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NONLINEAR ANALYSIS OF TRADITIONAL STONE MASONRY IN MUD MORTAR HOUSES IN NEPAL BEFORE AND AFTER RETROFITTING

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Abstract

Many houses in rural Nepal are constructed using stone masonry in mud mortar walls, a combination which has proven to be particularly susceptible to damage from earthquakes. Their seismic vulnerability is due both to their particular structural configuration (flexible floor diaphragms, lack of effective connections between structural elements and walls with very large mass) and to the mechanical properties of their masonry walls (nonlinear behaviour with low tensile strength). In this paper a nonlinear analysis method is used to evaluate the effectiveness of a proposed retrofit design, which includes the addition of reinforced concrete elements to traditional stone masonry in mud mortar houses.

The first step was to calibrate a constitutive nonlinear material model based on Drucker-Prager plasticity and Willam-Warnke failure criteria by replicating laboratory experiments on stone masonry in mud mortar samples using finite element analysis [1, 2]. This calibration process [3] allowed for the determination of an accurate constitutive nonlinear material model to simulate traditional stone masonry in mud mortar walls in ANSYS. Using these results, a three-dimensional model of a traditional house was set up in ANSYS and subjected to numerous pushover analyses while the boundary conditions and connections within the model were calibrated. By adjusting the supports and connections, the resulting damage patterns could be shown to correlate well with the crack patterns and failure mechanisms observed in traditional buildings following the 2015 Gorkha earthquake. This involved out-of-plane failure mechanisms occurring first in the end walls, as they are not typically connected back to the floor diaphragms, and was followed much later by in-plane failure of the side walls. This close correlation between numerical model results and field observations showed that the adopted nonlinear analysis method approximated the observed damage well.

The traditional house model was then updated to include the strengthening works that form part of the proposed retrofit design. This involved tying the structure together from the inside, with new reinforced concrete elements built up against the existing walls. Reinforced concrete column “strong backs” were used to connect the masonry walls to the floor diaphragms which were strengthened with reinforced concrete slab strips and ring beams to better withstand out-of-plane loading. The strength of the in-plane walls was also improved by applying a concrete plaster to both faces of the walls and installing a reinforced concrete block through the wall. The reinforced concrete elements were modelled using linear elements while the nonlinear material model for the stone masonry in mud mortar walls was adjusted to account for the through concrete elements and concrete plaster.

A pushover analysis of the retrofitted finite element model showed that the retrofit design was very effective at controlling out-of-plane failures and at efficiently transferring lateral forces to the in-plane walls, developing a global failure mode. The retrofit ensured that the building responded with box-like behaviour and sudden brittle out-of-plane flexural failure was replaced by controlled in-plane shear failure as the governing design condition, significantly improving the building’s structural capacity under seismic forces.

Keywords: Stone masonry; Drucker-Prager; Willam-Warnke; retrofit; pushover analysis; earthquake engineering.

1. Introduction

The 2015 Gorkha earthquake in Nepal highlighted the seismic vulnerability of uncoursed random rubble stone masonry in mud mortar (SMM) structures that are very common in rural areas of the country. Unfortunately, very little research on the material properties of these structures has been carried out to date, making an evaluation of their seismic capacity very difficult. Following the devastating earthquake in 2015 Build Change began work in Kathmandu and over the last few years they have completed a number of laboratory experiments to better understand traditional SMM structures, including uniaxial compression tests, combined axial and shear tests, and diagonal compression tests. The results of these tests allowed for important material properties such as the Young's modulus, compressive capacity, tensile capacity and shear capacity to be calculated. Finite element analysis (FEA) models were then set up (in ANSYS) for each experiment, with the aim of reproducing the results in a numerical model. By adjusting the material properties in the FEA, a constitutive nonlinear material model was calibrated and the numerical output was found to show a close correlation with the experimental results.

The purpose of this paper is to develop a non-linear FEA approach for evaluating the seismic capacity of traditional SMM buildings in Nepal and accurately simulating the type of failures that have been observed in traditional buildings in Nepal following the 2015 earthquake. This analysis will improve our understanding of these buildings' structural capacity under earthquake loading and indicate where the first failures are most likely to occur in the buildings, allowing for the design of targeted seismic retrofitting works. Build Change has also developed a retrofit design system that can be used on SMM buildings with various dimensions, so a separate model was created for this retrofit design and analysed to determine what impact this strengthening work would have on the seismic capacity of the building.

