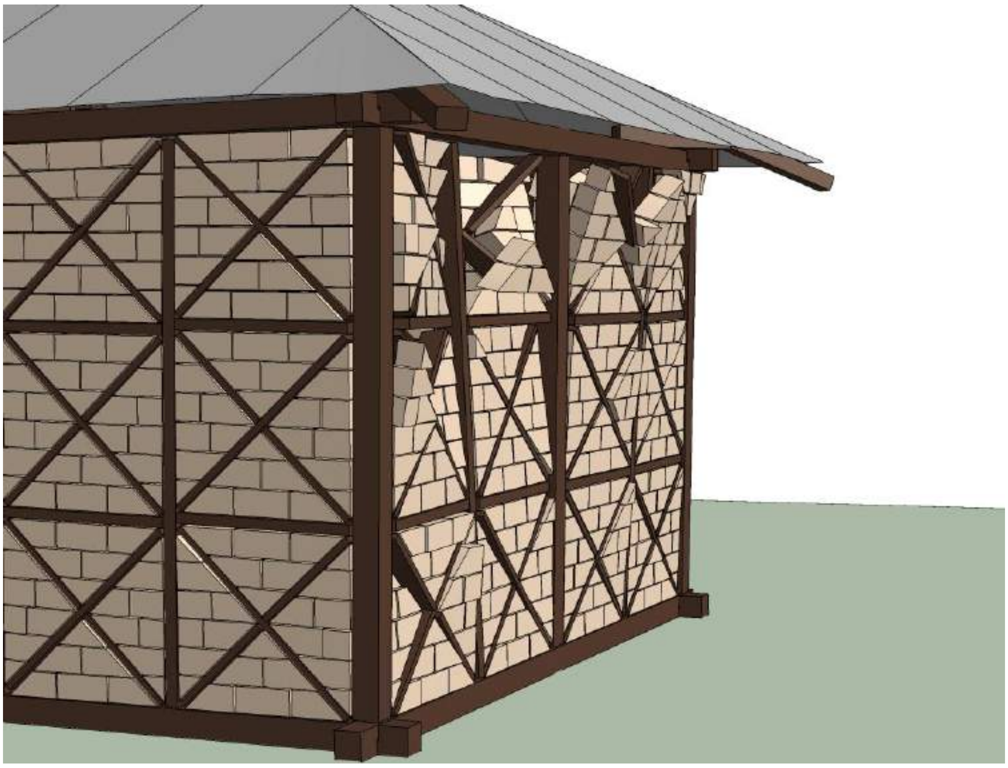


Figure 4-24. Onset of failure occurs at the top of end walls

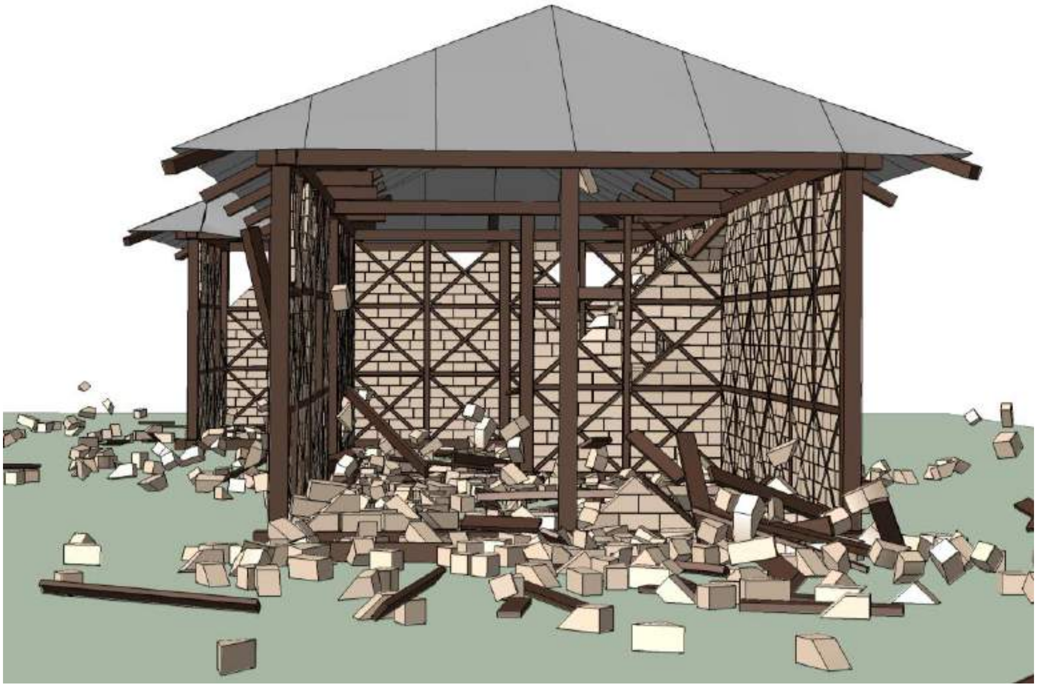


Figure 4-25. Once failure has initiated end wall collapses completely

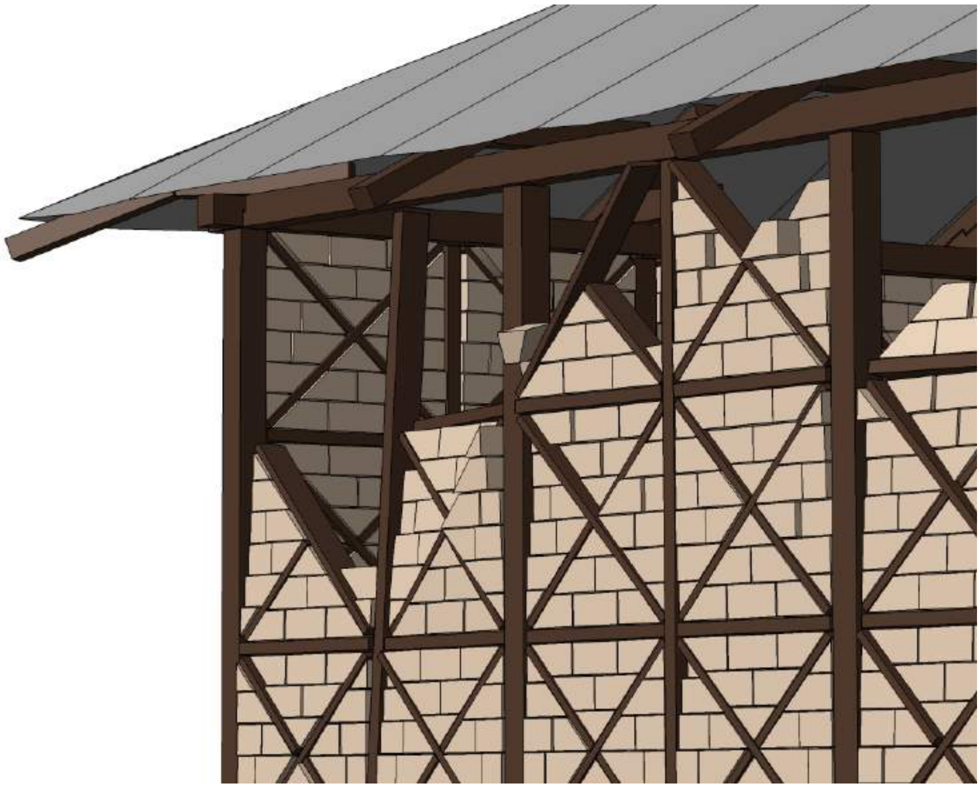


Figure 4-26. End wall damage

4.5 Sensitivity runs on sub frames

The sensitivity of the seismic performance of the building system has been further evaluated by carrying out quasi static cyclic pushover analysis on the benchmark test model shown in Figure 3-10. There are many parameters that need to be investigated to complete our understanding of the construction form. As a starting point, the effect of additional over burden and the effectiveness of braces were chosen as elements for further investigation. The same displacement controlled loading as shown in Figure 3-9 was used for all these runs to enable comparisons between the results.

It was envisaged that the increased levels of over burden, representing extra compression on the masonry walls from additional floors above, would increase the friction force required to yield the structure as the masonry pieces tried to slide past each other. In this case, larger amounts of energy could be absorbed by the walls enabling them to withstand larger seismic forces.

The second parameter to be investigated was the effect of making the timber diagonal braces less effective, by shortening them by increasing amounts, to account for lack of fit from the original construction, the effects of timber shrinkage with time and ultimately indirectly mimic a frame without diagonal braces and thus avoiding the need to re-mesh the analysis model. This would allow the quantification of the behaviour of the frame without braces all together. However, it does need to be recognised that the assembly will require a minimum level of lateral stiffness that is either achieved through timber bracing or the infill material or more likely a combination of both to resist more frequent loads such as wind loading.

Finally combinations of overburden and lack of fit of the braces was investigated.

4.5.1 Corner Connections

During preliminary runs of these sub frame sensitivity models we found that the timber frames connections at the corners, as shown in Figure 4-27, were failing. This behaviour leads to the frames opening up and being unable to confine the masonry infill. This subsequently resulted in premature collapse of the sub frame which undermines the otherwise stable behaviour of the structural system.

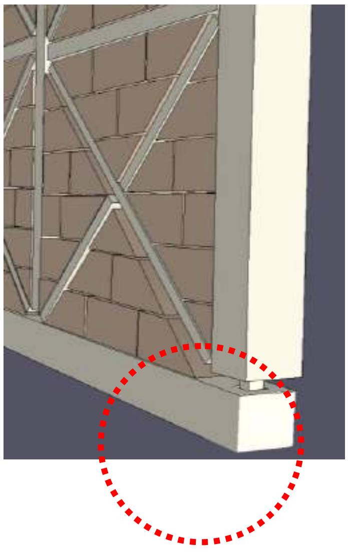


Figure 4-27 Potential source of weakness at the corners of the timber frame construction

Discussions with Randolph Langenbach suggest that such failures at the frame corners have not been observed in the field. Whilst there is compelling evidence after the 1999 Izmit earthquake in Turkey that traditional buildings performed better than modern reinforced concrete frame buildings (See Appendix B2) the presented data does not go into sufficient detail to be able to point to the observed failure mechanisms in dhajji dewari buildings. Further field studies should be undertaken when the opportunity for it arises to properly document the observed failures in dhajji dewari buildings after earthquakes by Earthquake Engineering Field Investigation Team (EEFIT), Earthquake Engineering Research Institute (EERI) or similar post earthquake field missions.

For the purposes of this work it was decided to make these connections strong (i.e. unbreakable) in the analysis models so that the behaviour of the rest of the frame could be evaluated for the complete loading cycles they were being subjected to. From a practical engineering perspective we are saying that these connections will need to be so detailed that they do not fail. This could be achieved by the strengthening of the corners by the addition of timber blocks around the connection and/or strategic strengthening using nails, screws or metal straps/plates. Clearly a better understanding of the demands and the local behaviour around the connections is required.

Before proceeding the verification model was rerun with the strong corners. This demonstrated the seismic response of the assembly was minimally affected by this change in modelling assumption.

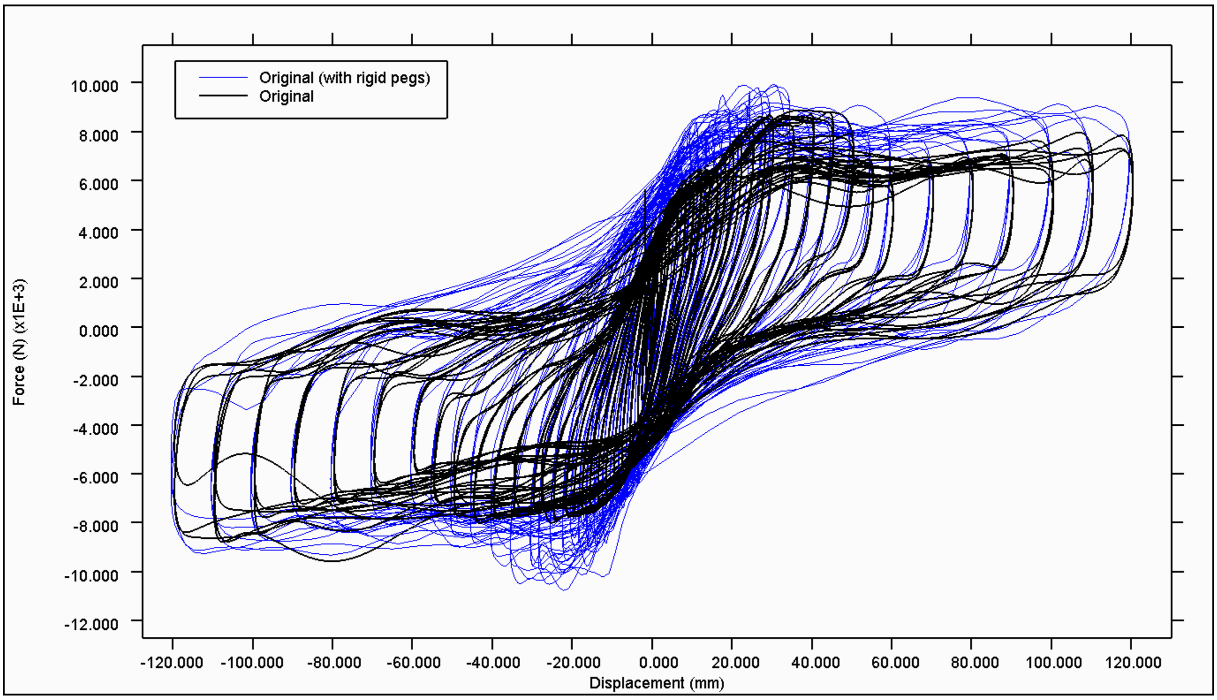


Figure 4-28 Comparison of the benchmark model with modified model that has unbreakable corner timber connections.

The behaviour around these critical connections is discussed further in Section 4.5.8

4.5.2 Overburden

Three levels of over burden were considered:

1. 4.6kN/m as a line load along the top beam and represents the weight of an additional timber floor
2. 9.2kN/m as a line load along the top beam and represent either two additional timber floors or one floor with a three inch thick mud screed.
3. 18.3kN/m as a line load along the top beam and represent two additional floors with three inch thick mud screeds

The detailed calculations of the chosen over burden values are shown in Appendix D8 and are summarised in Figure 4-29.

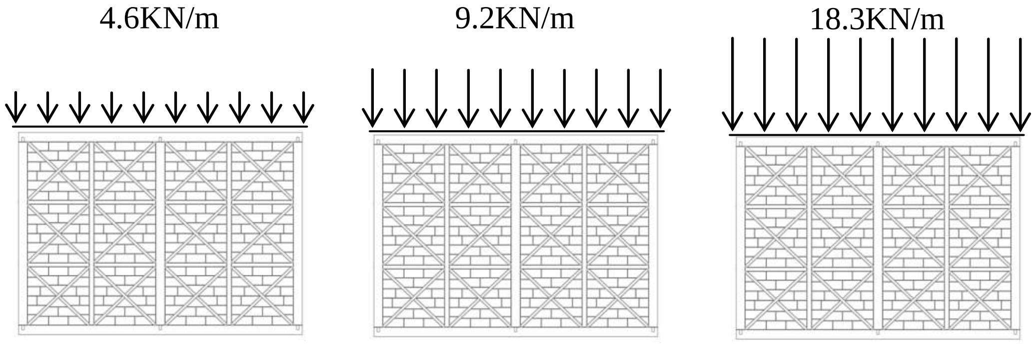


Figure 4-29 Three levels of over burden. 4.6 kN/m, 9.2 kN/m and 18.3kN/m

The seismic performances of the sub frames with the three levels of over burden were evaluated with and without nails.

The overburden was applied as a line load acting vertically downwards on the top timber beam. The sub frame was subjected to the same cyclic displacements as before. This allows us to look at the resistance of the frames if they had higher levels of compression acting on them. In this instance the overburden acted as additional pre compression on the dhajji dewari walls.

Increasing over burden levels increases the resistance offered by the masonry due to increased friction capacity of the assembly as shown in Figure 4-30. Because the timber is modelled using elastic material properties, failure of the timber sections and thus loss of load carrying capacity of the sections is not explicitly captured in the current analysis model. Further work is needed to validate this assumption.

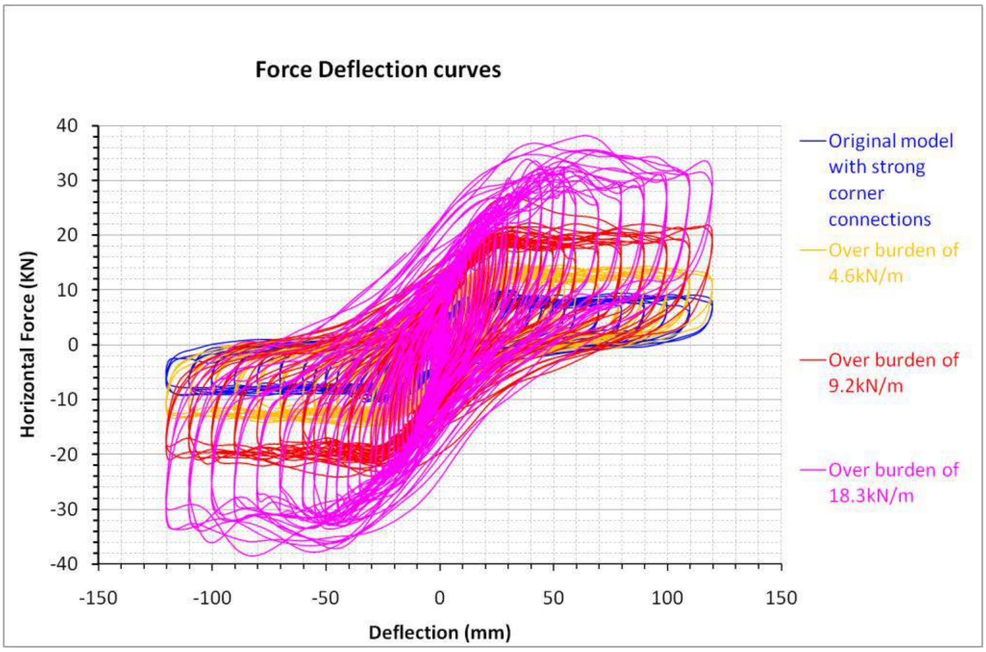


Figure 4-30 Hysteresis loop comparison between overburden levels.

The same models were evaluated without nailed connections. The results in Figure 4-31, compared to the plots of Figure 4-30, show that the nails contributed significantly towards the stable response of the sub assembly by confining the masonry panels. This maintains higher normal forces on the masonry which results in less pinching of the hysteresis loops and therefore greater levels of energy absorption.

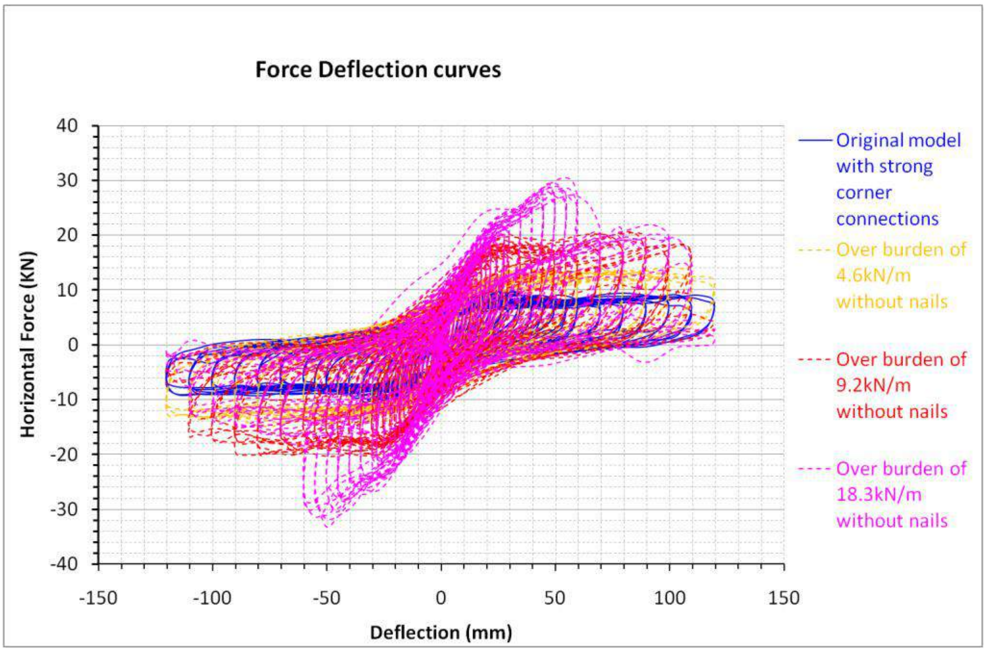


Figure 4-31 Hysteresis loop comparison between overburden levels without nails

The area inside the hysteresis loops is equal to the amount of energy absorbed in the system. These runs show that even without nails increased levels of over burden increase the energy absorption capacity of the dhajji dewari structural system as shown in Table 4-1.

Table 4-1 Energy absorption comparison with varying levels of over burden		
Model description	Work Done (Joules) Area inside the hysteresis loop	Normalised to the original model
Original model with strong corner connections	5.07×10^4	1.00
Over burden of 4.6kN/m	7.56×10^4	1.49
Over burden of 9.2kN/m	1.09×10^5	2.16
Over burden of 18.3kN/m	1.51×10^5	2.99
Over burden of 4.6kN/m without nails.	7.22×10^4	1.42
Over burden of 9.2kN/m without nails.	8.03×10^4	1.58
Over burden of 18.3kN/m without nails.	8.27×10^4	1.63

The energy absorbed in the system was calculated by two different methods as shown in Figure 4-32. The automatic energy calculation by LS-DYNA agrees very closely with a manual integration of the force vs. deflection curve. This is a good indicator that the analysis model is behaving appropriately. The LS-DYNA calculated energy will also account for energy lost due to the nominal levels of damping used in the analysis which is not captured in the hysteresis loops.

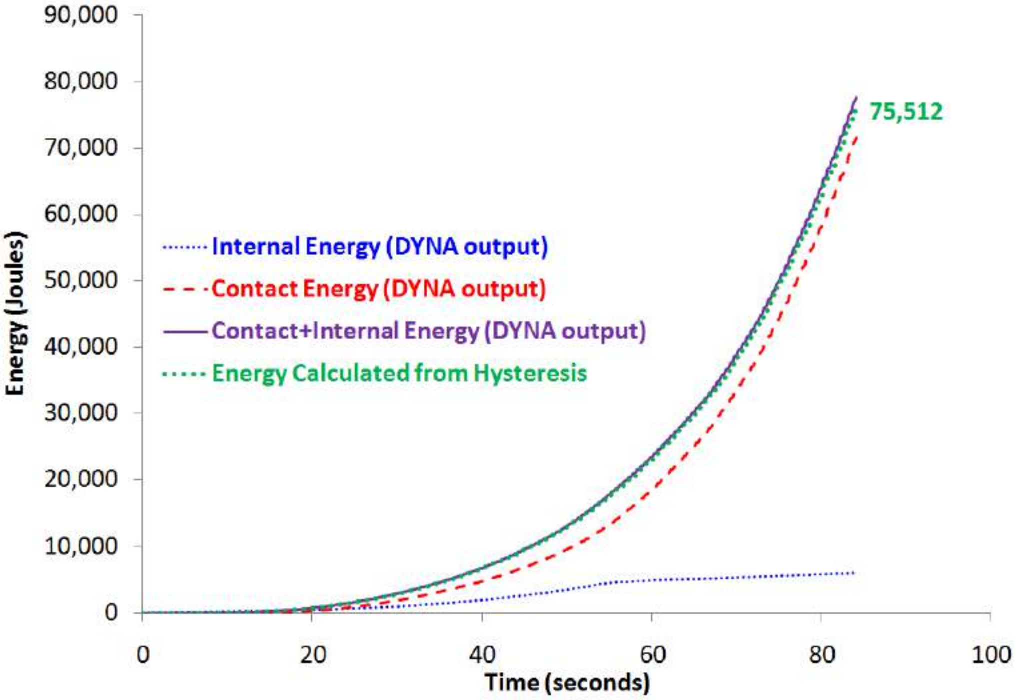


Figure 4-32 Dissipated energy for the over burden of 4.6kN/m.

The model with an increased over burden of 18.3kN/m has absorbed 3 times more energy for the same displacement demand as in the original model.