2. Traditional Buildings in Nepal

Most buildings in rural areas of Nepal are constructed using load bearing walls constructed from uncoursed random rubble stone masonry in mud mortar with an approximate thickness of 450mm. The number of storeys varies from one to three, with the most typical arrangement being a two-storey building as shown in Fig. 1. The attic floor is supported by an internal timber frame with a row of central timber columns supporting a primary longitudinal beam, with transverse joists spanning out to the side walls. The layout, size and materials used in the construction of traditional houses are very similar across the country, with only small variations in opening layout or overall dimensions observed in different districts. Thus, the models used in this paper could be considered representative of a wide range of houses throughout Nepal.

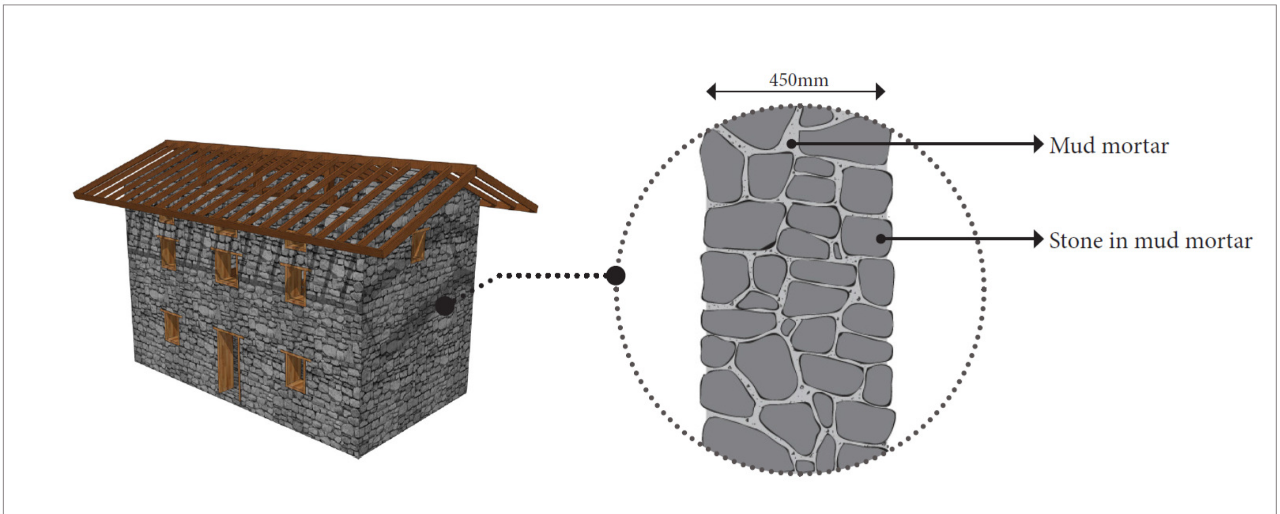


Fig. 1 – A typical traditional Nepalese house (left) and a section through a SMM wall (right).

2.1 Structural Layout

Traditional houses in Nepal are typically two-and-a-half storey buildings (two full storeys plus a reduced attic area) and have very similar plan dimensions. The masonry walls are constructed from stone masonry in mud mortar and they extend approximately one metre above the top floor level to form the attic space and support the roof framing above. A line of timber posts running through the centre of the house, provide support to the floor at each level and the roof above. The floors are built from timber joists spanning between the masonry walls and a central timber beam, which support timber planking and a mud slab. The roof consists of wooden beams, rafters, and purlins with either slate/stone tiles or corrugated galvanised iron roofing on top. Fig. 2 presents the layout and dimensions of the model used in the analysis for this paper.

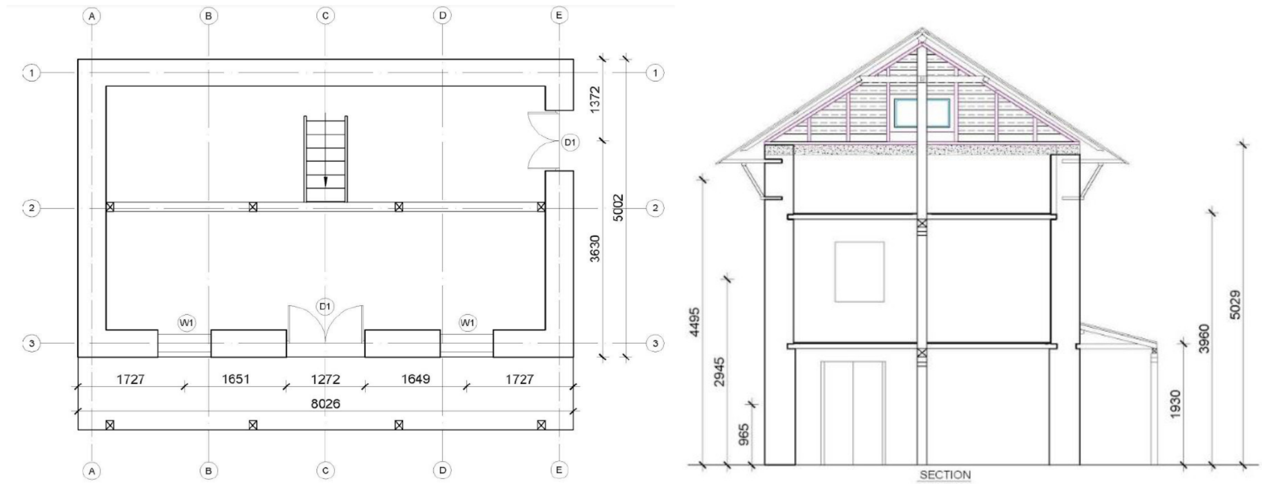


Fig. 2 – Typical ground floor layout (left) and section (right) through a SMM house.

2.2 Retrofit Design

The retrofit design involves tying the structure together from the inside, with new reinforced cement concrete (RCC) elements built up against the existing walls. RCC column "strongbacks" and RCC slab strips are connected to the SMM walls with reinforced through concrete elements, to better withstand out-of-plane loading. The strength of the in-plane walls is also improved by applying a cement plaster to both faces of the walls and installing through concrete elements at regular intervals to reduce out-of-plane delamination failures. The masonry gable walls at the attic level are replaced with lightweight timber walls and the timber roof is securely fixed back to the main structure. These retrofit modifications are presented in Fig. 3.

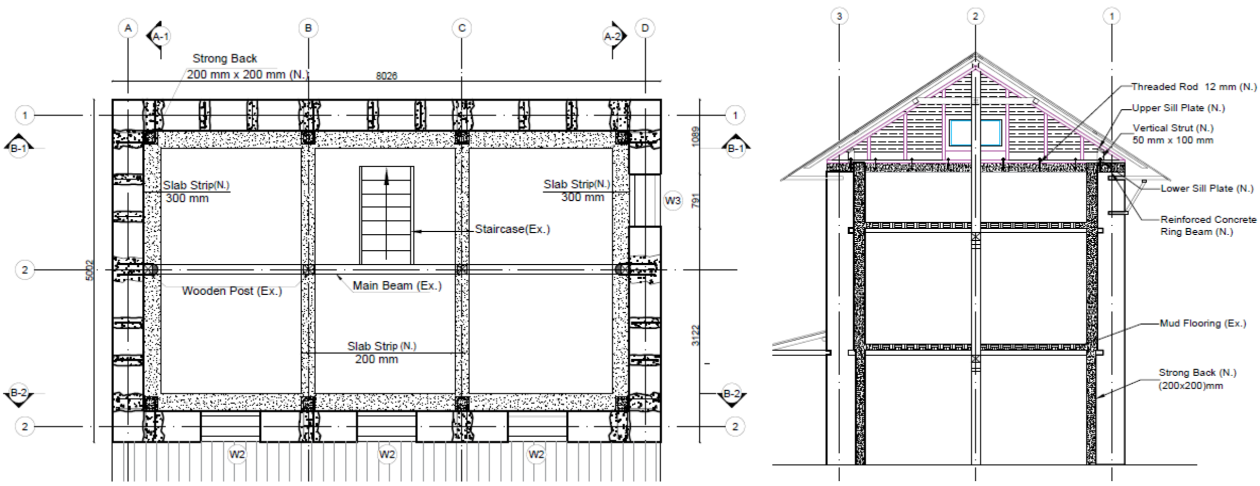


Fig. 1 – First floor plan (left) and section (right) showing the new RCC elements built inside the existing SMM building.