In these runs the loading was applied as a forced displacement in a controlled manner (i.e. quasi static cyclic loading).

Applying higher levels of overburden (or pre compression) on this construction system shows clear benefits. In an actual earthquake the overburden load from the upper stories of multi storey dhajji dewari buildings will exert additional inertia forces on the structure because the mass of the upper floors is also accelerated by the earthquake. This will generate higher lateral loads that must be resisted by the dhajji dewari frames. It still needs to be demonstrated that the increased energy absorption and greater lateral force resisting capacity shown by the frames in these analyses results does actually translate into a structure where its seismic resistance has increased more than the additional demands generated by having the upper floors exert additional inertia forces on the frames. Clearly a system of prestressing the walls, without the need for additional mass, would give all the benefits without attracting the increased inertia forces.

4.5.3 Lack of Fit

In the original analysis model the braces fit precisely into the timber frame. It is thought that the actual construction as practiced in the field rarely achieves this level of accuracy. The timber diagonal braces have been shortened by increasing amounts, to account for lack of fit from the original construction, the effects of timber shrinkage with time and ultimately indirectly mimic a frame without diagonal braces whilst avoiding the need to re-mesh the analysis model. This would allow the quantification of the behaviour of the frame without braces all together. Therefore three levels of lack of fit of braces have been investigated based on the calculations shown in Appendix D9, summarised below and shown in Figure 4-33.

This will mean that the braces will only engage after some movement of the frames has occurred.

1. Braces that are 7.5mm too short at either end giving a total brace shortening of 15mm.
2. Braces that are 12.5mm too short at either end giving a total brace shortening of 25mm.
3. Braces that are 25.0mm too short at either end giving a total brace shortening of 50mm.

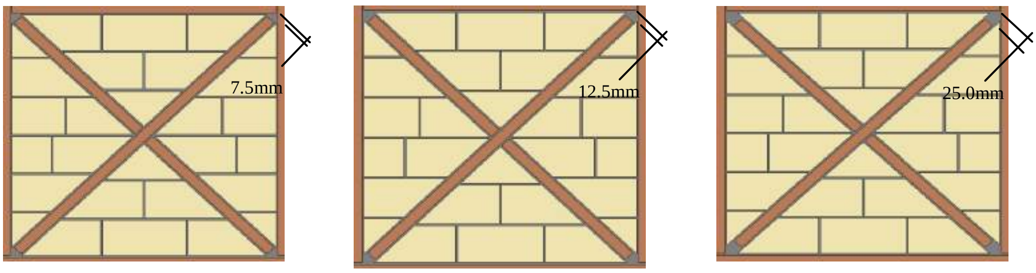


Figure 4-33 Shortening of the diagonal braces by 15, 25 and 50mm.

Superposition of the force vs. deflection curves for the three levels of brace length reduction, as shown in Figure 4-34, shows that having the braces or not having them engaged has a modest impact on the hysteresis behaviour of the structural assembly.

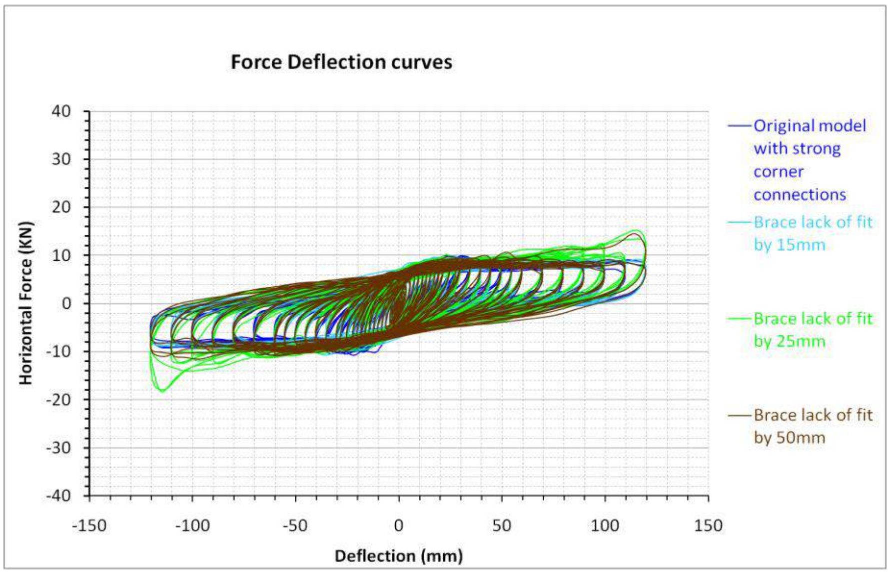


Figure 4-34 Hysteresis loop comparison as the fit of the braces is reduced

Plotting the curves one by one, as shown in Figure 4-35, shows that not having the braces fully engaged reduces the amount of pinching in the hysteresis loops.

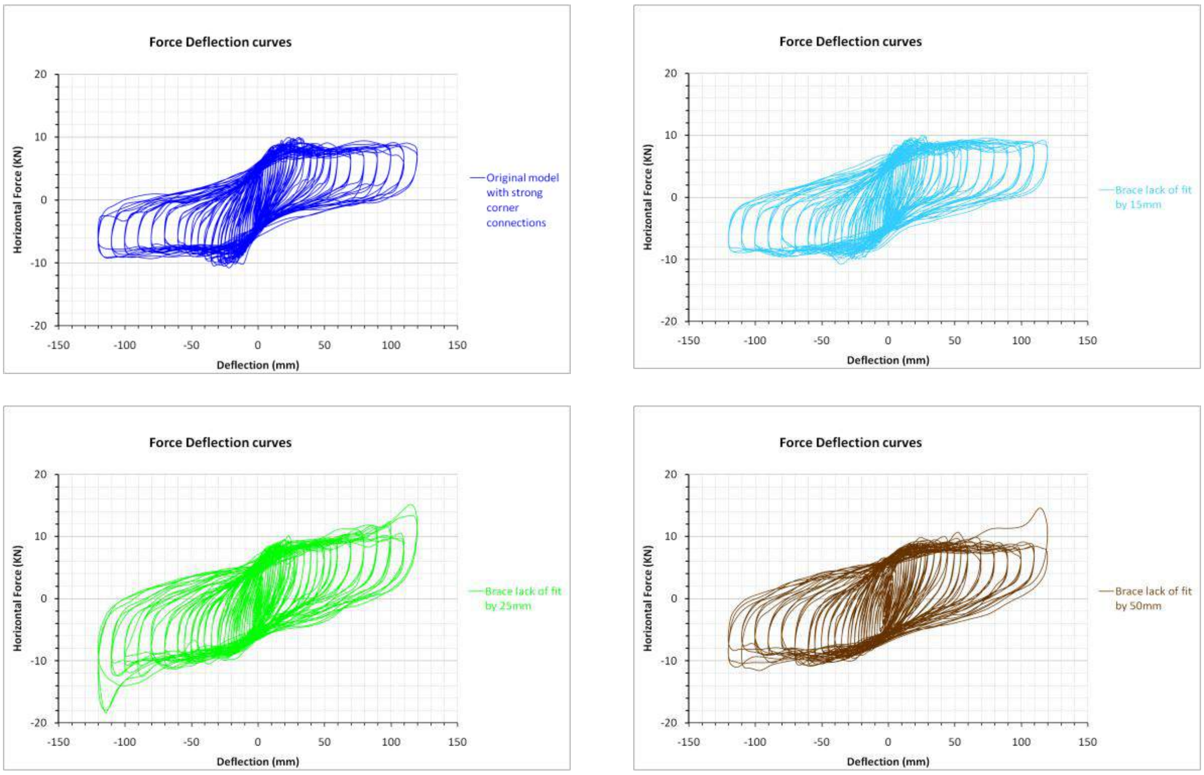


Figure 4-35 Hysteresis loop comparison as the fit of the braces is reduced

A closer examination of the work done reveals that there is a 9% to 21% increase in the amount of energy absorbed in the system as the braces are reduced in length (as shown in Table 4-2). This means that we are increasing the flexibility of the frames and encouraging the masonry units to more readily slide across each other and therefore absorb energy through friction between the units. This suggests that that the bracing may be less important except as temporary works during construction.

Table 4-2 Energy absorption comparison with reducing levels of brace fit.

Model description	Work Done (Joules) Area inside the hysteresis loop	Normalised to the original model
Original model with strong corner connections	5.07E+04	1.00
Brace lack of fit by 15mm	5.54E+04	1.09
Brace lack of fit by 25mm	6.09E+04	1.20
Brace lack of fit by 50mm	6.13E+04	1.21

Although the improvement is small and clearly not as beneficial as increased levels of over burden or having strategically located nails, it merits further research on a number of counts:

1. Potential significant reduction in the amount of required timber
2. Construction of the masonry infill without having to cater for diagonals is simpler and faster.
3. Removal of bracing will increase the masonry panel size which increases the risk that the masonry will become unstable during an earthquake. It should be noted that the collapse of larger pieces of masonry in an earthquake is more dangerous to residents than the collapse of smaller pieces. However, it is suspected that building tightly packed triangular masonry panels that have a tight fit around the timber members is practically hard to achieve and many of these panels may end up being more loosely finished then rectangular panels.
4. Work is required to better understand the safe spacing, both vertically and horizontally, for a variety of masonry units.

4.5.4 Overburden and lack of fit

The combination of shorter braces with increasing levels of over burden show reduced pinching of the hysteresis loops as shown in Figure 4-36. However, the maximum resisted load builds up slower when compared to the case where only the over burden was increased.

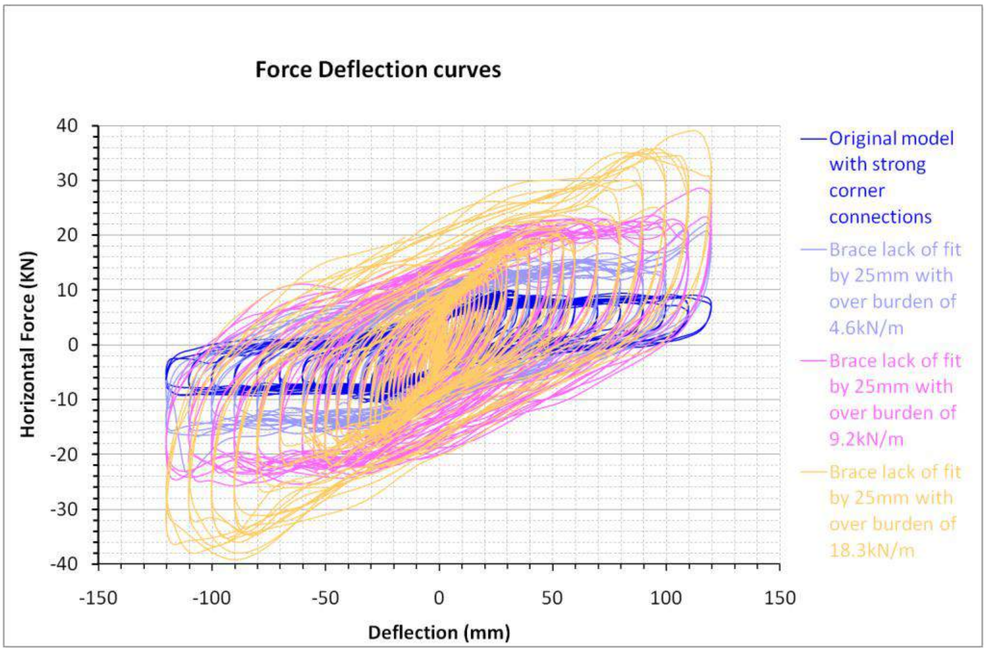


Figure 4-36 Hysteresis loop comparison

Comparison of the work done within the hysteresis loops shows that for over burden levels of 4.6kN/m and 9.2kN/m greater amounts of energy are absorbed when the braces are not as tightly engaged (83% vs. 49% and 168% vs. 116%). However, this pattern is not repeated for the highest considered over burden level (178% vs. 199%).

Table 4-3 Energy absorption comparison with reducing levels of brace fit.

Model description	Work Done (Joules) Area inside the hysteresis loop	Normalised to the original model
Original model with strong corner connections	5.07E+04	1.00
Brace lack of fit by 25mm with over burden of 4.6kN/m	9.29E+04	1.83
Brace lack of fit by 25mm with over burden of 9.2kN/m	1.36E+05	2.68
Brace lack of fit by 25mm with over burden of 18.3kN/m	1.41E+05	2.78

The horizontal shear forces across the edge posts are large and a potential vulnerability and merits further work to better understand the demands on these connections and the timber sections as previously outlined.

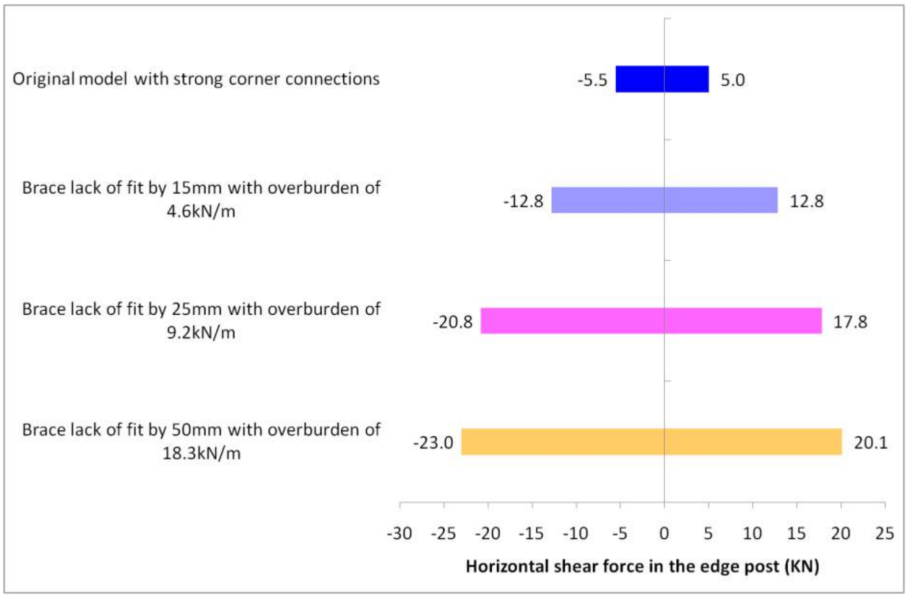


Figure 4-37 Edge post horizontal shear forces with 25mm lack of brace with and increasing levels of over burden.

4.5.5 Comparison of runs at an over burden of 4.6kN/m

The 25mm brace shortening increased the energy absorption capacity of the assembly by 20% and the increase in over burden by 49% when assessed in isolation. When combined these two features increased the energy absorption capacity of the system by 83% which is modestly more than the sum of the individual runs. This suggests that allowing the frame to move in a stable manner benefits the energy absorption of the system at this level of over burden.

However, having a more flexible system, such as mimicking the removal of the braces by shortened them, means that the building frame will deflect more than a braced frame and that it is likely to sustain damage at lower levels of ground shaking. Initially the damage will be to non structural finishes but the larger displacements will place greater demands on the connections. However, as long as the main timber frame and its main connections do not break it is believed the masonry will settle back into the mud mortar and its residual strength should be relatively unaffected unless the building develops a significant irrecoverable displacement.

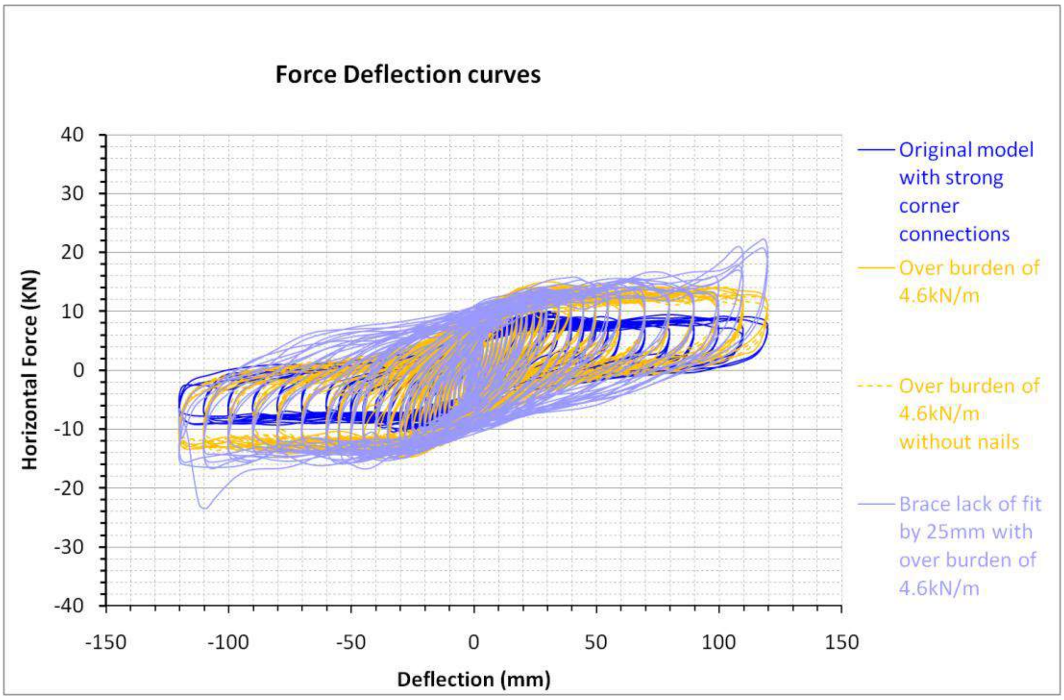


Figure 4-38 Hysteresis loop comparison

Table 4-4 Energy absorption comparison with reducing levels of brace fit.

Model description	Work Done (Joules) Area inside the hysteresis loop	Normalised to the original model
Original model with strong corner connections	5.07E+04	1.00
Over burden of 4.6kN/m	7.56E+04	1.49
Over burden of 4.6kN/m without nails	7.22E+04	1.42
Brace lack of fit by 25mm with over burden of 4.6kN/m	9.29E+04	1.83

Removal of the nailing resulted in a 7% reduction in energy absorption capacity.

Further investigation is merited to quantify the energy absorption capacity of the frames under bi- and tri-directional dynamic earthquake time history excitation. This work is necessary to fully understand the stability of the masonry and timber assembly under multi directional excitation.

4.5.6 Comparison of runs at an over burden of 9.2kN/m

The 25mm brace shortening increased the energy absorption capacity of the assembly by 20% and the increase in over burden increased it by 116% when assessed in isolation. When combined, these two features increased the energy absorption capacity of the system by 168%. This is considerably more that the linear sum of the two and suggest that allowing the frame to move in a stable manner benefits the energy absorption of the system at this level of over burden.

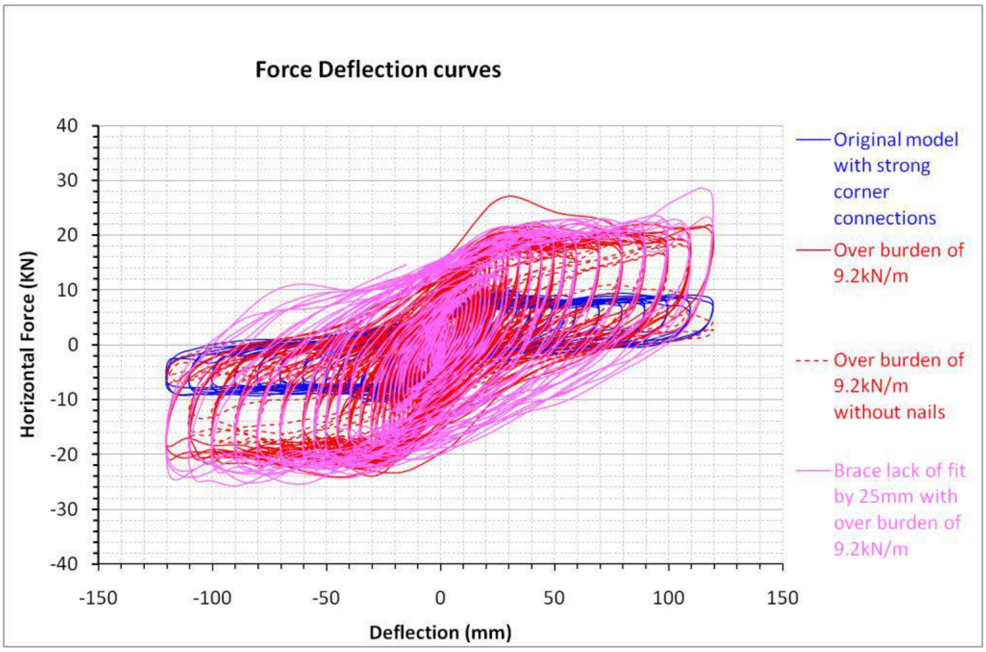


Figure 4-39 Hysteresis loop comparison

Table 4-5 Energy absorption comparison with reducing levels of brace fit.		
Model description	Work Done (Joules) Area inside the hysteresis loop	Normalised to the original model
Original model with strong corner connections	5.07E+04	1.00
Over burden of 9.2kN/m	1.09E+05	2.16
Over burden of 9.2kN/m without nails	8.03E+04	1.58
Brace lack of fit by 25mm with over burden of 9.2kN/m	1.36E+05	2.68

Further work is necessary to confirm the observed behaviour and ideally it will result in an optimum configuration of brace length (or no braces as the case may be) and the best amount of pre compression to the timber and masonry assembly.

4.5.7 Comparison of runs at an over burden of 18.3kN/m

The 25mm brace shortening increased the energy absorption capacity of the assembly by 20% and the increase in over burden by 199% when assessed in isolation. When combined these two features increased the energy absorption capacity of the system by 178%. This is less than the linear sum of the two and suggests that at this level of over burden the system has struggled to maintain the same level of stable hysteretic behaviour. It does remain to be explored how the frames would behave under increased levels of imposed displacement demand as applied in the analyses runs. Pushing the frames further will provide additional valuable information and needs to be carried out as part of further research.

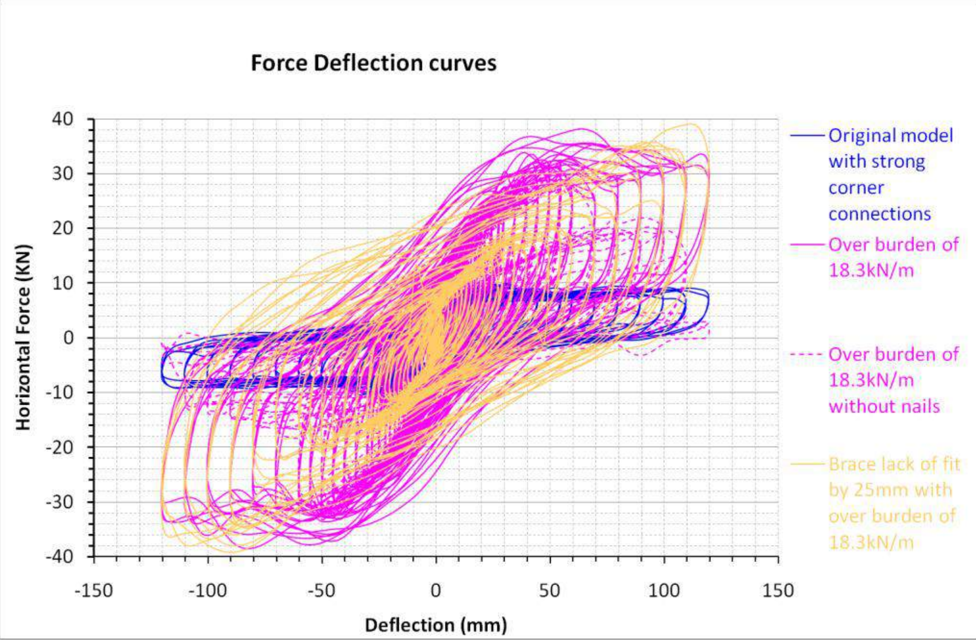
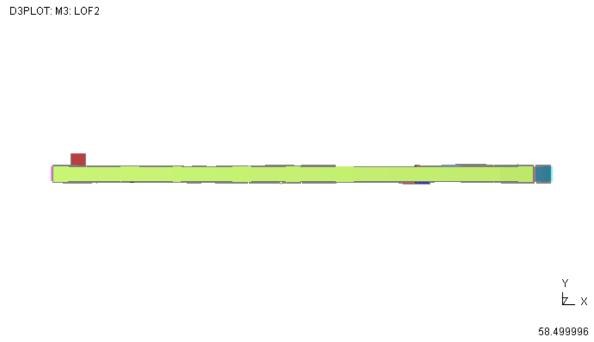


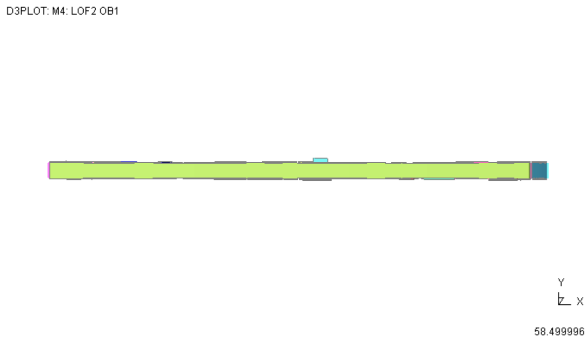
Figure 4-40 Hysteresis loop comparison

Table 4-6 Energy absorption comparison with reducing levels of brace fit.		
Model description	Work Done (Joules) Area inside the hysteresis loop	Normalised to the original model
Original model with strong corner connections	5.07E+04	1.00
Over burden of 18.3kN/m	1.51E+05	2.99
Over burden of 18.3kN/m without nails	8.27E+04	1.63
Brace lack of fit by 25mm with over burden of 18.3kN/m	1.41E+05	2.78

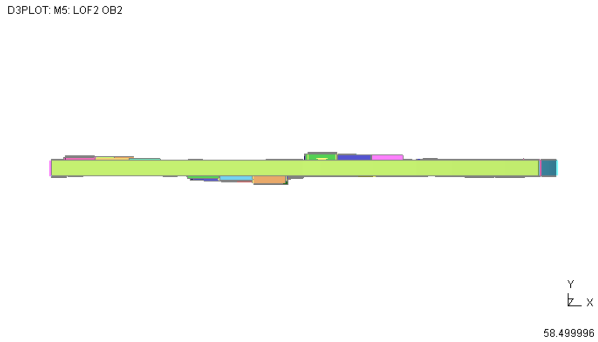
Further work is required to understand the reduction in the absorbed energy when the shortened braces are assessed with the higher levels of overburden. A possible explanation is that at the higher over burden levels the masonry is slowing walking out of the wall which reduced the area under contact as shown in Figure 4-41. This then leads to reduced levels of energy dissipation. An alternative explanation might be that the frame was not displaced far enough and that the frames should be rerun with a higher displacement demands. Clearly further work is required to improve our understanding.