3 Numerical Modelling Overview

Due to the large size of the models and the random nature of the SMM walls construction, a macro-modelling approach was adopted for the numerical analysis in this paper, with the stones, mortar joints and interfaces globally represented by single continuous elements. ANSYS was chosen to carry out this macro-finite element modelling, as it allows for nonlinear behaviour to be simulated through the use of different material models. [4] In this case a combination of Drucker-Prager (DP) plasticity and Willam-Warnke (WW) failure criteria have been used to reproduce the constitutive behaviour of the masonry, assuming an elastoplastic law with tension cut-off. This approach has been used previously [5, 6], where the researchers combined DP and WW criteria to evaluate the seismic vulnerability of a masonry church, confirming that this method can be used to simulate entire buildings.

3.1 Constitutive Nonlinear Material Model

The material model used in this paper was calibrated based on the results of experiments performed by Build Change in Kathmandu and the interested reader can refer to a previous paper [3] that describes in detail the steps involved in this calibration process. In summary, a nonlinear static analysis was carried out on three-dimensional finite element models representing the laboratory test specimens with the aim of replicating the plastic deformation and failure mechanisms observed in those experiments. This resulted in a calibrated nonlinear material model which could then be used to simulate the behaviour of other structures built from SMM. Other material properties were also determined during this process, including modulus of elasticity values, Poisson ratios and densities [3]. Table 1 presents the final values for the calibrated nonlinear material models, while Table 2 summarises some other pertinent parameters used in the FEA models.

Table 1 – Parameters used in the constitutive nonlinear material models.

Phase	Parameters	Symbol	units	Initial Values	Calibrated SMM	Calibrated Retrofit
Elastic	Young's Modulus	E	MPa	60	60	60
	Poisson's Ratio	v	-	0.25	0.44	0.44
Plastic	Cohesion	c	MPa	0.1	0.2	0.2
	Internal Friciton	φ	°	35	55	55
	Dilatancy Angle	δ	°	15	25	25
Failure	Compressive Strength	f_{cWW}	MPa	2.4	4	4
	Tensile Strength	f_{tWW}	MPa	0.02	0.02	0.06
	Shear Transfer (closed)	β_c	-	0.75	0.75	0.75
	Shear Transfer (open)	β_t	-	0.15	0.15	0.15

Table 2 – Other parameters used in the FEA models.

Material	Density	Young's Modulus	Poisson's Ratio
	(kg/m^3)	(MPa)	(-)
Reinforced Concrete	2500	22,360	0.2
Timber	525	9,820	0.3
Masonry	2200	60	0.44

3.2 ANSYS Setup

The masonry walls in the three-dimensional finite element models were discretised with isoparametric three-dimensional eight node elements (SOLID65). The wooden floors, timber roof and RCC elements were modelled using linear one-dimensional isoparametric elements (BEAM188). Floor finishes (such as mud toppings) were not included in the models but their weights were applied to the beams and walls supporting them by means of zero-dimensional distributed masses (MASS21).

The timber-to-timber and timber-to-wall connections were typically modelled with member releases, to ensure that no moment was transferred and that a flexible floor diaphragm was simulated. The central timber beam was modelled as a continuous element, as it is usually formed from one piece of timber. As this central timber beam is typically built into the end walls, which are susceptible to out-of-plane failure, the assumptions made for these connections are particularly important. It was decided to model the embedded part of the timber beam as a 3D solid element fully bonded to the surrounding masonry wall and connected to the timber frame using a spring. The same spring connection detail was used for the through concrete elements which were also embedded into the masonry walls and connected to the RCC columns and beams in the retrofitted model. The stiffness of the springs was calculated based on push out tests conducted by Build Change of the through concrete elements embedded in masonry walls, which gave an equivalent spring stiffness of 770 N/mm.

The choice of boundary conditions was also very important, particularly relating to the timber posts and RCC columns. As the timber columns are not fixed to the foundation, and merely have a shear key detail to prevent lateral movement, the vertical support condition needs to accurately reflect this. Thus a nonlinear spring support was adopted in the vertical direction to simulate both the high compression capacity of the support and the very low tension capacity in uplift cases. The RCC column bases were assigned a higher tension capacity than the timber columns as the RCC columns are cast monolithically with a small reinforced concrete pad foundation founded below ground level.

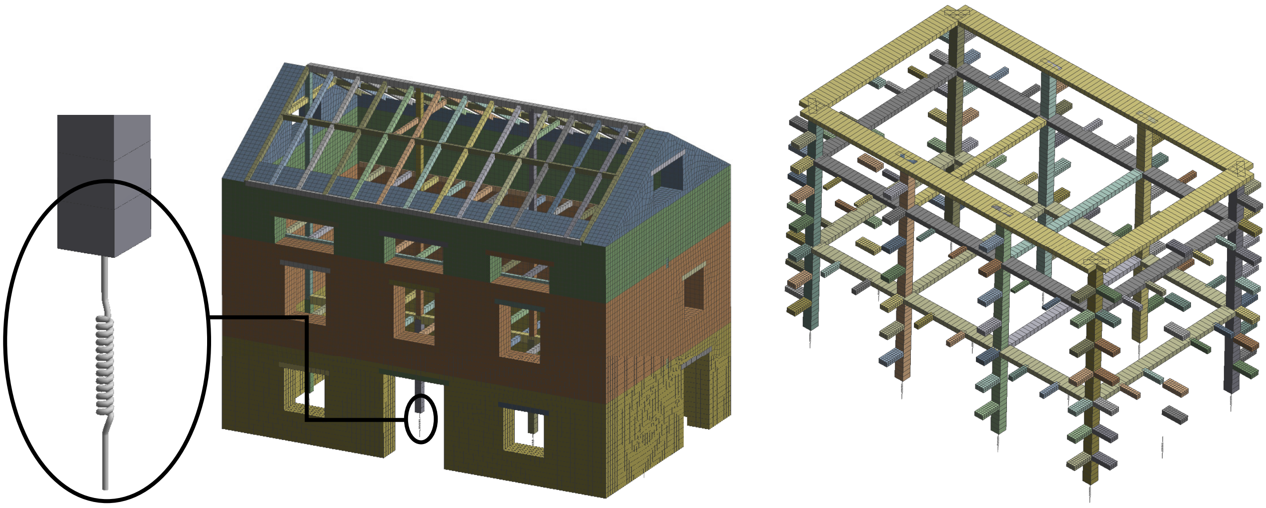


Fig. 4 – Nonlinear spring supports (left) were used to model the base of timber columns in the model of the existing traditional house (middle) and the model of the RCC frame and through concrete elements which was used to retrofit the traditional house (right).

4 Pushover Analyses

The three-dimensional finite element models of the traditional building and the retrofitted building were then subjected to nonlinear static pushover analyses. The models were subjected first to constant gravity loads and subsequently, to linearly increasing horizontal accelerations to simulate an “effective earthquake force”. Using acceleration to apply earthquake loads to a structure is much more accurate than applying forces to various elements, as the acceleration is mass proportional and does not concentrate stresses in particular elements [7]. Fig. 5 shows the total deformation under horizontal acceleration of 0.12g in both the longitudinal and transverse directions.

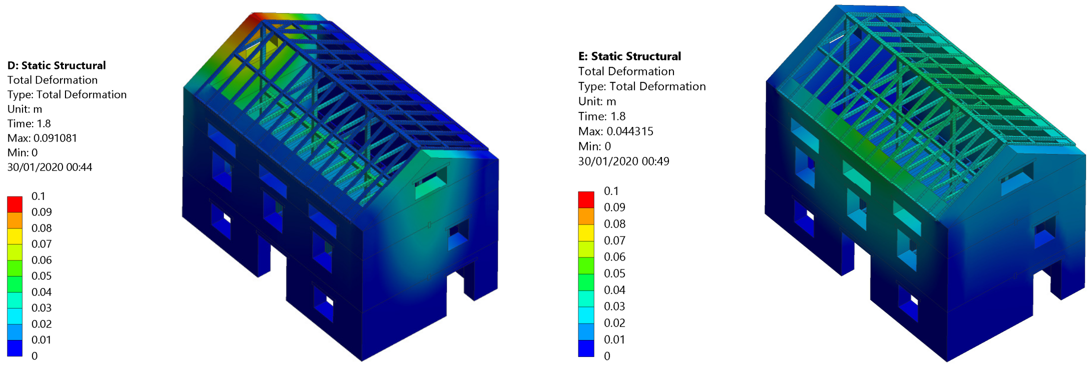


Fig. 5 – Deformation for a horizontal acceleration of 0.12g in the longitudinal (left) and transverse directions (right).