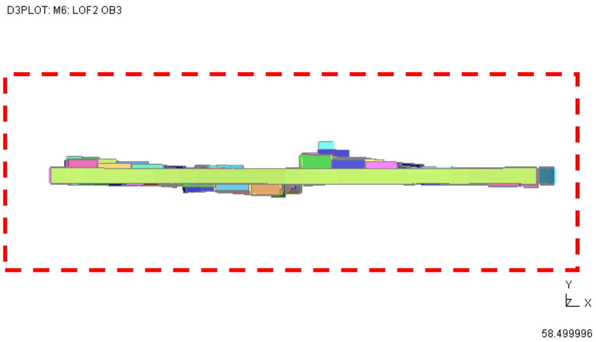
Brace lack of fit by 25mm



Brace lack of fit by 25mm with an over burden of 4.6kN/m



Brace lack of fit by 25mm with an over burden of 9.2kN/m



Brace lack of fit by 25mm with an over burden of 18.3kN/m

Figure 4-41 Out of plane masonry response to in plane loading (shown with exaggerated displacement)

4.5.8 Connection behaviour.

The analysis work undertaken was not designed to specifically look at the detailed non linear response of the critical corner connections, as already alluded to in Section 4.5.1. However the structural behaviour around this connection merits further discussion given its critical contribution towards keeping the dhajji dewari system together and thus working.

It is critical in all timber framing configurations that the main frame is sufficiently strong and that its connections have enough ductility and / or strength to withstand the cyclic earthquake load demands safely. Failure of the main timber frame at the connections may lead to the unzipping of the structure and needs to be guarded against. We have measured the cross section forces of the edge posts as shown in Figure 4-42.

All of the runs displayed indicate a significant increase in the horizontal shear forces experienced by the edge post as either the over burden is increased, the nails are removed or the braces are shortened.

Clearly further work is needed to better understand the axial, bending and shear demand acting at the connections as well as the gross timber sections.

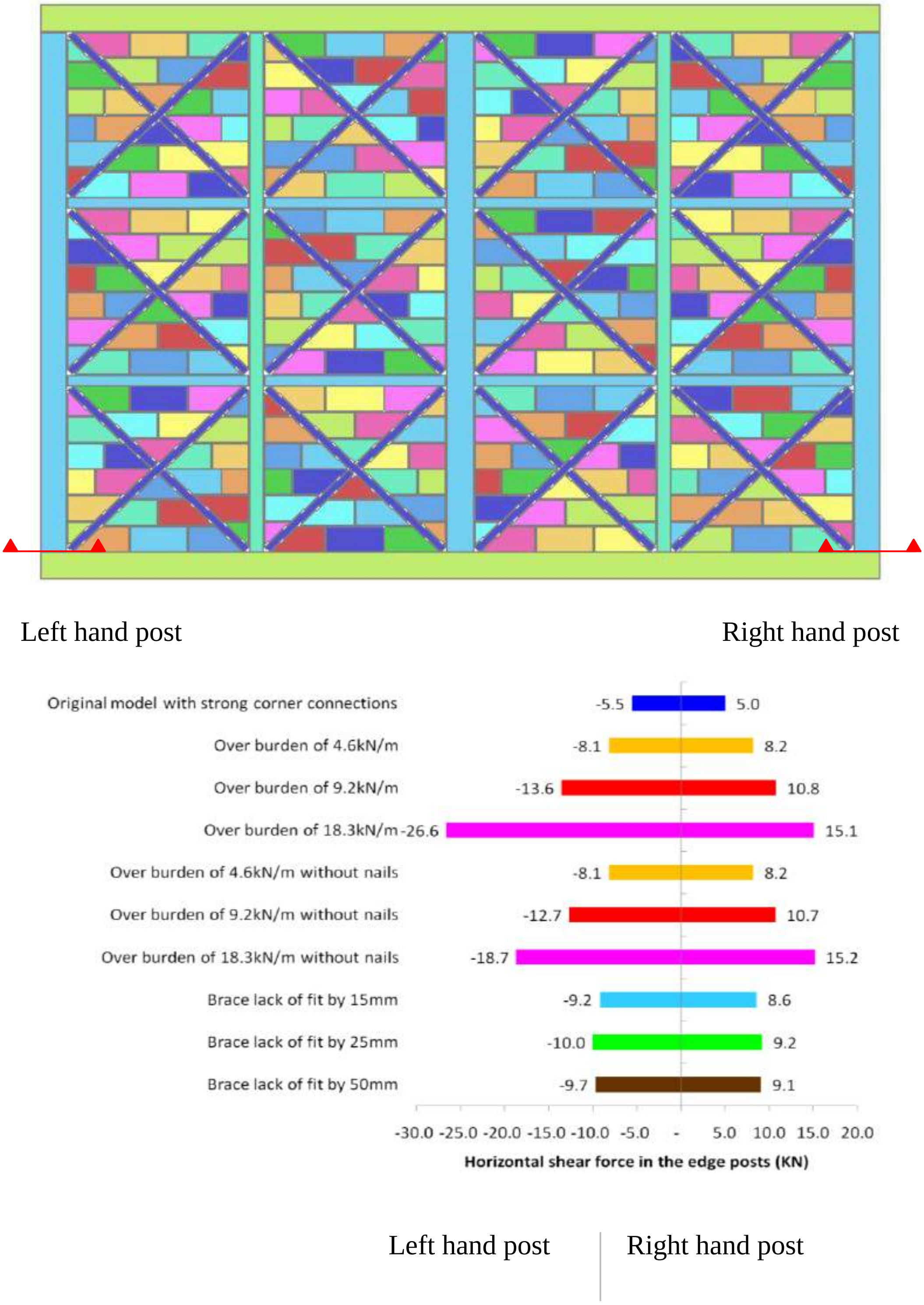
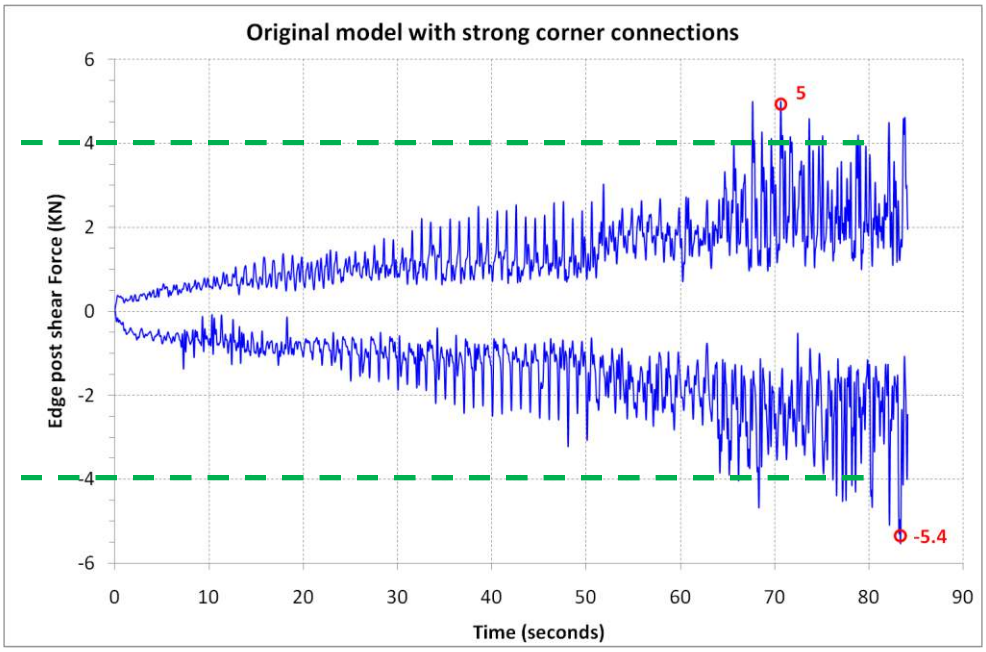


Figure 4-42 Comparison of horizontal shear forces in the edge posts.

Figure 4-43 demonstrates how the analysis is able to track the cross section forces and that these are broadly equal and opposite in sign and are correlated to the direction in which the displacement is being applied to the frame.



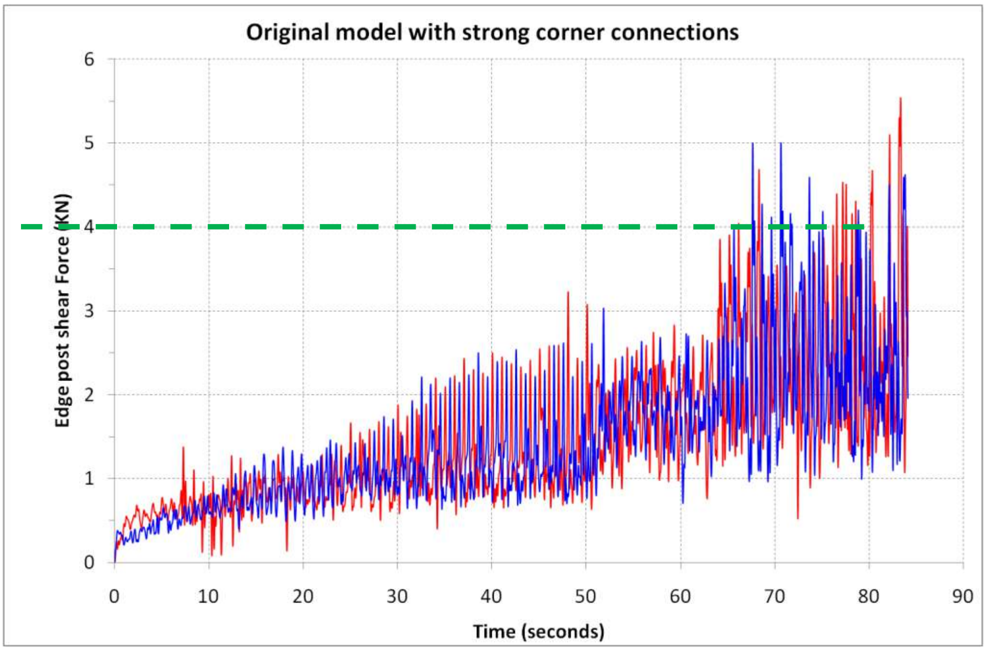


Figure 4-43 Edge post horizontal shear force time history at the left and right hand frame side

The timber pegs at the mortise and tenon joints were given an ultimate shear capacity of 4KN in the initial models. The forces measured in the sensitivity runs consistently show significantly larger shear force demands just above these connections. This implies actual early failure of the connections in their current form during an earthquake. Whilst we have made these connections artificially strong in these sensitivity runs we do need to be mindful that in the physical testing carried out at UET Peshawar, the bench mark test model and these runs indicate that the connections are a potential weakness in the dhajji dewari frame system. This merits further investigation around these critical connections to enable us to calculate the force and deformation demands acting on these connections during an earthquake and evaluate their safe limits. Detailing rules are required to guide the engineer and builder to make these connections robust and ductile.

5 Sustainability

The title of this report is: "Affordable seismically resistant and sustainable housing" and the report has so far concentrated on the seismic behaviour of dhajji dewari. Therefore some justification is required for the sustainability aspect of this reports title.

The thinking outlined below has not been prepared by experts in sustainability but should be considered as a first attempt to articulate dhajji dewari's structural sustainability credentials.

5.1 Materials

The photos in section 2 and Appendix A illustrate that dhajji dewari is made up of stone, timber, mud and mainly metal corrugated roof sheets. In traditional dhajji dewari construction most materials are sourced locally.

5.1.1 Stone

The stone for the foundations and infill material come from the owners land or nearby mountains.

Transport for the stone is usually by manual labour with minimal use of fossil fuels for transportation.

Preparation of the stone is by manual labour without resorting to energy intensive fabrication.

Often stone are reused if building again, such as after an earthquake.

5.1.2 Timber

Traditionally timber was always sourced locally. Judicious use of masonry for durable structures that utilise timber but not requiring excessive amounts of it can encourage the replanting of forests in areas where timber resources have become depleted. dhajji dewari also lends its self to using lots of small timber pieces and is therefore able to utilise timber more efficiently and readily make use of recycled timber

5.1.3 Mud

Mortar to lay the infill and plastering is traditionally locally sourced and consists of mud which may have been mixed with river sand and sometimes animal hairs or straw to help reduce cracking of mud plaster renders.

5.1.4 Other

Nails, metal straps and metal roof sheets are probably the newest material addition to the construction method in modern dhajji dewari.

Clearly there are other aspects, such as insulation levels, method of cooking and heating (or cooling), amount of day lighting to name a few of the other aspects of dhajji dewari that impacts up on its overall environmental impact but these issues are common non structural issues that have to be address by all other construction forms as well.

6 Findings

The key findings from each of the main analyses described in section 4 are presented in the following section.

6.1 Benchmark tests

This phase of the work is perhaps the most interesting as it provides the best opportunity to date of validating the approach in both a quantitative and qualitative manner. Qualitatively, the photographs supplied by UET match well with the analytical model. It seems that the structural mechanisms mobilised in the physical tests were broadly reproduced by the analysis. The analysis showed the same key deformation as the physical tests (lifting of corner columns, separation between timber and infill elements, and distortion of the timber frame).

Quantitatively, the results are encouraging. Both the model and physical wall exhibit stiffness degradation as the displacement cycles increase, although this is more gradual in the real wall, the stiffness reduces substantially for cycles above 50mm amplitude in all the hysteresis curves (see Figure 4-6 and Figure 4-7).

The peak capacity of the analytical wall model is lower than that of the UET specimens. This can probably be attributed to the difference in jointing. The UET wall specimen made greater use of nails which will have contributed to the initial strength of the timber frame. However, once some of the connections had pulled out many of the nails will have become in effective and thus more closely resembling the analytical model we had built.

The opportunity to benchmark this work has been invaluable and we are very grateful to the team at UET for sharing their work. Just as analytical work is reliant on the assumptions made, all physical testing is subject to variability. With this in mind, it would be useful to have a larger sample of tests to produce a more statistically significant data set. Additionally it would be useful to have test data for out-of-plane deformation as this seems to be particularly important to the dhajji dewari's structural integrity.

6.2 Quasi-Static Pushover Tests

The pushover curves generated in DYNA show a long and gradual drop-off in force as displacement increased. Even at very large displacements (>1m), some of the walls remained in-tact. This stable behaviour was no doubt influenced by the lightweight roof. With a multi-storey or concrete roofed Dhajji, the additional over turning moments generated by the vertical load and the horizontal deformation would be more severe and could threaten the global stability of the building.

Out-of-plane stability was, predictably, the major issue for the pushover in both the long and short directions; the walls were able to accommodate large amounts of in-plane deformation without loss of integrity.

6.3 Time History Analysis

The time-history work underlined the importance of connection detailing to the performance of this structure. In the more onerous time-history, which included near source effects, the presence of ‘nailed’ connections was sufficient to prevent the structure from collapse. As highlighted in the pushover tests, the out of plane behaviour of the walls was the primary cause of structural instability. Limiting the out of plane demands by having regular cross walls and connections that hold all the timber pieces together all contribute towards making the frames stable. This has a significant beneficial effect on the overall behaviour of dhajji dewari buildings.

This phase of work also demonstrated DYNA’s capacity to model a full scale building under realistic earthquake time-histories. In performance based seismic design of structures, analytical models are usually subject to several time-histories. This is to account for variation in earthquake characteristic (frequency content, duration etc). Time restrictions have limited this to two time-histories on this project and before making substantive design recommendations a larger set of records should be considered.

6.4 Sensitivity runs

The sensitivity runs undertaken provide an opportunity to study the response of the system to a change in one parameter at a time whilst also allowing us to look at their combined effect.

Increasing the level of overburden, to give higher compression on the walls, is beneficial as it raises the amount of energy absorbed as the masonry units slide over each other under horizontal loads. However some of the apparent gains may be offset in reality by larger inertia forces generated from having heavier buildings. A practical method to pre compress the walls vertically without incurring the penalties associated with heavier construction would be ideal.

Further work is needed to determine if critical timber connections are strong and ductile enough to keep the timber frame together. Without the timber frame, confinement is lost to the masonry panels and the wall system quickly becomes unstable.

Reducing the length of the braces has resulted in modest improvements in the level of absorbed energy through reduced pinching of the hysteresis curves. However this does merit more detailed work because if the system behaves even slightly better without braces then the construction process could be simplified. Omitting some or all of the bracing would result in saving in timber volumes and reduced installation time for the timber frame. Placing the infill will be easier because it does not need to be made to keep adjusting to the diagonal braces.

Although the model has a limited number of nails, their contribution to keeping the frames together is seen as a very important. More work is needed to determine where nailing is the most effective and how nailing or strapping could be detailed to get the highest return. The building fabric will continuously transmit moisture to any embedded metal pieces such as nails and their long term integrity is questionable unless proper attention is given to using nails with appropriate corrosion protection such as galvanised nails or even copper nails. It would be prudent to determine limits on the contribution nails can make to the stability of dhajji dewari buildings in order to account for losses of metal cross section due to corrosion.

7 Summary

This work has demonstrated that it is possible to model the behaviour of traditional dhajji dewari buildings. Qualitatively the analytical model has reproduced the physical mechanisms seen in physical tests of a similar wall and has produced quantitative results which are consistent with the physical specimens (given the differences in geometry and connections). The following points summarise the key findings from the research and analysis conducted to date:

1. Dhajji dewari can safely resist earthquakes in high seismic regions of the world when built properly and maintained adequately. This makes dhajji dewari a valid form of construction in seismic areas.
2. The timber framing provides stable confinement to the infill masonry as long as it remains together. Therefore it is critical that the timber connections are detailed to have sufficient strength and ductility. Strategic use of nails and / or metal straps improves the performance of the connections.
3. Seismic energy is dissipated through friction between the masonry panels and the timber frame and within the yielding of the connections.
4. Increased levels of over burden acting on the masonry increases the energy absorption capacity of the assembly but if this is achieved through heavy roofs the increase in inertia forces may be greater than the apparent strength gain.
5. Shortening the braces to make them less effective leads to nominal improved seismic energy absorption of the system. Certainly the performance was no worse when compared to having fully engaged braces, and brace removal potentially leads to simpler construction and less timber volume.

Broadly dhajji dewari is similar conceptually to ‘confined masonry’ construction which has concrete ring beams and columns confining the unreinforced masonry infill. The main difference is that in a ‘confined masonry’ system the sand cement mortar used to bond the masonry pieces together is brittle and stiff while traditional dhajji dewari has mud mortar which is very weak which allows it to start yielding even under relatively small lateral loads. In dhajji dewari construction the masonry panel sizes are typically smaller than in confined masonry construction too. The energy in the dhajji dewari system is dissipated mainly in friction between the infill pieces and not through the non-linear material deformations of the frame members as would be the case in modern steel or reinforced concrete construction. Therefore if key connections can be prevented from falling apart, then the integrity of the timber frame is secured and the infill dissipates the seismic energy through friction energy which is mobilised as the masonry pieces slide across each other.

The analysis showed the merit of using nails to help hold the system together. Further studies are warranted to establish optimum nailing configurations and arrangements of the components for dhajji dewari building. Whilst nails are prone to rusting, the value of good carpentry connections should not be over looked to make its rightful contribution to good seismic behaviour.

It is possible to imagine that after an earthquake that there will only be limited and repairable damage to a dhajji dewari building due to the unique properties of the system. This is a significant benefit over many modern engineering concepts and is suited as a housing type that is relatively easy to build and repair.

8 Conclusions and Recommendations

If we are to create communities that are both sustainable and resilient, it is necessary to adopt construction technologies that make best use of available resources and are safe. Dhajji dewari offers hope to this cause by using durable renewable or recycled materials that are likely to be locally available, therefore can be easily maintained and repaired. This research shows that it also offers a form of construction that is inherently seismically resistant, and if damaged can be repaired relatively easily.

This research is an important step in understanding the behaviour of dhajji dewari structures and generating wider acceptance of this building system amongst the general public, donors and government. Having created a validated analytical model, further sensitivity analyses can be undertaken to test the performance of many critical elements of the house. Our suggestions of further necessary investigations are outlined in Table B5-1 It is also possible to adapt the model to assess other structural configurations such as the building model shown in Figure 8-1.

Further investment and research is needed, ultimately leading to:

- An evidence based earthquake engineering building standard and construction guidelines for dhajji dewari buildings.
- An evidence based earthquake engineering building standard and construction guidelines for retro-fitting existing dhajji dewari buildings.
- Training materials aimed at self-builders, university students, architects and engineers and government.

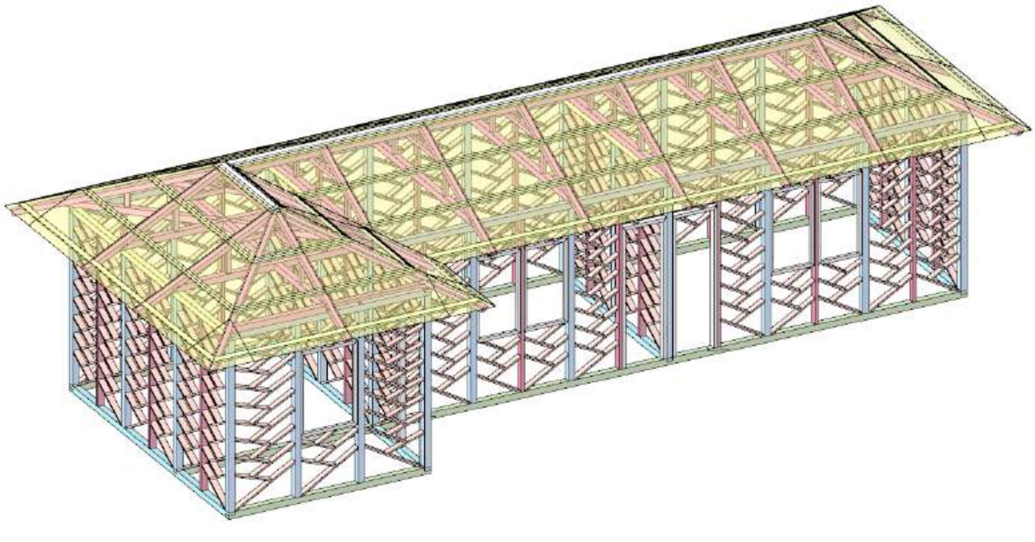


Figure 8-1 Dhajji dewari building with alternative timber framing configuration requiring engineering assessment

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[34] Time history selection software by Damian Grant

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Photo and image sources

All images and photos in this report are from Arup or were generated as part of the engineering analysis by Arup unless noted otherwise below.

Figure Number	Source
Figure 2-1	Tom Schacher
Figure A1-1 FigureA2-19	Jitendra Bothara
Figure A1-2 Figure A1-4	Randolph Langenbach
Figure A1-3 FigureA2-3 to FigureA2-18 FigureA2-20 to FigureA2-49 FigureA2-51 Figure 3-1 to Figure 3-4	UN-Habitat, Islamabad, Pakistan
Figure A1-6	from www.nvmdigital.com, and by by Hans Peter Schaefer www.histariege.com/le_mas_d_azil.htm
Figure A1-7	Wilfredo R. Rodríguez H.
Figure A1-8	Raquel Paula, Vitor Colas
Figure A1-9	www.turkishculture.org/pages.php?ChildID=278&ParentID=6&ID=27&ChildID1=278
Figure A1-10	Tulay Cobancaoglu
FigureA2-1	Based on sketches from UN-Habitat, Pakistan
FigureA2-2	Indian Building Code IS-4326

Appendix A

Field information on dhajji dewari buildings

A1 Dhajji dewari examples from around the world

Such houses are found in both the Pakistani and Indian sides of Kashmir. Similar houses are found in Britain, France, Germany, Central America, South America, Turkey, Portugal and Italy. They are known as ”Half-timber”, “Colombage”, “Fachwerk”, “Taquezal or Bahareque”, “Quincha”, “Hımış”and “Gaiola” respectively. This form of construction is also known as “Brick nogged timber frame construction” in India and housing like this are called Ginger-Bread houses in the city of Port au Prince in Haiti.

Examples from around the world are shown in Figure A1-1 to Figure A1-12.

Figure A1-1.A Dhajji building in Simla, India without bracing elements.

Figure A1-2 . Multi-storey Dhajji building in Shrinagar, India. Photo source © Randolph Langenbach.

Figure A1-3. A two storey Dhajji building that survived the 2005 Pakistan earthquake.

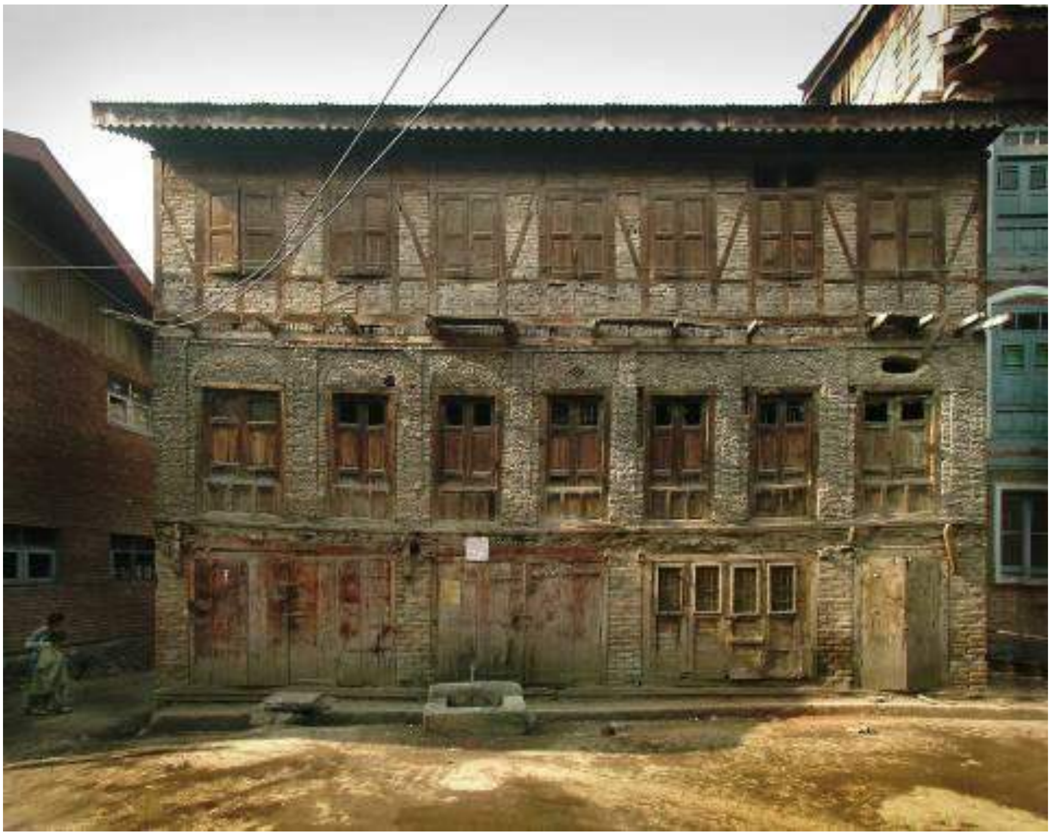


Figure A1-4. A building with Dhajji in upper most storey only, from Srinagar, India. Photo source © Randolph Langenbach.

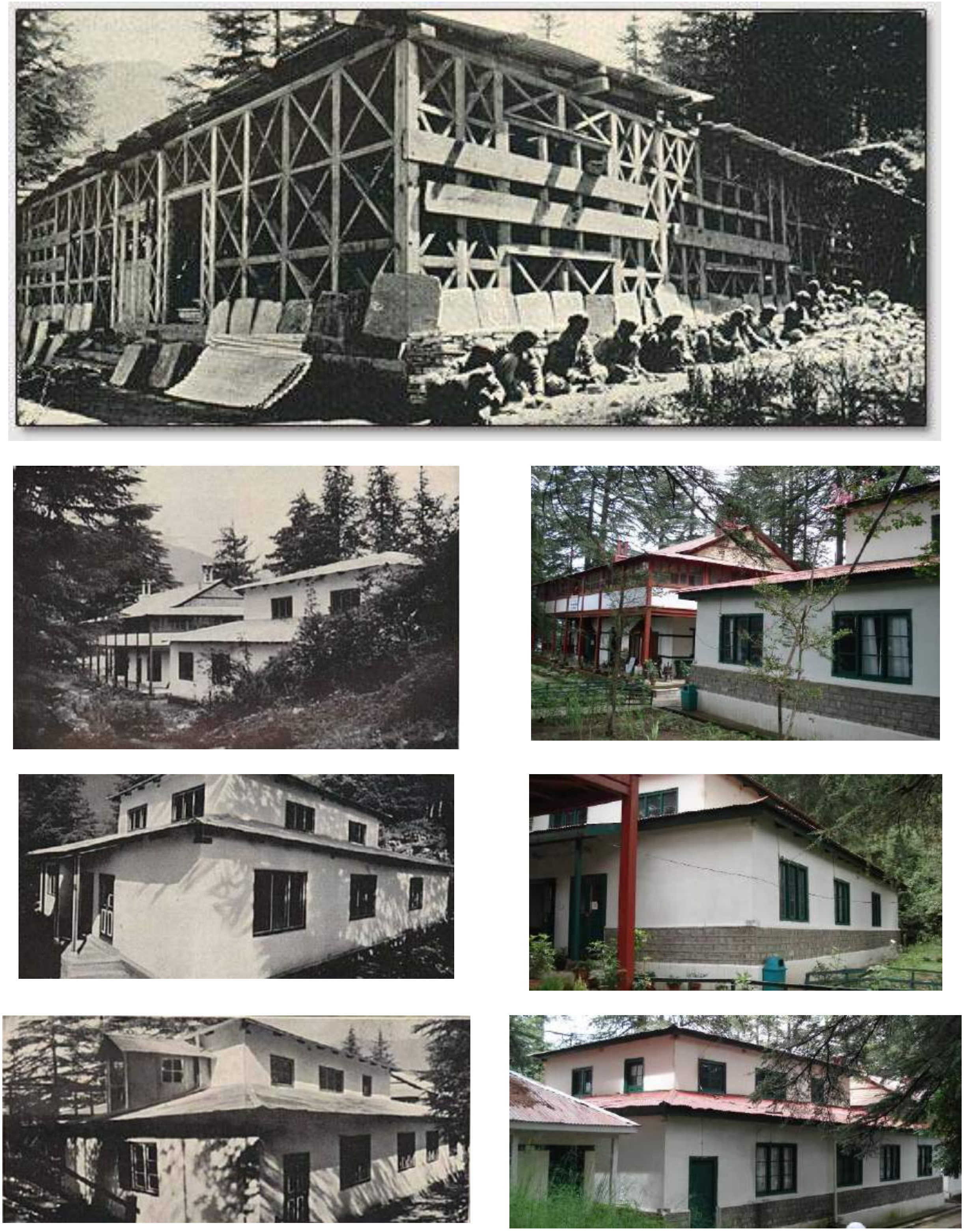


Figure A1-5. Examples from Northern India [Headquarters and Medical Research Laboratories at Naggar, Kulu, Punjab, India – from article written around 1930s (See Reference [25], Colour Photos taken in August 2009 (See Reference [26])

Half-timber house Britain

Photo Source: www.histariege.com/le_mas_d_azil.htm

Photo Source: Hans Peter Schaefer (found on the www)

Figure A1-6. Examples of similar construction types from Britain, France and Germany

Taquezal / Bahareque (source Wilfredo R. Rodríguez H.)

Taquezal is the term used in Nicaragua, and Bahareque in El Salvador)

Quincha (source unknown)

Figure A1-7 Examples of similar construction from Venezuela and South America.

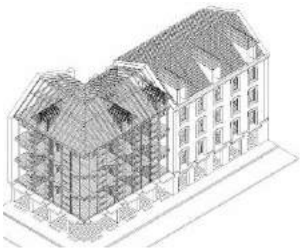


Figure A1-8 Examples of “Gaiola” construction from Portugal (See References [12]

(See Reference [22])

Figure A1-9 Examples of hımış construction from Turkey

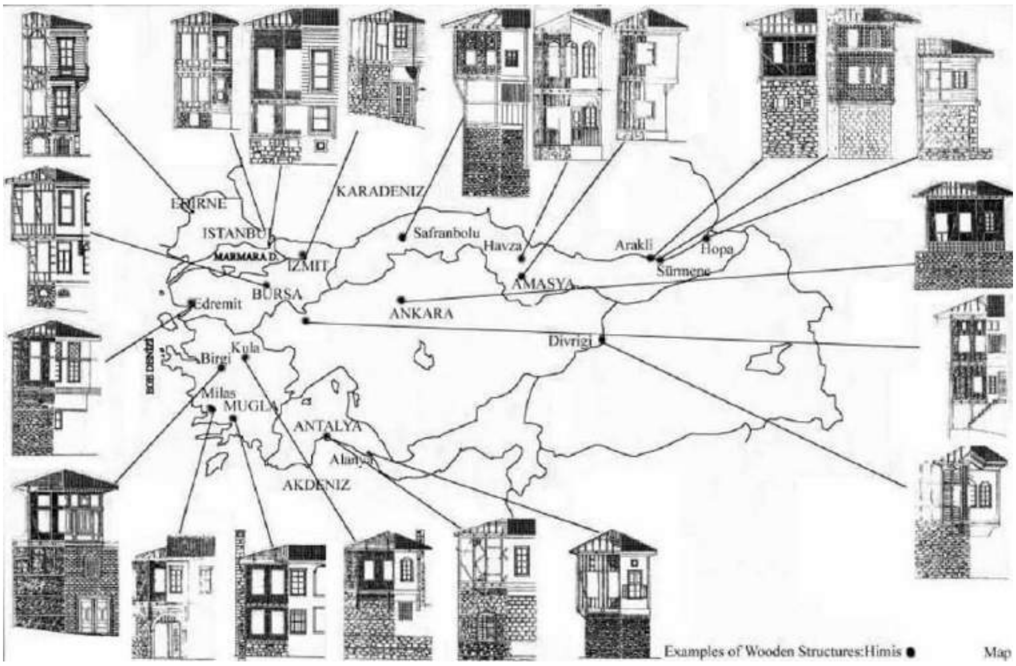


Figure A1-10 Examples of similar types of construction from Turkey (See Reference [13])

Figure A1-11 Examples of similar types of construction from Nicosia, Cyprus

Photo source: Alliance-Haiti

Photo source: Conor Bohan (http://haiti-patrimoine.org/?p=103)

Figure A1-12 Example of similar types of construction from Port au Prince, Haiti

A2 Field observations from Pakistan on Dhajji Dewari Construction

A2.1 Variability of Dhajji houses and anticipated behaviour – prior to engineering analysis

There are many ways in which reinforced concrete, steel, masonry or modern timber framed buildings are engineered and constructed even though there are many established engineering codes and industry standards.

However, unlike buildings made from these materials, the Dhajji construction form is even more variable and is not covered by building codes or established industry standards anywhere in the world.

It is clear that to date this construction form does not follow any firm construction principles. Furthermore, given that there are no codes or guides around this building system, the engineering challenge to analyse such structures is beyond the capability of most practicing engineers. This is because the building components are very variable, the building details are different for each building, the construction typology is generally not taught as part of engineering courses and the usual software tools used in structural and civil engineering are not designed for analysing and designing such structures.

For the purposes of documenting the engineering understanding of Dhajji construction, brief explanations were written down (prior to undertaking the detailed engineering analysis) to explain how we think the Dhajji system behaves structurally under earthquake loading. The purpose of this is in part to document the assumed knowledge and in part to serve as a back check to explain the analysis results.

A2.2 Foundation

The building has a shallow foundation typically consisting of a rubble stone (field stone) strip footing. Typically, these buildings have shallow dug foundations without any proactive drainage provisions around the timber frame base.

Nowadays solid masonry (not concrete blocks or hollow clay tiles) or even nominally reinforced concrete may be used to form the shallow foundations. It is not thought that any anchorage will have been traditionally provided between the timber frame and the strip foundations.

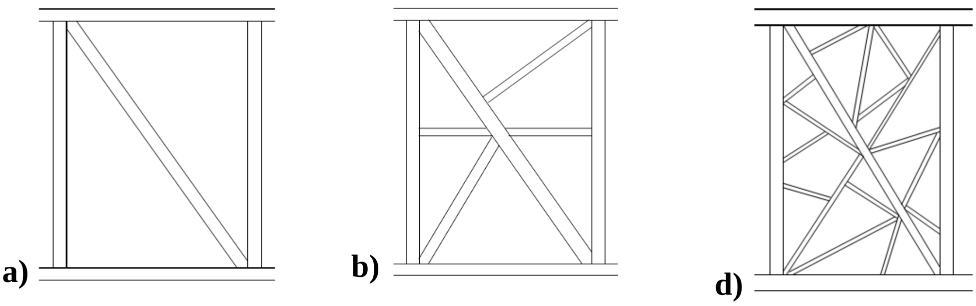
It is not thought that anchorage of the posts to the foundation will traditionally have been undertaken. Examples are shown in FigureA2-3, FigureA2-10 and FigureA2-11. During reconstruction efforts after the 2005 Pakistan earthquake connections between the posts and the foundation has been recommended in the ERRA guidelines (See FigureA2-7, FigureA2-8 and Reference [3]).

A2.3 Timber Frame System

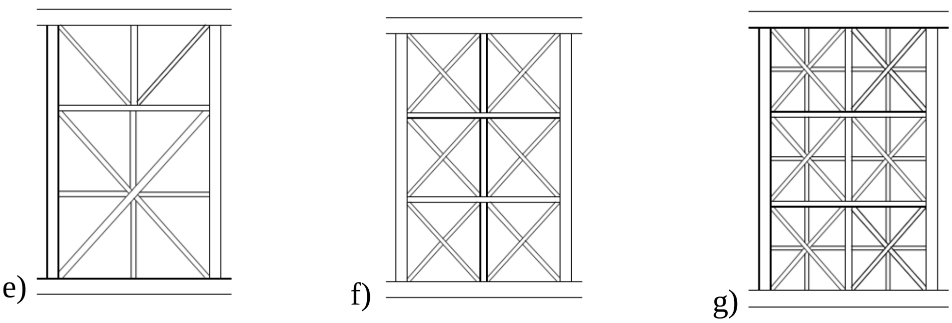
Dhajji dewari is a timber framed building with infill masonry wall panels. Traditionally timber posts and beams frame between one another with no special connections, apart from the occasional mortise and tenon joints. Currently extensive use is made of nailing and sometimes metal strapping. The timber frame is extensively braced, with small timber sections filled with stone/brick masonry infill laid in mud mortar.

A2.4 Infill

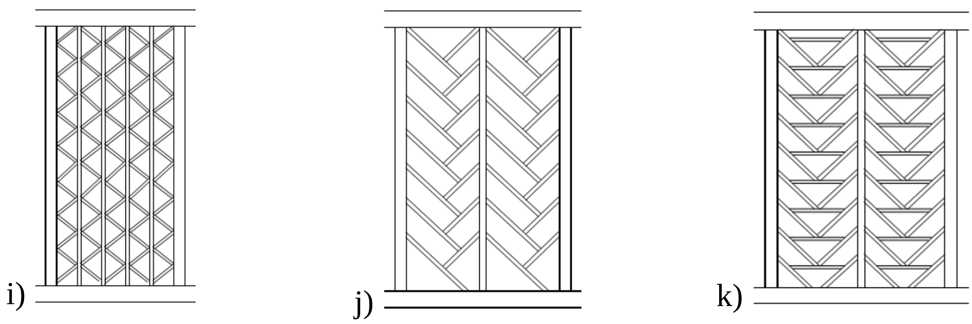
Generally stone infill is added between the extensive bracing patterns. There are no firm principles that are used to decide on the most appropriate bracing pattern construction form to adopt. In other words, the location of the principal timber columns, the secondary frame members and the extent, location and configuration of the adopted bracing pattern depends entirely upon the choices of the home builder/carpenter. Examples of some idealised bracing patterns observed in the field after the 2005 Pakistan earthquake are shown in FigureA2-1.



a) to d) - braced frame with increasing levels of random subdivisions



e) to g) – frame with intermediate columns and regular cross bracing with increasing levels of refinement going from left to the right



h) to k) – frame with intermediate columns and various regular bracing patterns

FigureA2-1.Typical bracing patterns being used post 2005 Pakistan earthquake.

The infill fulfils functional (enclosure and partitioning) and structural requirements.

Because of the low infill panel strength and high flexibility of the timber frame (due to the generally loose timber connection) the in-plane wall panels crack in the very early stages of ground shaking. This softens the frame and has the effect of immediately decoupling the Dhajji buildings period of vibration from the likely high energy content period range of an earthquake. This results in reduced inertial forces being imposed on the building. It is thought that the first phase of earthquake response is movement along the masonry-timber interfaces, before the masonry itself is stressed enough to begin to crack.

It is thought that the cracking and sliding of masonry units along mortar joints increases the damping levels in the building thereby helping to dissipate energy and reduce the earthquake loads acting on the building.

During long duration earthquakes, a few isolated infill panels may topple without jeopardizing the stability of the building as the timber frame essentially remains elastic and provides vertical load path and lateral stability to the building structure assuming adequate coupling of the perpendicular walls is provided.

The closely spaced timber framing and bracing mitigates out-of-plane toppling of the infill walls. These elements provide support points from which the masonry panels can retain their stability through arching action. This ensures that the out-of-plane friction forces are greater than the out-of-plane inertia forces acting to dislodge the infill from the walls. It is important that long walls are regularly connected to perpendicular walls to avoid rigid body global out-of-plane failure of wall panels.

A2.5 Gravity Load-Resisting System

The vertical load resisting system in a Dhajji building is through the timber framing. Because the stone/brick masonry infill with mud mortar is placed into the frames after the building frame has been built it is not thought that the infill carries any of the vertical loads until the building settles. With time the timber frame deforms under permanent gravity loads and the timber shrinks as it dries out. It is thought that this compression of the infill panels is in part responsible for their stability during out of plane shaking. In multi-storey dhajji dewari buildings additional vertical load is exerted on to the lower walls from the weight of the upper floors.

In the case that a building is extended upwards at a later date or of multi-storey construction some degree of vertical loading of the complete lower infill walls will occur.

A2.6 Lateral Load-Resisting System

The lateral resistance of a Dhajji building comes from a combination of the extensively braced timber frame with stone/brick masonry infill laid in mud mortar. This combination of timber framing and masonry infill resists the earthquake loads in a composite way.

Because of the weak mortar, the masonry infill panels quickly crack in-plane under lateral loads and thereby absorb energy through friction between the infill material and hysteretic behaviour of the many mud layers that form the mortar between the stones/bricks and timber framing and bracing. The timber frame and closely spaced bracing, which essentially remain elastic, prevent any large cracks from propagating through the infill walls. The framing provides robust boundary conditions for the infill material to arch against and thus resist significant out of plane inertial loads. Because the framing and bracing is so extensive, it is possible to build the walls out of relatively thin masonry panels. This helps to reduce the mass of the building and therefore the inertial forces that must be resisted by the building system during an earthquake.

A2.7 Typical Building Dimensions

The length of a typical Dhajji building is 10m to 20m and the width 5m or more. The building has 1 to 4 storey(s). The typical span of the roofing/flooring system is 3-4m. The typical distance between walls (frame + infill wall) depends on room size but is estimated to be around 3m to 5m.

Distance between columns is typically 1m. The typical storey height in such buildings is 2.5 to 3.5m. The typical structural wall density is up to 10 % the foot print of the house.

A2.8 Floor and Roof System

The flooring system consists of wood planks and/or beams. In other words the timber columns should be connected by primary timber beams. Secondary timber beams span between the primary beams with timber floor boards that are likely to be nailed to the secondary beams. In traditional Dhajji buildings it will have been common for the timber floor to be overlain by a mud screed for levelling purposes.

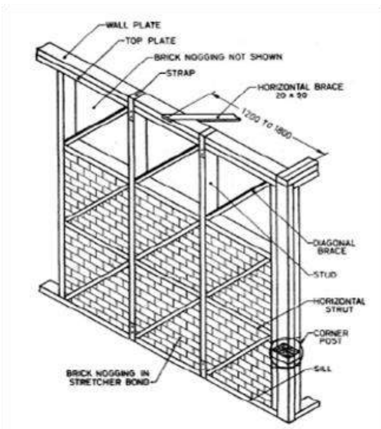
The mud screed serves the secondary purpose of fire protection to the structural timber floor. In the case that the floor beams have been covered with wooden floor boards it is thought that the floors, although not rigid are sufficiently stiff to distribute lateral loads to the Dhajji wall system.

The roofing system consists of wooden trusses clad in metal, asbestos, cement or plastic corrugated roof sheets. The roofing system typically consists of timber A-frame trusses spanning between principal timber columns, though this is not always the case. Sometimes the timber trusses are found to span between primary beams rather than columns. The timber trusses are typically configured to form a gable roof or even better in a hipped roof configuration. Hipped roofs are preferred as they have better all round stiffness properties compared to roofs with gable ends.

Traditionally, rough cut purlins were used to span between the roofs trusses on to which shakes (wooden roof tiles) were placed as the weather surface. More recently the roof covering has been made of various types of sheeting such as metal, asbestos, cement or plastic corrugated sheets. The authors do not know of cases where clay tiles have been used on these types of buildings in Pakistan or India.

A2.9 General images

This section illustrates the variability of Dhajji construction through a selection of photos.



FigureA2-2 Sketch of a Dhajji building from the Indian Building Code IS-4326

FigureA2-3 Foundation below plinth beam, note no connection between plinth wall and timber beam (existing building)

FigureA2-4 Stone strip footing under construction to raise the timber frame off the ground (new building)

FigureA2-5 Strip footing made from stone with concrete capping and embedded steel reinforcement

Where the timber frame is founded on stone, the stone will provide a natural damp proof course to the building. More recently, where concrete is used in the foundations, a damp proof course is typically missing. This will allow moisture to travel up to the timber frame and thereby threatening the longevity of the frame.

FigureA2-6 RC plinth band under construction (note poor splicing of stirrups)

FigureA2-7 Bolts in the foundation to tie-up plinth timber

FigureA2-8 Bolting of plinth band with foundation. Note that a damp proof course has not been used

FigureA2-9 Timber ground beam (or sill beam) raised above the ground to protect the timber frame from rotting. Note attempted connection of the timber beam with the foundation. Damp proof course not provided.

FigureA2-10 Corner connection detail with generous overlaps.

FigureA2-11 Post and plinth beam connection details. Note that the timber frame is founded directly on stone which will help keep the timber dry.

FigureA2-12 Timber frame ready to receive stone infill. Bracing has a zigzag pattern and bracing does not coincide/node-out.

FigureA2-13 Timber frame ready to receive brick or stone infill, bracing is very much in the form of x-bracing.

FigureA2-14 Construction of wall with stones and mud mortar. The planks on the other side of the wall act as formwork and will be removed after the infill placement is completed.

FigureA2-15 A wall ready to receive final coats of mud plaster (view taken from inside a building).

FigureA2-16 A completed wall, note large stones

FigureA2-17 A completed wall. Note that the entire building has been raised above the ground level by a stone masonry foundation.

FigureA2-18 A Dhajji building with gable wall with planks

FigureA2-19Connection of first storey post with second storey post with a cylindrical wood member passing through floor beam (Simla, India)

FigureA2-20 Tie-up of roof with wall structure

FigureA2-21 A view showing roof and braced timber frame

FigureA2-22 Mixed bracing pattern making uses of salvaged timber and window frames.

FigureA2-23 Internal view of completed wall. Note that the timber frame is not protected from getting wet as it is built straight in to the ground.

FigureA2-24 Timber, stone laid in concrete. This is not traditional Dhajji construction. There is a risk that the entire panel may fall out as a rigid object during an earthquake.

FigureA2-25 An internal view of a hipped roof. Apart from the CGI sheets there is no bracing to stiffen the roof structure

FigureA2-26 Timber connection details. Unless nails have been used this connection will not have very limited tension capacity to poor joint interlock.

FigureA2-27 Simple strapping of nailed scarf joint will provide confinement to the joint and help increase its capacity. The nailing increases the scarf joints tensile capacity.

FigureA2-28 Strapping around the timber will help confine the scarf joint; long straps will help with tension as will the nails.

FigureA2-29 Poorly built scarf connection. The joint is a poor fit and the wooden peg that should pre-stress the joint appears loose. Note that the joint has been crudely reinforced with nails.

FigureA2-30 Common random bracing in small panels

FigureA2-31 X-Bracing pattern using partially cut stone

FigureA2-32 Bracing using large timber braces as timber resources are less constrained in the higher altitude. Note the more layered infill effect due to the type of locally available rock

FigureA2-33 Large panel that appears to be poorly in contact with the timber frame. Lack of tight fit may result in the entire panel falling out-of-plane. Again note the layering of the infill due to local rock characteristics

FigureA2-34 Mixed reinforced concrete and timber framing in Dhajji style using X-bracing pattern

FigureA2-35 Reasonably cut stone laid with a lot of mud mortar. Note that the timber has been soaked in old engine oil as a way to give it some better protection against rotting.

FigureA2-36 Wall built before the 2005 earthquake

FigureA2-37 Wall built after the 2005 earthquke

FigureA2-39 “Engineered” Dhajji building frame under construction

FigureA2-38 Zig-zag bracing that nodes out. Note that the roof is erected early as it helps to stabilise the walls before their completion and shelter the masons doing the noggin and interior finishes. Also note the hipped roof giving stiffness in both principal building directions.

Note anchor rods in the foreground and note that a damp proof course is not used.

FigureA2-40 Inside view of a 1 storey house under construction. Note that the timber frame is in direct contact with the ground.

FigureA2-41 preferred roof framing for those who can afford more

FigureA2-42 Anchor bolt in stone masonry wall laid in sand and cement mortar. Note that the bolt is already rusting and is unlikely to receive any rust treatment prior to connecting to the frame base plate

FigureA2-43 base plate corner detail and bolt anchorage holes. Note that orthogonal walls are already planed and interconnected with the perimeter timber ring beam.

FigureA2-44Mixed construction form. Dry stone wall with Dhajji timber frame on top but without traditional infill material. CGI sheets are used as cladding instead.

FigureA2-45 Internal view of large room with partially completed infill walls. Note the lack of bracing in any direction from the roof and apparently large distances between orthogonal walls. The system may be too flexible without enough support to the walls.

FigureA2-46 Double storey hipped roof built in to the slope on one side.

FigureA2-47 Simple frame at the start of the process of converting the posts into Dhajji walls with hipped roof. Note that gutters are not provided to any of the roofs.

FigureA2-48 Dhajji frame under construction. Note that the front sits on top of a retaining wall and the back is partially a retaining wall.

FigureA2-49 Full height built in wardrobe with reasonable thick solid timber back – possibly it acts in part as a timber shear wall but it is not a detail that the engineering solution should depend upon.

FigureA2-50 Dhajji frame built on a poor quality dry stone wall

FigureA2-51 Mixed wall construction. Dry stone wall on the left hand side and Dhajji construction on the right hand side

FigureA2-52 Single storey house with Dhajji walls. It was not clear if the roof damage was due to the 2005 earthquake or if this was post earthquake construction.

A2.10 Construction Materials

A2.10.1 Traditional construction materials

Traditionally Dhajji houses will have been built from the materials listed in **Table A2-1**.

Table A2-1 Traditional materials from which Dhajji were/are made

Component	Material
Foundations	Stone masonry or fired brick
Damp proof course	If the foundation was stone then the stone will act as the damp proof course.
	With masonry foundations it is not thought that a damp proof course such as slates or a thin stone layer will have been used
Wall framing	Timber posts with timber braces (limited use of nails or metal straps)
Wall fill material	Rubble or cut stone dressed in mud mortar. Mud mortar may be strengthened by the addition of lime and/or the addition of natural fibres (pine needles, goats/horse hairs). In the Vale of Kashmir (in which Srinigar is located), the infill is almost always of brick because clay, rather than rubble stone, is the locally available material.
Floor framing	Timber beams overlain with wooden planks
Floor screed	Compacted and levelled earth screed
Roof framing	Timber framing, with joinery and timber dowels and wedges
Roof cladding	Shakes (i.e. wood roof tiles) connected to timber purlins
Openings (doors, windows)	Typically timber frame arranged to go around openings, single pane glass held in place with timber beading or glazing putty
Cooking facilities	Typically simple oven like fire place made from clay founded on stone/extra thick layer of screed with chimney pipe crudely going out through the closest wall.
Plumbing	Typically not provided in rural settings, possibly copper pipes in major urban places (but this would need to be confirmed)
Sewage	Typically not provided in the house in rural places
Electrical supply	Typically not provided – existing houses have loose wires.

A2.10.2 Recent construction materials

Typical materials being used in recent Dhajji type houses construction are shown in **Table A2-2**.

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Table A2-2 Materials being used in recent Dhajji type houses

Component	Material
Foundations	Stone masonry or fired brick or concrete foundation. Likely to be stone or clay brick base with possibly a reinforced concrete ring beam at foundation ring beam level to receive the timber frame. Where a foundation is built the use of steel anchor bolts is becoming more frequent.
Damp proof course	If the foundation is stone then the stone will act as the damp proof course.
	With masonry foundations it is not thought that a damp proof course such as slates or a thin stone layer will have been used
Wall framing	Timber posts with timber braces, sometimes thin reinforced concrete columns are used with timber bracing. Nailing and use of metal strapping is gaining popularity.
Wall fill material	Low grade solid clay bricks laid in cement mortar Low grade hollow concrete blocks laid in cement mortar Dry stone infill (i.e. no morter) Rubble stone (fairly round) laid in mud mortar Hollow clay brick laid in cement mortar Large mass concrete panels
Floor framing	Timber beams overlain with wooden planks or chipboard or even with a reinforced concrete slab
Floor screed	Compacted and levelled mud screed or a reinforced concrete floor slab. Where slabs are used for the roof, water proofing is not typically provided.
Roof framing	Timber framing, occasionally may be a light metal frame
Roof cladding	Corrugated galvanised iron (CGI), asbestos or plastic sheets to form lightweight roofs. It is possible that some people might be building reinforced concrete flat roof as the roof space is often very valuable, especially in hilly areas.
Openings (doors, windows)	Typically timber frame arranged to go around openings, single pane glass held in place with timber beading or glazing putty.
Cooking facilities	Generally still based on an open fire principle but greater use of electric and gas cookers
Plumbing	Being provided more and more
Sewage	Plastic pipes
Electrical supply	Loosely provided over framing and walls – unlikely to be compliant with any electrical installation regulations

Appendix B

Current state of knowledge

B1 Perceived Theory on Dhajji Dewari

It is thought that the earthquake resistance of a dhajji dewari building is developed in the following way:

Because of the weak mortar, the masonry infill panels quickly crack in-plane thereby absorbing energy through friction between the cracks in the fill material and the hysteretic behaviour of the many mud layers. The timber frame and closely spaced timber bracing (which essentially remains elastic), prevents large cracks from propagating through the infill walls and thus provide robust boundary conditions for the infill material to arch against and resist the out of plane inertial loads. As the framing and/or bracing is so extensive it is possible to keep the masonry walls relatively thin without incurring out-of-plane infill stability problems. This helps to keep the mass of the building down and therefore limit the inertial forces that must be resisted during an earthquake. The “soft” behaviour of the system has the additional benefit of de-tuning the building from the energy rich content of earthquake excitation.

Good quality experienced craftsmen and quality timber are the vital components to ensure the proper performance of the buildings components during earthquakes.

The technology to build such a house is simple. Home owners have a large degree of control over the quality of the building materials they use because they are sourced locally from the natural environment and are not dependant on manufacturing processes.

The purpose of this work is to find out if a typical dhajji dewari building does indeed behave as is thought and ultimately test whether it can perform adequately under earthquake excitation. In other words part of the aim is to confirm that it is no accident that well built Dhajji buildings perform well during earthquakes.

B2 Current knowledge on Dhajji Dewari

There is anecdotal evidence that Dhajji buildings perform reasonably well during earthquakes. However unlike modern structural steel and reinforced concrete buildings there is very limited research that has been conducted to validate the performance of Dhajji construction. Formal identification of the critical details to ensure the reliable performance of this building system is required by the engineering community.

Much of the anecdotal evidence is based on the possibly selective usage of photographs. Currently we do not know of any rigorous surveys that were performed to quantify the number of Dhajji building in a region and how these faired during earthquakes. There are no proper records of Dhajji buildings that failed or any records of the specifics of failed details of this construction form.

As demonstrated in Appendix A1 many buildings around the world have been built using similar construction techniques and therefore we should better understand this construction form.

Table B2-1is an attempt to show the existing types of publications around the Dhajji construction form(s) known to the authors and how very limited the engineering scope of these documents are. This collection of information is by no means exhaustive or complete.

The approach taken with this analysis was to be pessimistic about the building system unless the reliable performance of the building system could be demonstrated through engineering analysis. In other words; use the results from the engineering analysis as the evidence to explain and hopefully justify the adequate performance of this construction type.

Table B2-1 Current knowledge on dhajji dewari construction, Literature Review

Ref.	Existing Dhajji research	Comment
[2]	Langenbach, R., 2008, “Don't Tear it Down: Preserving the Earthquake Resistant Vernacular Architecture of Kashmir”, UNESCO, http://www.traditional-is-modern.net/	Provide common sense guidelines and makes the case that these building perform well during earthquakes – Limited engineering back up to substantiate the case
[3]	Earthquake Reconstruction and Rehabilitation Authority (ERRA), Compliance Catalogue, Guidelines for the reconstruction of Compliant Rural Houses, 06 March 2008	Common sense guidelines – based on sensible suggestions but short on engineering evidence.
[7]	Samaresh Paikara, Durgesh Rai, Confining Masonry using pre-cast RC element for enhanced earthquake resistance, Proceedings of the 8th U.S. National Conferance on Earthquake Engineering, April 18-22, 2006, San Francisco, California, USA, Paper No. 1177.	Actual test data from laboratory work. Whilst the example was not a Dhajji frame there are similarities in the structural principles and kinematics of the problem.
[10]	Anand S. Arya, Masonry and Timber Structures including Earthquake Resistant Design, Published by Nem Chand and Bros, ISBN 81-85240-05-1.	Guidelines and recommended section sizes provided in Chapter 14 of this book. No indication of the engineering background to the recommendations given in the book.
[11]	Luis F. Ramos, Pauio B. Lourengo, Seismic analysis of the old town buildings in "Baixa Pombalina" - Lisbon, Portugal, North American Masonry Conference, June 1 - 4, 2003, Clemson, South Carolina, USA.	Non-linear static push over analysis was carried out using the FE programme DIANA. Masonry walls were modelled using 2D shell elements using a smeared cracking feature. It is not thought that the analysis included any specific Dhajji type details.
[12]	Raquel Paula, Vitor Colas, Rehabilitation of Lisbon’s old “Seismic resistant timber framed buildings using innovative techniques,” international workshop on earthquake engineering on timber structures, Coimbra, Portugal, November 2006.	This paper concentrates on retrofitting techniques using modern materials such as carbon fibre wrapping. There is no actual detailed testing or analysis shown of the existing structures to validate the performance of the suggested retrofitting techniques.

Table B2-1 – Literature review, continued.		
Ref.	Existing Dhajji research	Comment
[13]	Tulay Cobancaoglu, “Himis” construction system in traditional Turkish wooden houses Historical Constructions, P.B. Lourenço, P. Roca (Eds.), Guimarães, 2001, Mimar Sinan University, Department of Architecture, Istanbul, Turkey.	Useful document to understand the prevalence of “Himis” construction in Turkey. No engineering support is shown to validate the engineering performance of these buildings.
[14]	T. Schacher, Dhajji Research Project – report of field trip in Pakistan, from 17 to 31 August 2008, University of Applied Sciences of Southern Switzerland, World Habitat Research Unit.	Good selection of field photos with occasional annotations. No engineering analysis or testing available
[15]	I.N.Doudoumis, J.Deligiannidou, A.Keseli, Analytical modelling of masonry-infilled timber truss-works, 5th GRACM International Congress on Computational Mechanics, Limassol, 29 June – 1 July, 2005.	Attempt at analysis of 2D frames using SAP 2000. It is not thought the modelling captures the kinematics or boundary conditions of the problem. It appears that timber braces in tension are taking very substantial loads but this is not thought to be possible.
[16]	E.Vintzileou, A.Zagkotsis, C.Repapis, Ch. Zeris, Seismic behaviour of the historical structural system of the island of Lefkada, Greece, Science Direct, Construction and Building Materials 21 (2007 225-236.	Linear elastic response spectrum analysis using SAP2000 was undertaken on a sample building. The kinematics of the building behaviour, joint capacities and actual damping characteristics of the building were not represented. Attempts to correlate the analysis with test results was not undertaken.
[19]	Silvino Pompeu Santos, Ensaios de Paredes Pombalinas, Nota Technica No 15/97 – NCE, Lisboa Julho de 1997.	Aware that this source exist but was unable to obtain a copy.
[21]	UET Peshawar Various on going research on full scale testing of actual Dhajji Walls and Dhajji type timber connections	Awaiting formal publication of the results but are believed to be the first known efforts to conduct real engineering testing dhajji dewari buildings.
[23]	Guidelines for Earthquake Resistant Non-Engineered Construction, http://www.nicee.org/IAEE_English.php	The section on timber is appears identical to Chapter 14 from the book by Anand S. Arya.
Error! Reference source not found.	Athanasios Dafnis, Holger Kolsch, Hand-Guenter Reimerdes, Arching in masonry walls subjected to earthquake motions, Journal of Structural Engineering, February 2002, Pages 153-159.	Dynamic test data and computer analysis to demonstrate arching action in non-load bearing masonry walls. Whilst not of Dhajji construction the work demonstrates the principle behind the Dhajji infill material remaining in place.
Error! Reference source not found.	Demet Gülhan, Inci Özyörük Güney, Behaviour of traditional building systems against earthquake and its comparison to reinforced concrete frame systems; Experiences of Marmara earthquake damage assessment studies in Kocaeli and Sakarya.	Significant statistical evidence that Hımış type structures performed significantly better than reinforced concrete frame structures during the 1999 Izmit earthquake based on detailed post earthquake damage assessment survey results.

B3 Perceived behaviour during an earthquake

The principal seismic characteristics of Dhajji buildings are presented in **Table B3-1**. **Table B3-1**has been prepared prior to undertaking the engineering analysis using engineering judgement and common sense to describe how we believe Dhajji behaves during earthquakes.

Table B3-1 Key seismic characteristics of Dhajji type buildings

Structural Element	Seismic Deficiency	Earthquake Resilient Features	Earthquake Damage Patterns
Ground	Land on which the building is built is unsafe		Ground failure leads to complete destruction of any building
Foundations	Foundations may not have been provided (or not raised enough) placing the timber frame in direct contact with the ground.	Proper strip footing provides solid foundation for the timber framing.	Rotten timber frame leading to rapid collapse of the building
	On hilly sites slopes are sometimes retained by the back wall of the house.	Build well detailed retaining wall away from the house (also ensures water seepage through the retaining wall does not entre the house directly	Retaining wall failure during earthquake leads to partial or full collapse of the house.
	General deficiency: drainage not provided to the foundations away from the building	Stone foundation extends away from the building ensuring that timber stays as dry as possible	Rotting of the timber frame base leading to failure at the base of the building and then subsequent collapse.
Timber frame base beam	Built directly on to the ground leading to rapid rotting of the timber frame	Timber base built on top of a stone base or where reinforced concrete is used for the foundations a damp proof course should be used to prevent water making the timber frame wet	Leads to collapse of the building due to loss of strength in the timber frame
	Timber base ring beam anchorage to the foundations	Fixity to the foundations ensures building does not fall of its base, especially if the house has been built on a slope where the downhill side of the building is raised much more than the back of the house	House falls off the foundations leading to local damage or, in the case of more severe drops, complete collapse of the building when adequate seat lengths / fixity is not provided.
	Traditionally anchorage will not have been provided between the timber frame and the foundation	Lack of anchorage may have provided some form of natural base isolation to the building	
	Timber base ring beam not connected to walls running perpendicular (i.e. internal walls)	Provision of timber ties for internal cross walls help tie the walls together if done at the outset	Lack of tying of the timber frame base in both principal directions allows walls to move independently leading to differential movement and thus damage.

Table B3-1 (continued) Key seismic characteristics of Dhajji type buildings

Structural Element	Seismic Deficiency	Earthquake Resilient Features	Earthquake Damage Patterns
Walls	Wall principal posts do not align with roof trusses or second floor principal beams	Alignment of principal structural members ensure a simple load path and direct load distribution	Local torsion effects are introduced and timber members and their connections perform poorly in torsion leading to failure of the building frame
	Walls posts are not properly connected to the timber base plate	Proper use of timber to timber connections ensures reversible load paths. Use of proper strapping keeps the wall posts connected to the same ring beam which reduces differential demands on the walls	Separation of framing from one another.
	When more than 1 storey it is not clear if the columns are continuous between the floors	Continuity of main timber columns ensures load path continuity but having discontinuous columns may provide a form of limited seismic isolation as long as the upper columns cannot get dislodged - Engineering study required to gain a better understanding of this.	
	Perpendicular walls are not properly interconnected	Proper connection of orthogonal wall lines from the start	Separation of wall panels and loss of mutual support leading to out-of-plane failure of walls.
Wall bracing	Too few braces resulting in large infill panels	High level of bracing ensures small masonry panels giving the masonry many lines to arch against and the bracing helps prevent crack propagation in the infill	Out-of-plane failure of the masonry infill
	Extensive use of nailing in more recent Dhajji constructions will stiffen the timber frame up considerably. it is not clear if this is a good development as a stiffer frame will attract larger seismic forces	Nailing, when done well will help keep the all the timber pieces together.	Failure of the masonry infill. Shear failure at timber connections due to inadequate edge distances.
	There are very many bracing patterns – there is no real engineering evidence that quantifies the performance between various wall bracing patterns adopted. The extensive cross bracing feels like a formal engineering solution but is more likely to be stiff and thus attract larger seismic forces. The random looking bracing patterns with many odd sized brace sections look looser and may provide better energy absorption and period elongation opportunities to the building. Detailed Engineering analysis required to evaluate this quantitatively.		

Table B3-1 (continued) Key seismic characteristics of Dhajji type buildings

Structural Element	Seismic Deficiency	Earthquake Resilient Features	Earthquake Damage Patterns
Infill	Poorly built infill – large stones that have limited planes over which energy can be lost. Limited opportunity to absorb energy by yielding the mortar material	Use of bricks or well prepared stones laid tightly to ensure good bond between each stone/brick and one another, the timber frame and the timber bracing	Out-of-plane failure of the masonry infill
	Round stones used for the infill material which will pop out when squeezed.		Out-of-plane failure of the masonry infill
	Infill made from mass concrete which will fail as a rigid body		Out-of-plane failure of the masonry infill
	Infill poorly built with lots of gaps – masonry will not be able to arch properly		Out-of-plane failure of the masonry infill
Roof level ring beam	Roof trusses do not align with principal posts introducing significant torsion in to the roof level ring beam	Ring beam helps distribute lateral forces evenly between all the columns and walls. Ring beam provides the point at which lateral support is provided to walls preventing them from failing out-of-plane	Failure of the roof ring beam and thus prop to the walls. This will then lead to failure of the walls
	Ring beam too small		Failure of the roof ring beam and thus prop to the walls. This will then lead to failure of the walls
	Ring beam splices in the wrong locations and or of poor quality		Ring beam falls apart due to high force demand and/or inadequate connection capacity
Roof	Roof trusses not aligned with timber posts	Roof trusses aligned with wall posts. Hipped roof provided rather than roof with gable end. Horizontal bracing provided to connect walls. Good quality connections (that can handle load reversals) between the roof trusses and the roof ring beam	Loss of support to walls leading to wall collapse
	Roof truss not braced vertically between trusses		
	Roof truss not braced horizontally to provide good horizontal roof level diaphragm		
	Roof truss poorly connected to roof ring beam		
	Truss bottom chord has poor quality splice	Continuous bottom chord preferably made from one member alone	Tension failure of roof truss bottom chord splice
	Pitched roof have stiffness and strength in one direction only	Hipped roofs have stiffness in both orthogonal directions	Gabel wall infill fails out of plane.

<table>
<tr><th colspan="4">Table B3-1 (continued) Key seismic characteristics of Dhajji type buildings</th></tr>
<tr><td rowspan="3">Floors (when more than 1 storey high)</td><td>Floor beams only rest on top of first floor ring beams providing limited support to the walls</td><td>Floor beams detailed with sufficient overlap and locking with main beams to be able to take reversible loads</td><td></td></tr>
<tr><td>Loosely laid floor boards that do not help distribute loads between walls</td><td>Well connected floor boards providing a strong and stiff floor diaphragm</td><td></td></tr>
<tr><td colspan="3">If floors are not well tied together but the timber framing has very generous overlaps then it might be possible that there is a degree of natural isolation between the floors – more engineering analysis is required to gain a better understanding of this construction type</td></tr>
<tr><td>Other</td><td colspan="3">Introduction of mixed systems (reinforced concrete columns with timber roof) Bracing between columns is made from timber and panels are filled with stone and mud – in principle no reason why this should not work – Needs more engineering research but could significantly help with the affordability and sustainability of construction. – There are similarities between confined masonry construction and Dhajji.
Traditionally stone buildings have stone walls that are approximately 450mm thick – no reason why this can’t be thinned right down given enough small RC vertical and horizontal RC bands (maybe made from 100mx100mm sections with 6mm bars for reinforcement + wire for confinement.
More research required.</td></tr>
</table>

B4 Indicative vulnerability

Because a Dhajji building is made from timber and mud as the mortar in the infill frames it is susceptible to rotting and decay of the timber which adversely affects the likely seismic resistance of this building type (as estimated by Kubilay Hicyilmaz prior to starting the analysis work).

A well built and maintained building is expected to have an overall seismic vulnerability rating of D = medium low (i.e. good seismic performance).

The lower bound (i.e. the worst possible) is C = MEDIUM (i.e. moderate seismic performance)

The upper bound (i.e. the best possible) is E = LOW (i.e. very good seismic performance) as shown in Figure B4-1.

A	**B**	**C**	**D**	**E**
High	High-Medium	Medium	Medium-Low	Low
		←	Well built and maintained	→
←	Poorly built and maintained	→		

Figure B4-1 Dhajji seismic vulnerability rating

A well built but poorly maintained Dhajji building is expected to be very vulnerable with an overall rating of B.

A poorly built and poorly maintained Dhajji building is expected to be highly vulnerable.

It should be noted that these evaluations are based on the subjective assessment due to the limited engineering assessments of Dhajji building based on proper calculations.

B5 Questions about the seismic behaviour of Dhajji Dewari buildings

Table B5-1attempts to articulate some of the questions that need to be answered by the engineering community.

Table B5-1 current engineering knowledge and research needs.

Building component being discussed	Available information	Research requirements.
Timber frame behaviour (at a global level)	At a global level timber frames are typically analysed using 2D sections of typical frames (columns, beams and roof truss). However, these analyses often ignore the effects of the infill material	As far as the authors know analysis of a 3D frame including representation of the infill material behaviour has not been carried out.
Timber frame behaviour at a local level (joints)	Static strength test information is typically available. How the timber joints behave under tri-directional loading is rarely tested or analysed	Tri-directional testing of timber joints, especially gain a better understanding the effects of torsion on members loaded eccentrically.
Behaviour of nailed timber joints	Nailing is being used extensively instead of proper carpentry. How this modifies the joint behaviour and the entire frame behaviour needs to be better understood	Evaluate cyclic loading of nailed joints. Evaluate this on the overall seismic behaviour of entire buildings.
Infill masonry behaviour	Behaviour of brick infill panels has been extensively carried out for in plane behaviour, principally in reinforced concrete and steel framed buildings. Most of this work assumes that the infill is made from solid brick, concrete block or hollow clay tiles. There is next to no information available on the behaviour of stone infill laid in mud mortar	Research different stone shapes (round, cut, large, small) and amounts of mortar to determine best mix for seismic performance. Could freely available stone be used in reinforce concrete frames as thin walls with thin bands of reinforced concrete confinement elements; see commonalities with confined masonry construction.
Benefits of different roof construction types (gable roof vs. hipped roof)	Would be good to quantify the seismic benefit to building frames of the two different roof construction methods of gable roof vs. hipped roof.	Analysis of buildings with different roofs needed to quantify the benefit of one roof system over another
Benefit of in-plane bracing at the roof diaphragm level	Studies are generally not available to quantify the likely benefits of incorporating these features	Sensitivity analysis required to evaluate the effect of these various details on the over all structural behaviour of Dhajji buildings
Benefit of vertical bracing between roof trusses		
Benefit of roof decking to contribute towards the roof stiffness and strength		

Table B5-1– continued, current engineering knowledge and research needs		
The benefits of different wall bracing patterns	Ideally, there is an optimum bracing pattern that uses the minimum timber volume in the bracing material and gives the best overall infill panel performance and thus whole building performance	Sensitivity analysis required to establish optimum likely bracing arrangements to minimise timber consumption and construction time.
The benefits of anchoring the frame to the foundation	Don’t currently know if this is beneficial (assuming the building will not fall off its foundation)	Require non linear time history analysis to determine if anchorage of the building frame to the foundations is beneficial or not.
Building component being discussed	Comments	Research requirements.
The effect of retaining walls as part of the rear building wall	Need to better understand how the back walls, usually made from unreinforced masonry, alter the overall performance of Dhajji buildings under earthquake loads	Analysis required to gain a better understanding of the interaction between retaining wall and Dhajji frame to resist building inertia forces as well as retaining wall forces.
The benefits of using metal strapping around joints	Metal strapping is being used extensively. However, the actual benefits, if there are any, of these straps and how they are being detailed, needs to be better understood and detailing rules developed to make best use of these metal straps.	Benefits of metal strapping to be quantified in different configurations. Question does need to be asked if extensive strapping fundamentally alters the building behaviour in a detrimental way.
The effects of poorly prepared, loose timber connections	Need to gain a better understanding of how dependant this construction form is on having good quality joiners construct the timber frames	Lab test and/or analysis to determine effect of poorly built joints.
The effects of coupling of perpendicular walls	Need to gain a better understanding of the sensitivity of coupling (or as the case may be of not coupling) orthogonal wall panels and how this affects the overall performance of Dhajji buildings	Require 3D analysis, or lab testing to understand the degree that coupling of orthogonal walls is required for Dhajji buildings to perform well.
The effects of openings on the building	Need to gain a better understanding of the extent to which openings (windows and doors) affect the performance of Dhajji building in 3D	Require assessment of whole building performance as a function of increasing openings and irregular distribution of openings
The sensitivity of the building frames to the distance between orthogonal walls.	People want to build large rooms, therefore safe distances between wall support points needs to be established. Also what is a safe Dhajji wall height and how should one calculate the typical h/t ratio for a Dhajji wall?	Sensitivity analysis needed to establish when Dhajji walls become unsafe and if there is a reasonable way of quantifying this in term of h/t ratios for Dhajji buildings.

Table B5-1– continued, current engineering knowledge and research needs		
The sensitivity of the building frame to the degree of fit of the bare timber frame (i.e. what are the benefits of pre-stress on the frame)	Don't know how this affects the performance of Dhajji frames or if shrinkage effects negate this effect anyhow	Research needed to determine how sensitive Dhajji buildings are to constructing the frames tightly with as much pre-stress as possible
The effect of having more than one storey and the implications of discontinuous timber posts	The joinery between floors needs to be better understood and the seismic performance of these buildings requires investigation	Survey of existing multi-storey Dhajji buildings is required to determine connection details between the lower and upper floors.
The effects of flexible floor diaphragms	Need to better understand if more rigid floor diaphragms could make a relatively big but simple improvement to the performance of Dhajji buildings	Perform testing or analysis to quantify the effects of various degrees of floor diaphragm stiffness.
Most of these questions are based on the needs for single storey Dhajji buildings. Clearly multi-storey Dhajji frames will have additional research questions that do need to be answered.		

Appendix C

Reconstruction using Dhajji Dewari in Pakistan after the 2005 Earthquake

C1 Post 2005 Reconstruction in Pakistan using Dhajji Dewari

Through contacts in UN-Habitats offices in Islamabad, Pakistan, it has become known that over 100,000 new homes have been constructed as dhajji dewari houses after the 2005 earthquake [1].

It is unthinkable in the developed world that housing for so many people could be built in the 21st century with virtually no scientific research. This process is essential for validation of the construction form and identification of the critical structural components of the building system so that they can be earmarked for special care and attention during the design and construction process.

Whilst the efforts of many dedicated people have gone into building new homes after the 2005 earthquake it is unfortunate that the engineering community, Governments, the World Bank and similar funding agencies have not had the foresight to encourage research into such construction forms. This is especially true with regards to the sustainability of these buildings which are more or less completely made from locally available materials with minimal fabrication requirements. Whilst no formal calculations have been made to assess the environmental impact of Dhajji houses it is assumed that their ecological footprint would be minimal.

Arup's funding for this project is a small effort to try and close the knowledge gap that exists in the engineering community for this type of construction.

A beautifully illustrated photo book called "HOME, rebuilding after the earthquake in Pakistan" documents some of reconstruction efforts after the 2005 earthquake (see Ref [27]).

Appendix D

LS-DYNA Analysis model details

D1 Geometric modelling details

This section documents the modelling details and important "Keywords" used in setting up the LS-DYNA analysis model. The "Keywords" in the analysis input file are always preceded by a "*" and this convention has been used here to help create a link between the documentation and the analysis.

D1.1 Timber Frame

D1.1.1 Timber wall structure

The timber components of the dhajji dewari walls have been modelled explicitly using solid elements. Details of these elements are given in Figure D1-1 and **Table D1-1**.

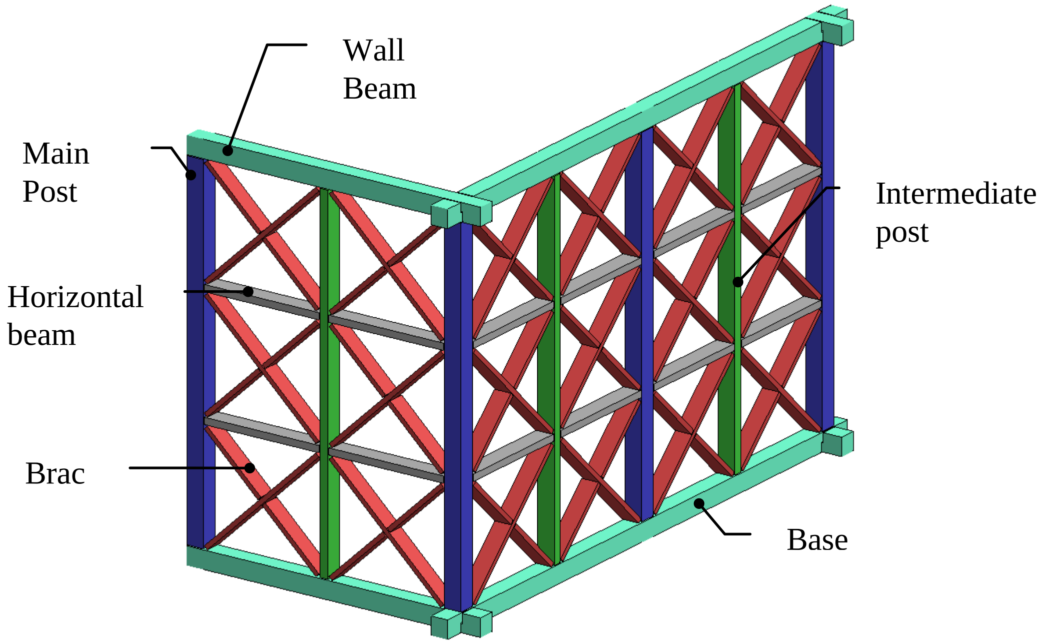


Figure D1-1 Timber wall components

Table D1-1 Element sizes for all timber wall sections modelled using fully integrated, type 2, 8 noded solids		
Description	Section	
	inches	mm
Primary Columns	4 x 4	101.6 x 101.6
Secondary Columns	4 x 2	101.6 x 50.8
Secondary Horizontal	4 x 1.5	101.6 x 38.1
Diagonal Bracing	4 x 1	101.6 x 25.5
Upper Ring Beam	4 x 4	101.6 x 101.6
Base Ring Beam	4 x 4	101.6 x 101.6

D1.1.2 Timber Roof Truss

The local behaviour of the roof was thought to be of secondary importance in these analyses. However, it was important to capture the effects of the roof structure on the overall performance of the building. To minimise computational demands, the bulk of the roof was modelled using 1 dimensional beam elements. However, where the roof and walls connect, solid elements were included to allow frictional contact and more realistic interaction at the joints between members. Further details are given in Figure D1-2 and **Table D1-2**

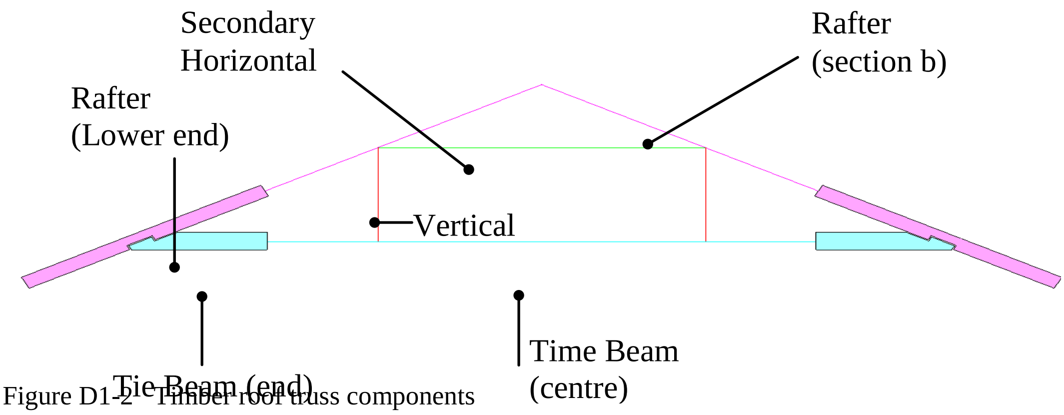


Figure D1-2 Timber roof truss components

Table D1-2 Timber roof truss component member sizes using fully integrated, type 2, 8 noded solids and integrated , Type 1, 3 noded beams		
Description	Section dimensions	
	inches	(mm)
Rafter (Lower end)	4x3	101.6 x 71.2
Rafter (section b)	4x3	101.6 x 76.2
Tie Beam (ends)	4x4	101.6 x 101.6
Time Beam (centre)	4x3	101.6 x 76.2
Secondary Horizontal	3x2	76.2 x 50.8
Secondary Vertical	3x2	76.2 x 50.8
Purlins	3x2	76.2 x 50.8

D1.2 Timber Connections

The two primary types of carpentry connection in the walls have been modelled explicitly:

Simple mortise and tenon joints (top and bottom of all primary columns).

Lap joint, in combination with a mortise and tenon joint (between perpendicular members in the upper and lower ring beams and at the end of interior walls).

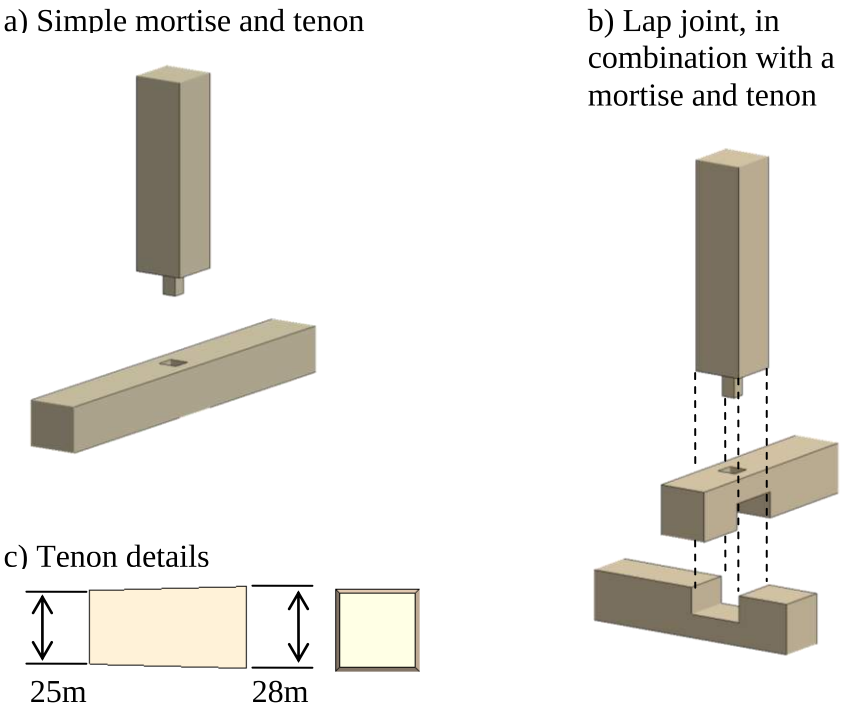


Figure D1-3 Typical timber joints

The tenons (pegs) used in both connection types have been tapered to prevent them jamming, (see Figure D1-3c) and have a minimum of 1mm clearance. To create a mechanism for shear failure of the tenons, they have been connected to columns via zero length discrete beams with stiffness and failure characteristics given in **Table D1-3**and Figure D1-4. Mortise capacities are likely to vary significantly depending on the age, condition and material used; the values below are estimates only. These elements use material *MAT_NONLINEAR_PLASTIC_DISCRETE_BEAM.

Table D1-3 Mortise characteristics for *MAT_NONLINEAR_PLASTIC_DISCRETE_BEAM.

Translational stiffness	$1x10^{6}$	N/m
Rotational stiffness	$1x10^{6}$	Nm/rad
Failure displacement	$1.5x10^{-2}$	m
Initial yield force	1000	N
Ultimate capacity	4000	N

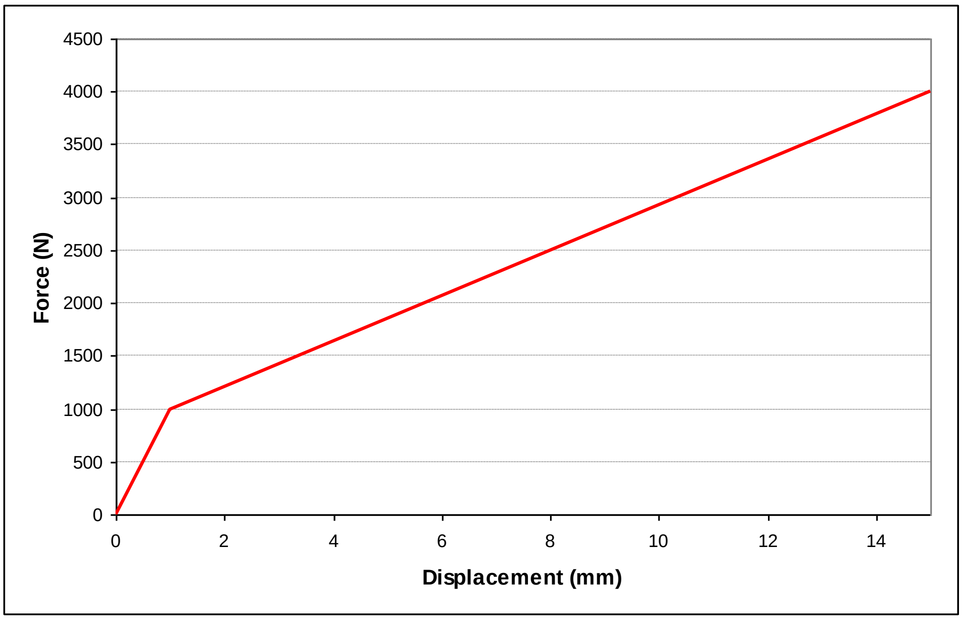


Figure D1-4 Translational force/displacement characteristics of mortise elements (as defined in LS-DYNA)

D1.2.1 Openings

Openings, consistent with the geometry provided, have been included in the model. Discrete elements connecting door and window frames have been used to simulate the estimated strength of carpentry joints in these locations, see Figure D1-5. These elements use material *MAT_NONLINEAR_PLASTIC_DISCRETE_BEAM with properties as per **Table D1-4**.

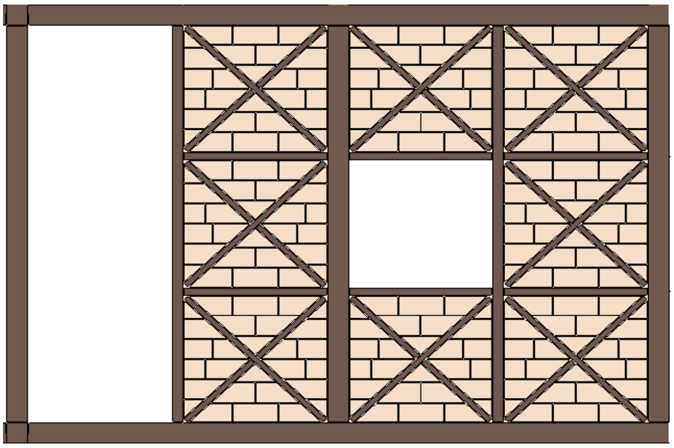


Figure D1-5 Discrete beam at openings as indicated

Table D1-4 Window/door frame connection properties for *MAT_NONLINEAR_PLASTIC_DISCRETE_BEAM.		
Translational	$1.0x10^{6}$	N/m
Rotational Stiffness	$1.0x10^{6}$	Nm/rad
Failure Displacement	$5.0x10^{-2}$	m
Ultimate capacity	5000	N

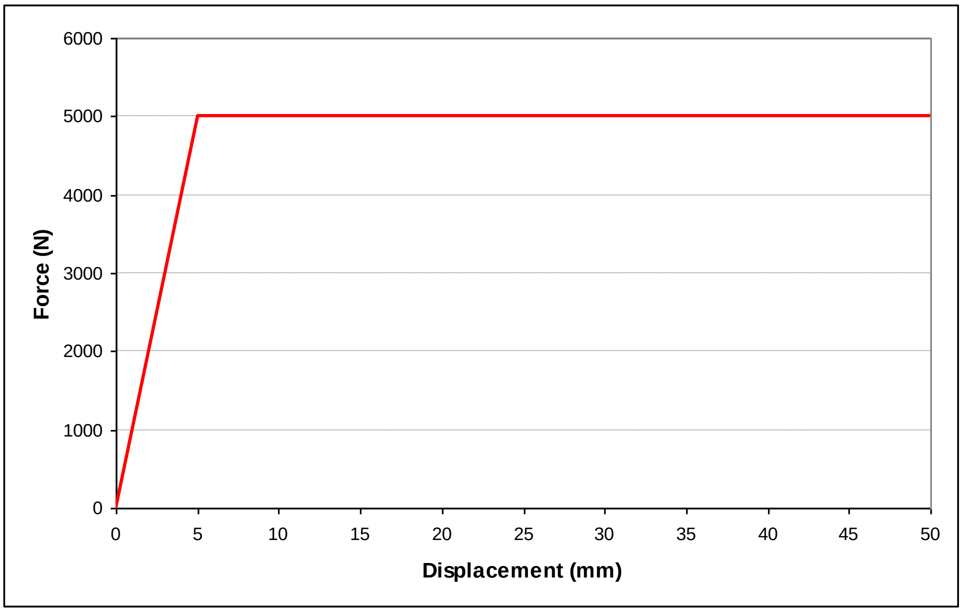


Figure D1-6 Translational force/displacement characteristics of joints (as defined in LS-DYNA)

D1.3 Metal Connections

D1.3.1 Foundation Anchorage

To represent the bolts used to anchor the timber Dhajji frame to the foundations, the model is connected to the ground via a series of springs at the base of each primary column.

D1.3.2 Metal Straps

Simple metal straps as shown in FigureA2-9, FigureA2-20, FigureA2-27 and FigureA2-28 are used frequently by builders to add some extra strength to the frames. At this stage of the engineering investigations strapping across timber components has not been modelled but should to be investigated in the future.

D1.3.3 General nailing of connections

An idealised nailed connection has been provided between all secondary horizontals and adjoining columns (Figure D1-7a), and at the top and bottom of secondary columns (Figure D1-7b). These elements use material *MAT_NONLINEAR_PLASTIC_DISCRETE_BEAM with properties as per Section D2.5.

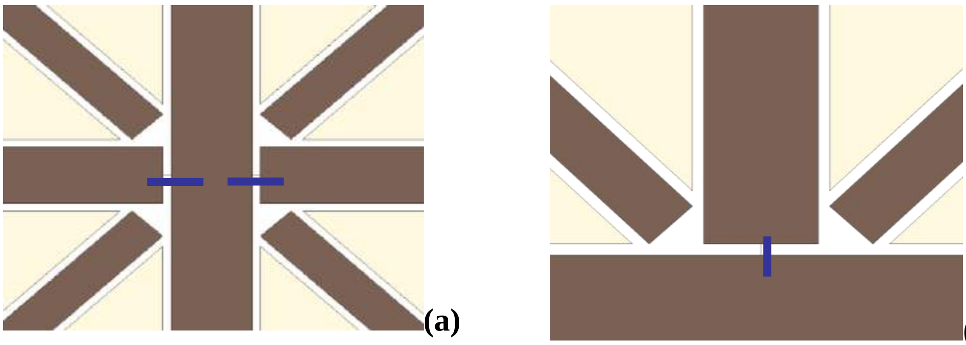

(a) (b)

Figure D1-7 Nailed connections a) secondary horizontals b) secondary columns

D1.3.4 Roof Sheeting and connections

The dhajji dewari building under consideration has a Corrugated Galvanised Iron (CGI) roof. The following details typify the CGI used:

- CGI sheets used for roofs have a gauge of around 26 (0.476mm) giving a weight of 3.662kg/m^2.
- Typically 3 rows of purlins are used for 8 feet long CGI sheets and 4 rows for the 10 and 12 feet long sheets.
- CGI sheets may typically extend approximately 4 inches beyond the eve boarding.
- CGI sheets are typically connected to the purlins with nails every 2 to 3 corrugations.
- Overlap between CGI sheets is typically 1.5 to 2 corrugations
- Typical corrugation dimensions: peak to trough depth = 1/2" or 3/8", pitch= 2" to 2 1/2"

Table D1-5 Typical CGI sheet dimensions found in Pakistani Kashmir

Length		Width	
(feet)	(Meters)	(feet and inches)	(Meters)
8’	2.438	2’-6”	0.762
8’	2.438	3’-0’’	0.914
8’	2.438	3’-6’’	1.067
10’	3.048	2’-6’’	0.762
10’	3.048	3’-0”	0.914
10’	3.048	3’-6’’	1.067
12’	3.658	2’-6’’	0.762
12’	3.658	3’-0”	0.914
12’	3.658	3’-6’’.	1.067

The CGI sheeting has been modelled using two dimensional shell elements, based on an 8’x2’-6” CGI sheet. The CGI elements are attached via discrete ‘nail’ elements to the purlins below, see Section D2.5 for nail details. This allows the roof to ‘rip apart’ if sufficient loads are applied to it.

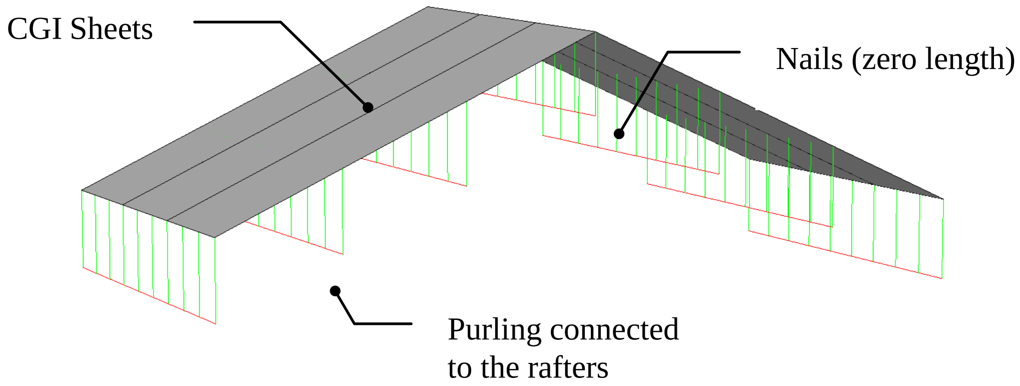


Figure D1-8 Exploded view of cladding-purlin connection

D1.4 Infill

Traditionally the infill material in Dhajji construction has consisted of stones laid in mud mortar. In reality the stones are deformable, often highly irregular in shape and laid randomly in mud mortar. The mud mortar is weak, highly deformable and variable in its application thickness. However, within a Dhajji building, the stones used for the infill are thought to be much stiffer and stronger than the other building components. Consequently, the stone in the infill remains elastic at all times. Deformations are concentrated in mud mortar, sliding between the infill stones and the surrounding timber frame.

In effect there is no simple way to model random stone infill wall panels. Therefore, to keep things reasonably simple at this stage, a regular infill mesh more analogous to infill made from masonry has been used. This allowed the model to be built in a timely manner and importantly run in an acceptable time frame. The key to modelling the infill was to model the contact surfaces stiffness between all the pieces which allows some flexibility of the infill wall whilst giving acceptable contact surface stability during analysis.

Each section of infill panel has been broken into a number of rigid blocks, with a 5mm offset between adjacent elements; these represent the stones. Between blocks, 5mm thick null shells have been used to create a reliable contact surface; and account for the deformation that occurs in the mud. The validity of this approach has been benchmarked against physical tests kindly provided by UET Peshawar [21] and detailed in Section 3.5.

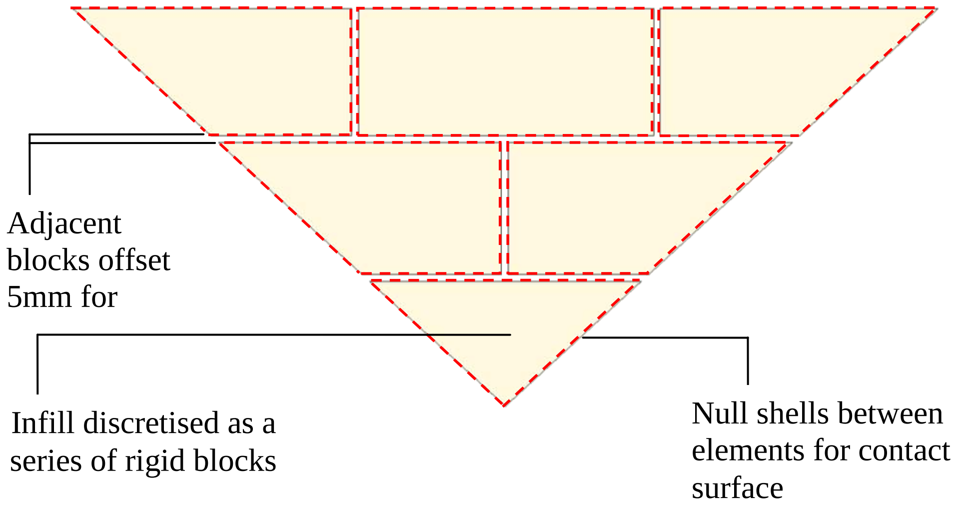


Figure D1-9 Infill modelling approach

D2 Material properties

D2.1 Timber

Essentially it is thought that the timber frame remains elastic. Therefore, the analyses were performed using elastic material properties (*MAT_ELASTIC).

The principal frame of beams, columns and braces are mostly made from softwood. The most readily available timber in the region is shown in **Table D2-1**whilst the material properties adopted for these analyses are contained in **Table D2-2**.

Table D2-1 Timber varieties use for Dhajji houses [18]

Common Names	Latin name	Strength	Young's Modulus	Poisson's Ratio	Shear Modulus	Density (kg/m^3)
Chir pine, Nakhtar	Pinus roxburghii	Moderate hard heavy	Not provided			610
Kail, Biar, Blue pine	Pinus wallichiana	Moderate hard, moderately heavy	Not provided			480
Diar, Deodar, Himalayan Cedar	Cedrus deodara	Light, soft	Not provided			570

Table D2-2 Timber Material properties

Young's Modulus (N/m^2)	$1x10^{10}$
Density (kg/m^3)	600
Poisson's ratio (μ)	0.1

D2.2 Infill

As discussed in section D1.4, the stone blocks of the infill material are modelled using *MAT_RIGID and a density of $2000kg/m^3$. The null shells between the rigid blocks use *MAT_NULL also with a density of 2000 kg/m^3.

D2.3 Anchor Bolts

The springs representing the anchor bolts are elastic (*MAT_SPRING_ELASTIC) and stiff ($K=1x10^7$ N/m). For future analyses these could be adapted to model the failure of real ground anchorage or omitted to investigate the effect of building the houses without anchorage to the foundations.

D2.4 Corrugated Galvanised Iron (CGI) Sheets

Due to the corrugated shape of the CGI roofing sheets, they have week and strong bending axes;

- Perpendicular to corrugations (high geometric stiffness results in high overall stiffness)

- Parallel to corrugations (no beneficial geometric effects, only the material stiffness contributes).

To allow this behaviour to be captured, an orthotropic material has been used (*MAT_ORTHOTROPIC_ELASTIC). This gives separate control over the elastic constants in the two in-plane directions. This option has been used to provide modelling flexibility, and in the current analyses both directions have been assigned the properties of the week axis. Typical steel material properties have been used for the CGI and are contained below in **Table D2-3**.

Table D2-3 Typical CGI Material Properties

	Parallel to Corrugations	Perpendicular to Corrugations	
Young's Modulus (E)	2.05×10^{11}	2.05×10^{11}	(N/m^2)
Shear Modulus (G)	7.88×10^{10}	7.88×10^{10}	(N/m^2)
Density (ρ)	7850		(kg/m^3)
Poisson's ratio(μ)	0.3	0.3	

D2.5 Nails

Nail properties were produced with consultation from Andrew Lawrence (Arup Timber specialist) and the relevant British Standard, BS 5268-2:2002 [20]. Details of the properties used can be found below in **Table D2-4** and Figure D2-1.

Table D2-4 Nail properties for *MAT_NONLINEAR_PLASTIC_DISCRETE_BEAM.

Diameter (assumed)	Wall Nails	Roof Nails	
	2	2	mm
Translational stiffness			
Shear	$1.0\text{x}10^6$	$1.0\text{x}10^6$	N/m
Pullout	$7.5\ \text{x}10^5$	$7.5\ \text{x}10^5$	N/m
Rotational Stiffness			
Twist	$1.0\text{x}10^6$	100	Nm/rad
Other	$1.0\text{x}10^6$	$1.0\text{x}10^6$	Nm/rad
Failure Displacement			
Shear	$1.0\text{x}10^{-2}$	$1.0\text{x}10^{-2}$	m
Pullout	$1.0\text{x}10^{-2}$	$1.0\text{x}10^{-2}$	m
Ultimate capacity			
Shear	750	750	N
Pullout	300	300	N

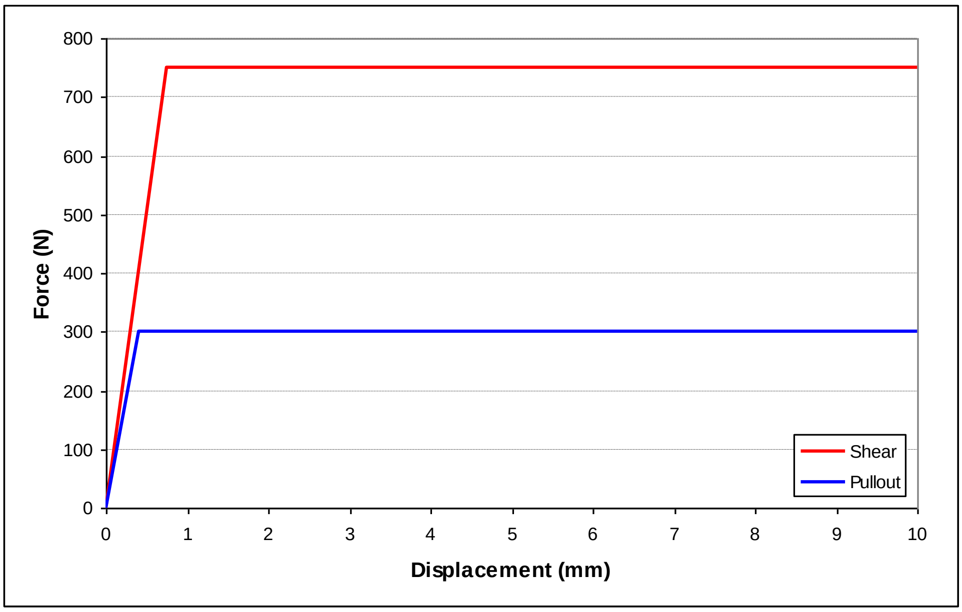


Figure D2-1 Force/displacement characteristics of joints (as defined in LS-DYNA)

D3 Contact surfaces

The analysis methodology requires a complex arrangement of contact surfaces. Recent developments in LS-DYNA have greatly improved the automated options for creating contact interfaces; however, this remains one of the most challenging aspects of analysis.

In the early stages of this project numerous approaches were tested. The challenge was to find a stable, yet efficient, contact surface which could be reproduced easily as the model developed. Many of the contact surfaces tested suffered from large penetrations (one element passing through another). This not only jeopardises the accuracy of the solution, but tends to destabilise the model and crash the analysis. To combat this issue several parameters can be adjusted:

Stiffness – Contact between materials of different stiffness can cause numerical instability, DYNA allows the user to impose a contact stiffness at the boundary between adjacent parts which in some instances resolves this issue. DYNA also has the facility to automatically calculate the contact stiffness based on stability considerations (using the SOFT card) which provides an alternative solution to this problem.

Segment-based contact options – The SBOPT card controls the penetration checking routine used by DYNA during contact calculations.

Search Depth – The DEPTH card controls additional options for the contact's search algorithm. Higher values of DEPTH can increase the reliability of a contact but have high computational costs.

Coating solid elements – For contact between solid elements, a rigorous approach to the contact definition can be achieved by coating the contacting elements with dummy shell elements (either segments or null shells). This confines the contact search to the shell elements and can improve reliability and speed.

Offset – Initial imperfections in the model geometry can lead to contact penetration problems. To prevent this from occurring, adjacent elements can be separated by tiny amounts (~0.01mm).

Most permutations of these parameters did not produce stable results. This was thought to result from the relatively course mesh of elements in the model, an underlying issue which was constrained by computational efficiency.

The following contact formulation was found to offer the best balance of stability, ease and efficiency:

*CONTACT_AUTOMATIC_SINGLE_SURFACE

Soft constraint (SOFT) =2 (contact stiffness determine by stability considerations)

Segment based option (SBOPT) =5.0 (improves calculation of warped and sliding contact)

Auto search depth (depth) =5 (checks both surface and edge-to-edge penetration)

This was applied to a set of null shells which were wrapped around the existing solid elements, see Figure D3-1

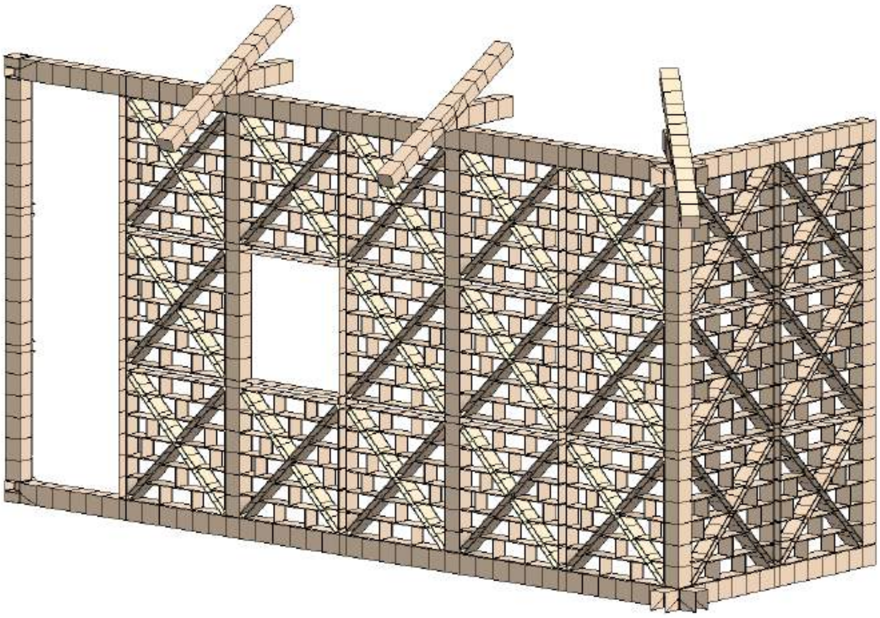


Figure D3-1 Null shells for contact surface

D4 Gravity (all analysis cases)

Gravity loading was applied globally for all analyses. Loading was increased linearly during the first 0.2 seconds of the analyses and allowed to settle, meaning that no further perceptible movements were occurring in the structure, before the application of the other loads from 0.5 seconds onwards.

D5 Summary of analysis models

During this work a number of numerical analysis models were made and run. The final models from which useful information has been obtained are summarised in **Table D5-1** and are covered in more detail in the remainder of this report.

Table D5-1 Main analysis models

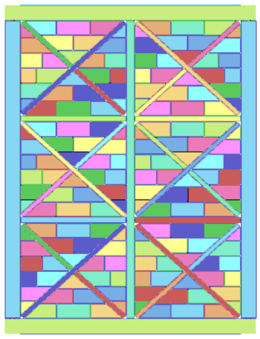
Model name	Analysis type	Loading to	Description	Model Image
Preliminary Analysis				
very_coarse_2x3_3D_6_offset_from_primer.key	Time history	In-plane sinusoidal displacement applied to base elements	Single panel, with coarse mesh. Scaled geometry to reduce run time. Contact surface options: SOFT = 2 SBOPT = 5 DEPTH = 5	
270209_with_peg_from_primer.key	Time history	Out of-plane sinusoidal displacement applied to base elements	2x3 panel. Pegs between primary vertical and horizontal members. Pinned connections between secondary vertical and horizontal members. Infill offset to create a 5mm gap with 5mm null shells everywhere.	
050309_2x3_3_corner_test.key	Time history	In plane sinusoidal displacement applied to base elements	Timber lap at corner of primary beams added to the model. Shear failure of mortise elements included as a discrete beam.	

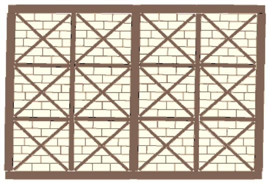
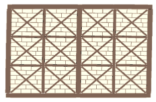
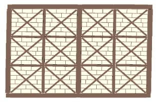
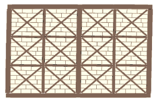
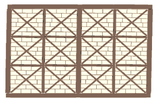
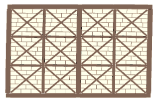
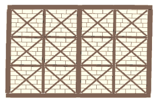
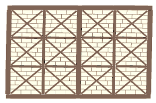
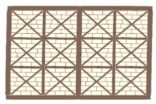
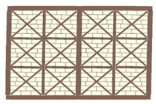
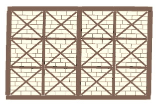
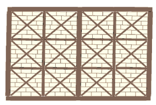
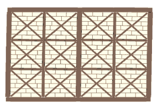
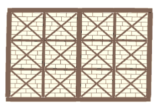
Table D5-1 Main analysis models (continued)				
Model name	Analysis type	Loading to	Description	Model Image
Benchmark Tests				
010409_4x3_dev15.key	Quasi-static cyclic pushover	displacement controlled	Cyclic pushover for benchmark with UET tests	
Full House: Quasi-static Pushover				
070509_updates_3.key	Quasi-static Pushover	displacement controlled	Pushover in long direction (with nails)	
070509_updates_3_y.key	Quasi-static Pushover	displacement controlled	Pushover in short direction (with Nails)	
Full House: Time History Analysis				
full_house_nails_090509.key	Time history	PEER1161	Full house, including nailed connections	
full_house_no_nails_090509.key	Time history	PEER1161	Full house, without nailed connections	
full_house_no_nails_090509.key	Time history	PEER 828	Full house, without nailed connections	
full_house_nails_090509.key	Time history	PEER 828	Full house, including nailed connections	

Table D5-2 Sensitivity Analysis Models (Quasi Static push over runs as variations on the benchmark analysis models)		
Analysis model name	Description	Image
original_rigid_peg_110616_RN.key	Original model with strong corner connections	
Overburden_1_rigid_peg_110616_RN.key	Over burden of 4.6kN/m	
OB2_rigid_peg_nodes_renum_110613.key	Over burden of 9.2kN/m	
OB3_rigid_peg_110616_RN.key	Over burden of 18.3kN/m	
Overburden_1_no_nails_110616_RN.key	Over burden of 4.6kN/m without nails	
OB2_no_nails_110616_RN.key	Over burden of 9.2kN/m without nails	
OB3_no_nails_110616_RN.key	Over burden of 18.3kN/m without nails	
LOF_1_rigid_peg_110616_RN.key	Brace lack of fit by 15mm	
LOF_2_rigid_peg_110616_RN.key	Brace lack of fit by 25mm	
LOF_3_rigid_peg_110616_RN.key	Brace lack of fit by 50mm	
LOF_2_rigid_peg_OB1_110616_RN.key	Brace lack of fit by 25mm with over burden of 4.6kN/m	
LOF_2_rigid_peg_OB2_110616_RN.key	Brace lack of fit by 25mm with over burden of 18.3kN/m	
LOF_2_rigid_peg_OB3_110616_RN.key	Brace lack of fit by 25mm with over burden of 9.2kN/m	

D6 Preliminary Analysis

As described in section , a series of preliminary analyses were undertaken to hone the modelling approach before constructing a complete house. The key models from these early analyses are presented below.

D6.1 Single panel models

This model represents the last in a series aimed at creating a stable contact formulation able to capture large scale deformation without an excessively long analysis time.

A single Dhajji wall panel was analysed under arbitrary in-plane sinusoidal loading (see Figure D6-1). To provide more realistic behaviour a pressure representing the weight of a single storey wall was applied to the top of the model (the timber elements are not jointed together and consequently friction is required for initial stability).

The output from this model was not an assessment of the Dhajji but confirmation of the model's stability under large deformations (as demonstrated in Figure D6-2).

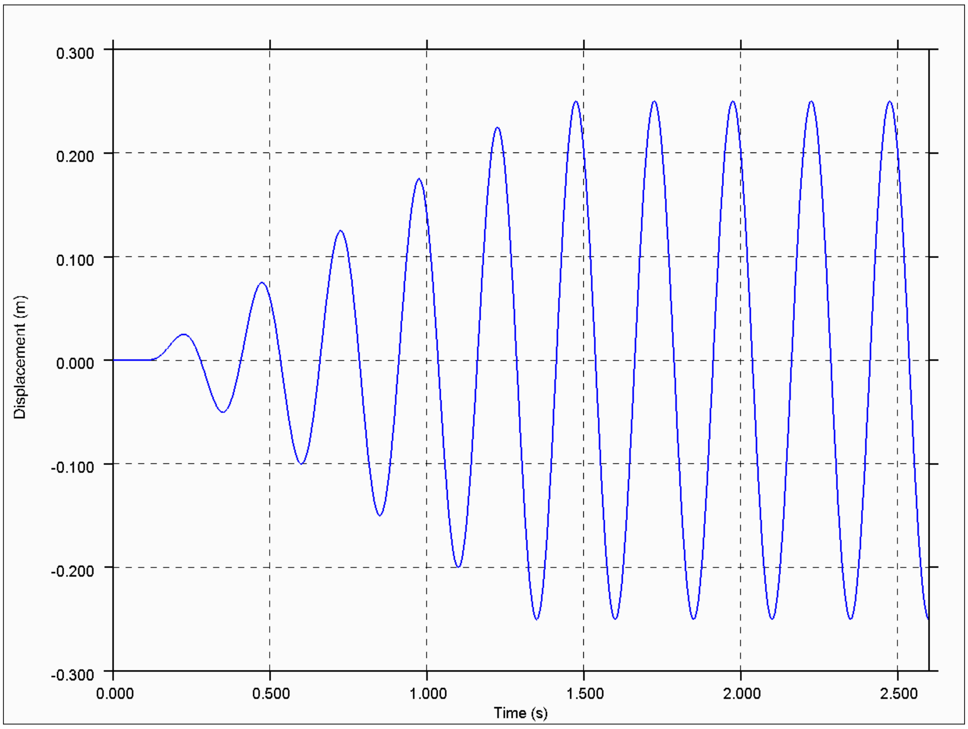


Figure D6-1 Displacement time history applied to the base of the model

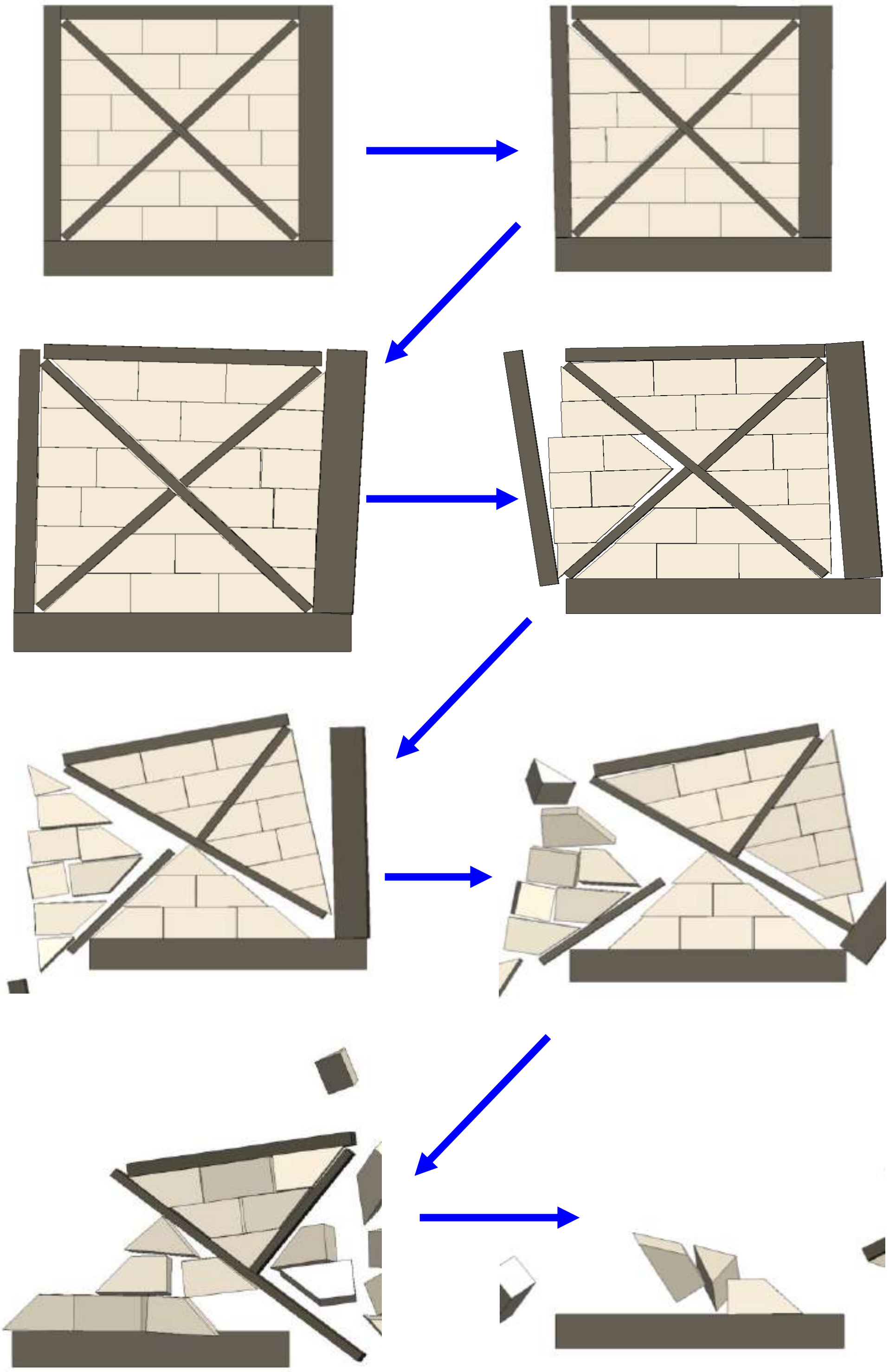


Figure D6-2 . Collapse of single panel under (arbitrary) dynamic loading

D6.2 Multiple panel models

Following the work on a single panel, a series of full height wall sections were analysed with the same arbitrary sinusoidal loading applied in and out of plane (see Figure D6-3). Once again, the purpose of these analyses was to check the stability of the contact surfaces and provide qualitative understanding of the deformation mechanisms not to assess the Dhajji. These models were also used to test the timber joints between the primary timber members (see Figure D6-3).

D6.3 Out-of-Plane loading

Under out-of-plane loading there was minimal permanent deformation of the wall (Figure D6-3). This is not likely to be representative of real performance owing to the arbitrary nature of the time-history (most notably, the sinusoidal input provides excitation at only one frequency).

Figure D6-3 . Deformation of wall section under (arbitrary) out-of-plane dynamic loading.

D6.4 In-plane loading

In-plane (arbitrary) loading also caused relatively small levels of permanent deformation (Figure D6-4).

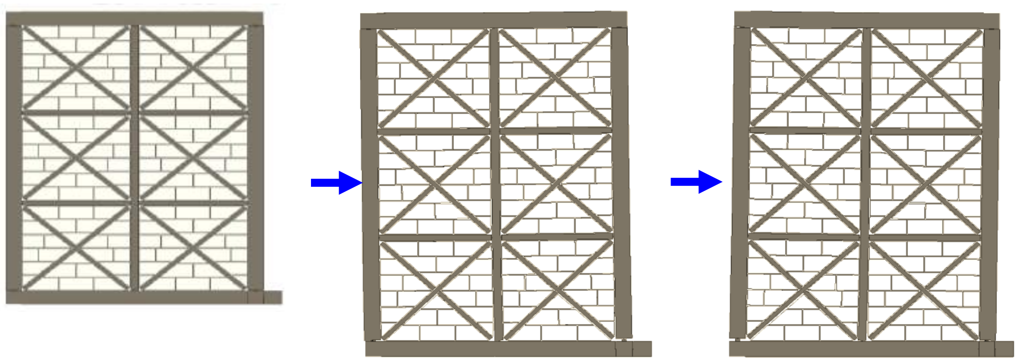


Figure D6-4. Deformation of wall section under (arbitrary) in-plane dynamic loading.

D7 Target Spectra

UBC97 was used as a basis for the spectral matching with the parameters shown in **Table D7-1**.

Table D7-1 Seismic parameters	
Zone	4
Soil Type	C
Mw	7-8
Distance	0-10km & 20-1000km

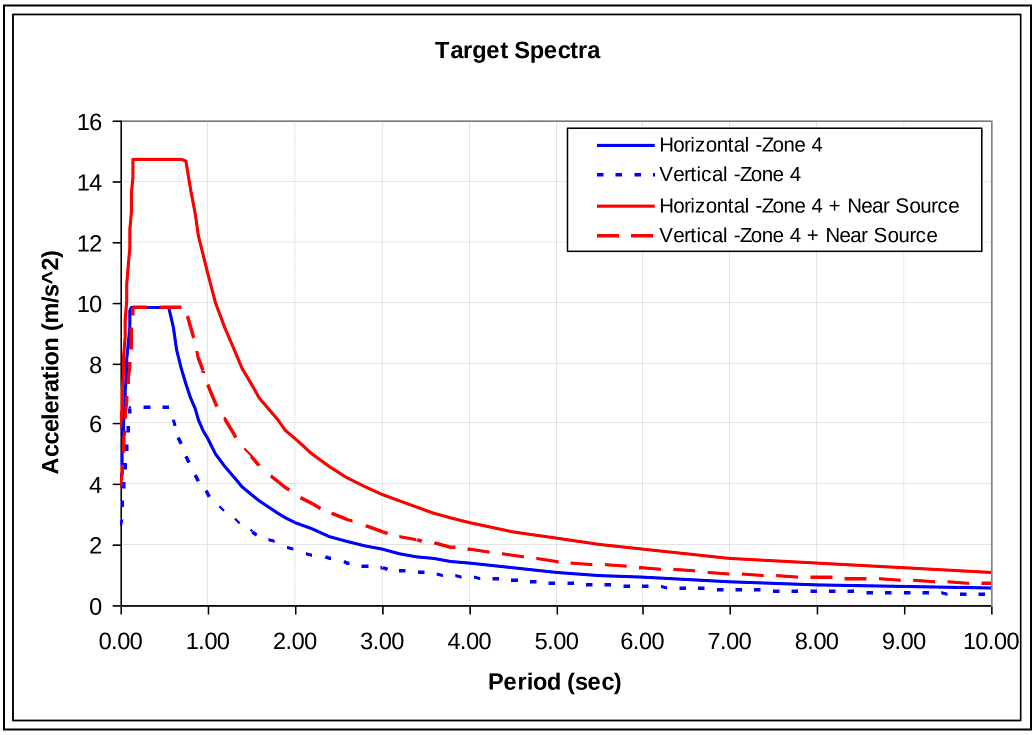


Figure D7-1 Target Response Spectra

8.1.1 Time History Selection

The program *RSPMatch2005* was used to generate spectrum-compatible records for the analyses. Unlike previous spectral matching programs that modified records in the frequency domain, *RSPMatch2005* makes adjustments to recorded accelerograms in the time domain, and therefore does not add unrealistic energy content to the records. Seed records were selected using the basic methodology outlined in Grant *et al.* (2008), whereby records with the best initial match of the target spectrum are selected, based on certain seismological filters. Ground motions were selected from the PEER NGA database, which provides extensive meta-data for each of its records, including the usable frequency range.

Two suites of ground motions were developed based on the two design spectra discussed in Section D7.

For spectrum 1, only records with moment magnitudes (Mw) between 7 and 8, epicentral distances greater than 20km, and maximum usable period greater than 2.0 seconds were considered.

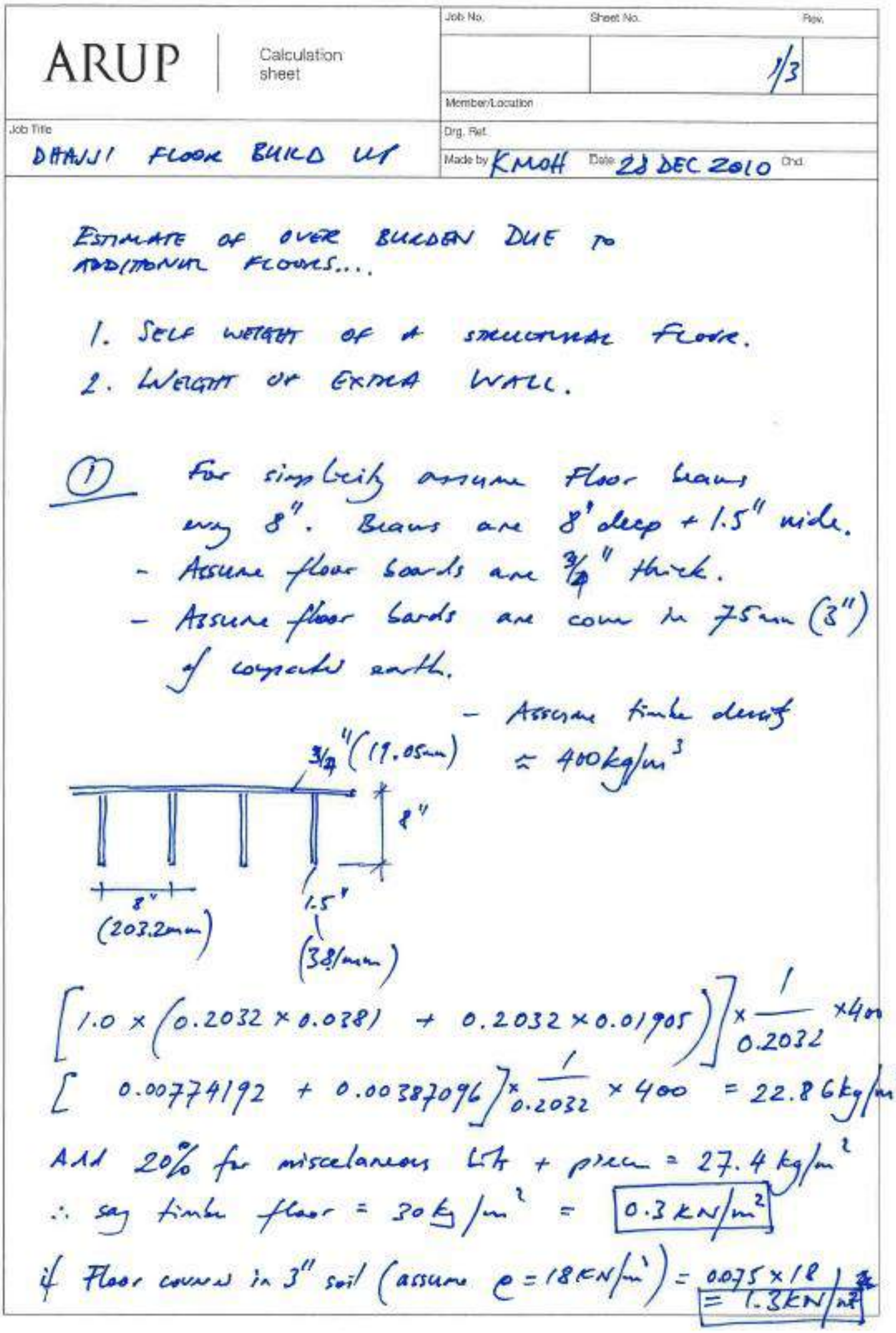
For spectrum 2, the same magnitude and usable period values were considered, but records with epicentral distances less than 10km were considered.

Summary information about the selected records, including linear scaling factor (applied before spectral matching), seismological characteristics and maximum usable period is shown in **Table D7-2**. More information can be found in http://peer.berkeley.edu/nga/.

Table D7-2 Summary information about the time history records

	Record Sequence Number	Earthquake Name	YEAR	Station Name	Earthquake Magnitude	Epicentral Distance (km)	Maximum usable period (s)	Scaling factor applied
Spectra 1	1161	Kocaeli, Turkey	1999	Gebze	7.5	47.0	10.0	2.8
	1493	Chi-Chi, Taiwan	1999	TCU053	7.6	41.2	26.7	2.3
	1776	Hector Mine	1999	Desert Hot Springs	7.1	74.3	7.7	6.4
	2107	Denali, Alaska	2002	Carlo	7.9	67.7	19.2	5.3
Spectra 2	1165	Kocaeli, Turkey	1999	Izmit	7.5	5.3	8.0	3.8
	1521	Chi-Chi, Taiwan	1999	TCU089	7.6	7.0	11.4	2.7
	1605	Duzce, Turkey	1999	Duzce	7.1	1.6	10.0	1.8
	828	Cape Mendocino	1992	Petrolia	7.0	4.5	14.3	1.4

D8 Over burden values


ARUP | Calculation sheet

Job No. | Sheet No. 1/3 | Rev.

Member/Location

Job Title: DHAJJI FLOOR BUILD UP

Drg. Ref.

Made by KMOH Date 28 DEC 2010 Chd.

ESTIMATE OF OVER BURDEN DUE TO ADDITIONAL FLOORS...

1. SELF WEIGHT OF A STRUCTURAL FLOOR.
2. WEIGHT OF EXTRA WALL.

(1) For simplicity assume floor beams every 8". Beams are 8" deep + 1.5" wide.
- Assume floor boards are 3/4" thick.
- Assume floor boards are cover in 75mm (3") of compacted earth.

- Assume timber density ≃ 400 kg/m³

3/4" (19.05mm)

8"

8" (203.2mm)

1.5" (38mm)

$$\left[1.0 \times (0.2032 \times 0.038) + 0.2032 \times 0.01905\right] \times \frac{1}{0.2032} \times 400$$

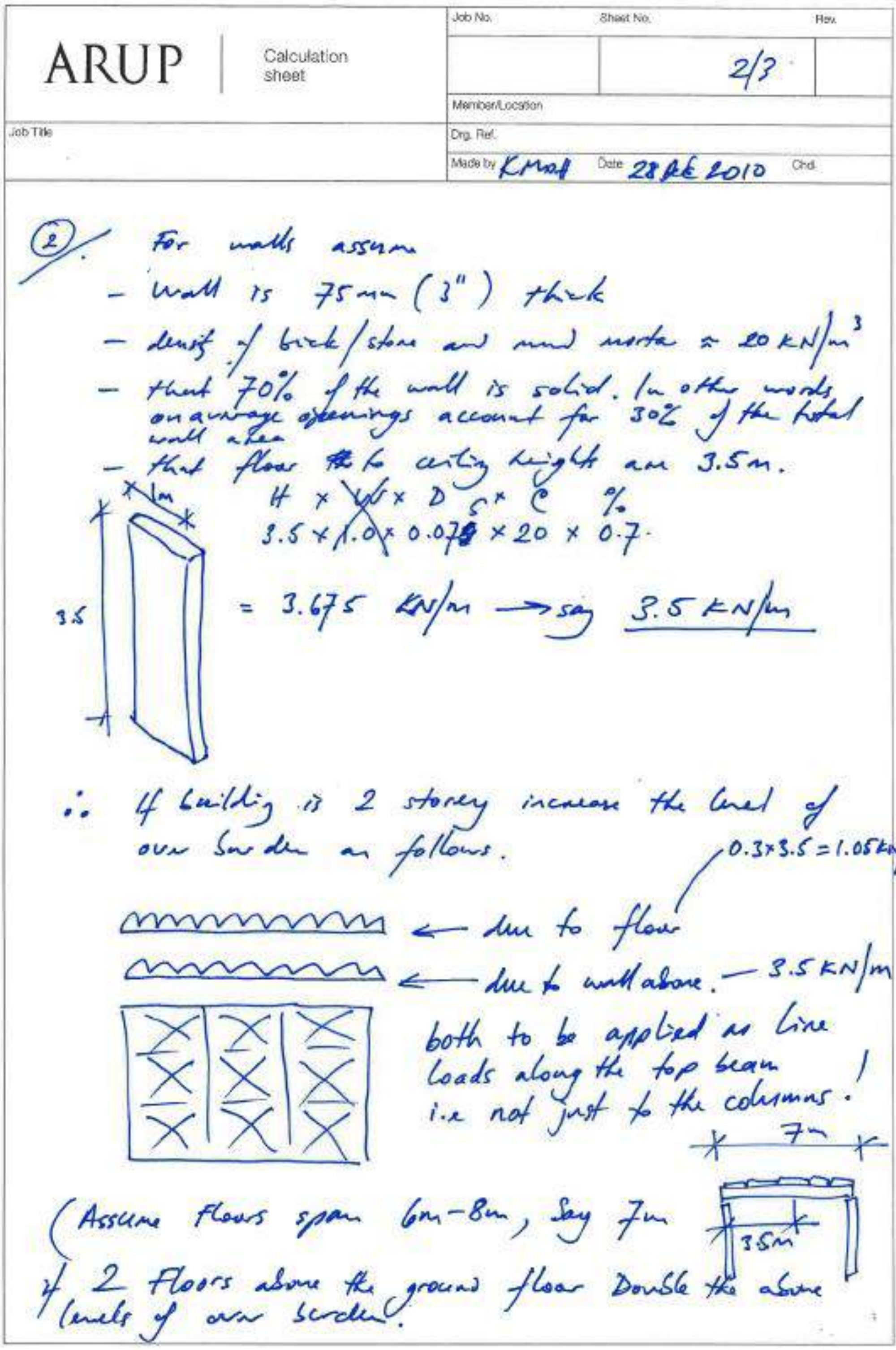
$$\left[0.00774192 + 0.00387096\right] \times \frac{1}{0.2032} \times 400 = 22.86\ kg/m^2$$

Add 20% for miscellaneous wt + pieces = 27.4 kg/m²

∴ say timber floor = 30 kg/m² = $\boxed{0.3\ kN/m^2}$

if Floor covered in 3" soil (assume ρ = 18 kN/m³) = 0.075 × 18 $\boxed{= 1.3\ kN/m^2}$

Figure D8-1 Hand calculations for over burden levels Sheet 1 of 3


ARUP | Calculation sheet

Job No.	Sheet No.	Rev.
	2/3	

Member/Location

Job Title

Drg. Ref.

Made by KMcF Date 28 Apr 2010 Chd.

(2). For walls assume
- wall is 75mm (3") thick
- density of brick/stone and mud mortar ≈ 20 kN/m³
- that 70% of the wall is solid. In other words on average openings account for 30% of the total wall area
- that floor ~~to~~ to ceiling heights are 3.5m.

1m

H × W × D × ρ × %

3.5 × 1.0 × 0.075 × 20 × 0.7

3.5

= 3.675 kN/m → say 3.5 kN/m

∴ If building is 2 storey increase the level of over burden as follows.

0.3×3.5 = 1.05 kN/m

← due to floor

← due to wall above. — 3.5 kN/m

both to be applied as line loads along the top beam i.e. not just to the columns!

7m

3.5m

(Assume floors span 6m–8m, say 7m

If 2 floors above the ground floor Double the above levels of over burden.)

Figure D8-2 Hand calculations for over burden levels Sheet 2 of 3

ARUP | Calculation sheet

Job No.	Sheet No.	Rev.
	3/3	

Member/Location

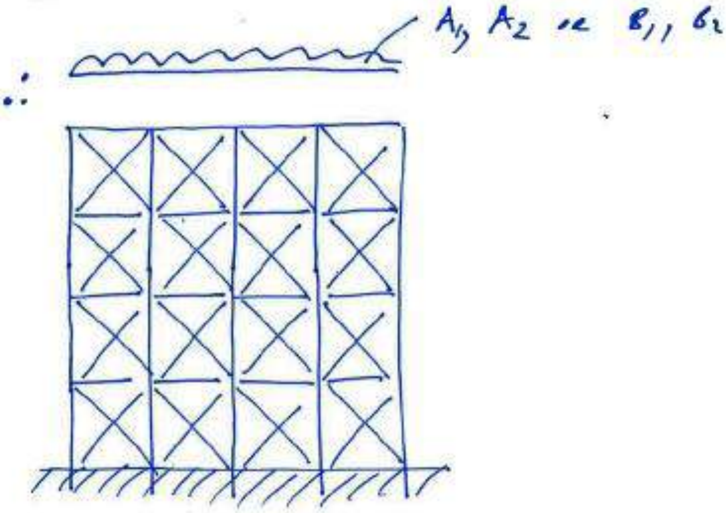
Job Title: DHAJJI FLOOR BUILD UP

Drg. Ref.

Made by: KMoff Date: 28 DEC 2010 Chd.

if Floor cured in Earth (3") then an additional load of $1.3 \times 3.5 = 4.6$ kN/m needs to be applied to the single wall as a line load.

∴



$A_1 = (1.05 + 3.5) = 4.55$ kN/m
$A_2 = (1.05 + 3.5 + 4.6) = 9.15$ kN/m
} For 1 extra floor level.

$B_1 = 2 \times 4.55 = 9.1$ kN/m

$B_2 = 2 \times 9.15 = 18.3$ kN/m

∴ Suggest we do 3 levels A_1, A_2 or B_1, and B_2

[Timber Section sizes will not be increased in these runs].

Figure D8-3 Hand calculations for over burden levels Sheet 3 of 3

D9 Lack of fit values

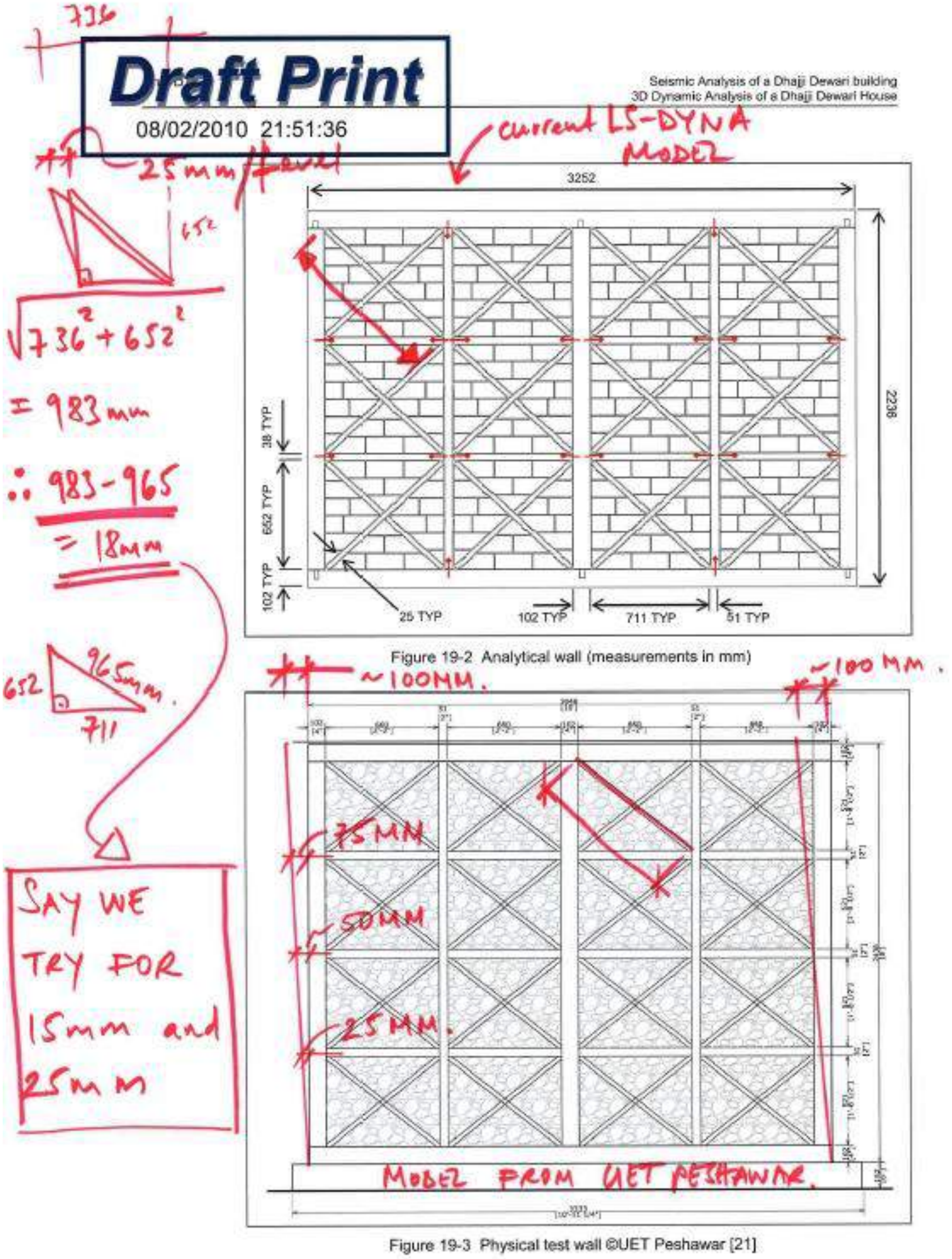


Figure D9-1 Hand calculations for brace shortening levels