4.1 Damage Patterns in the FEA Model of the Traditional House

Graphical results from the FEA models for the pushover analyses of the existing house are shown in Fig. 6 (longitudinal direction) and Fig. 7 (transverse direction) for horizontal accelerations of 0.06g and 0.12g. The output shows the maximum principal elastic strain, which provides a useful approximation for where cracks are likely to form due to the smeared crack model.

Fig. 6 – Damage Patterns in the Traditional House Model – Longitudinal Direction

Maximum Principal Elastic Strain output from ANSYS

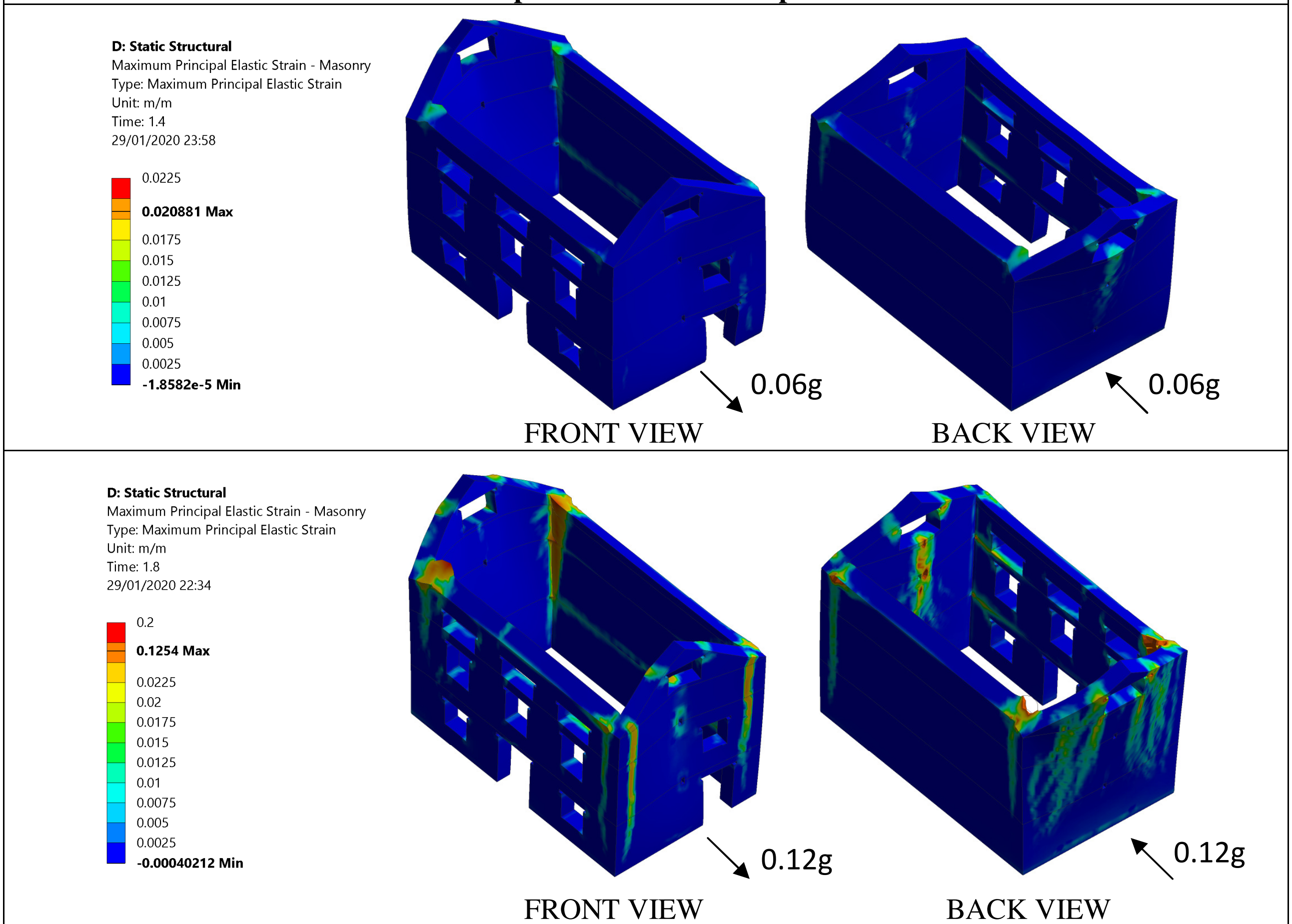
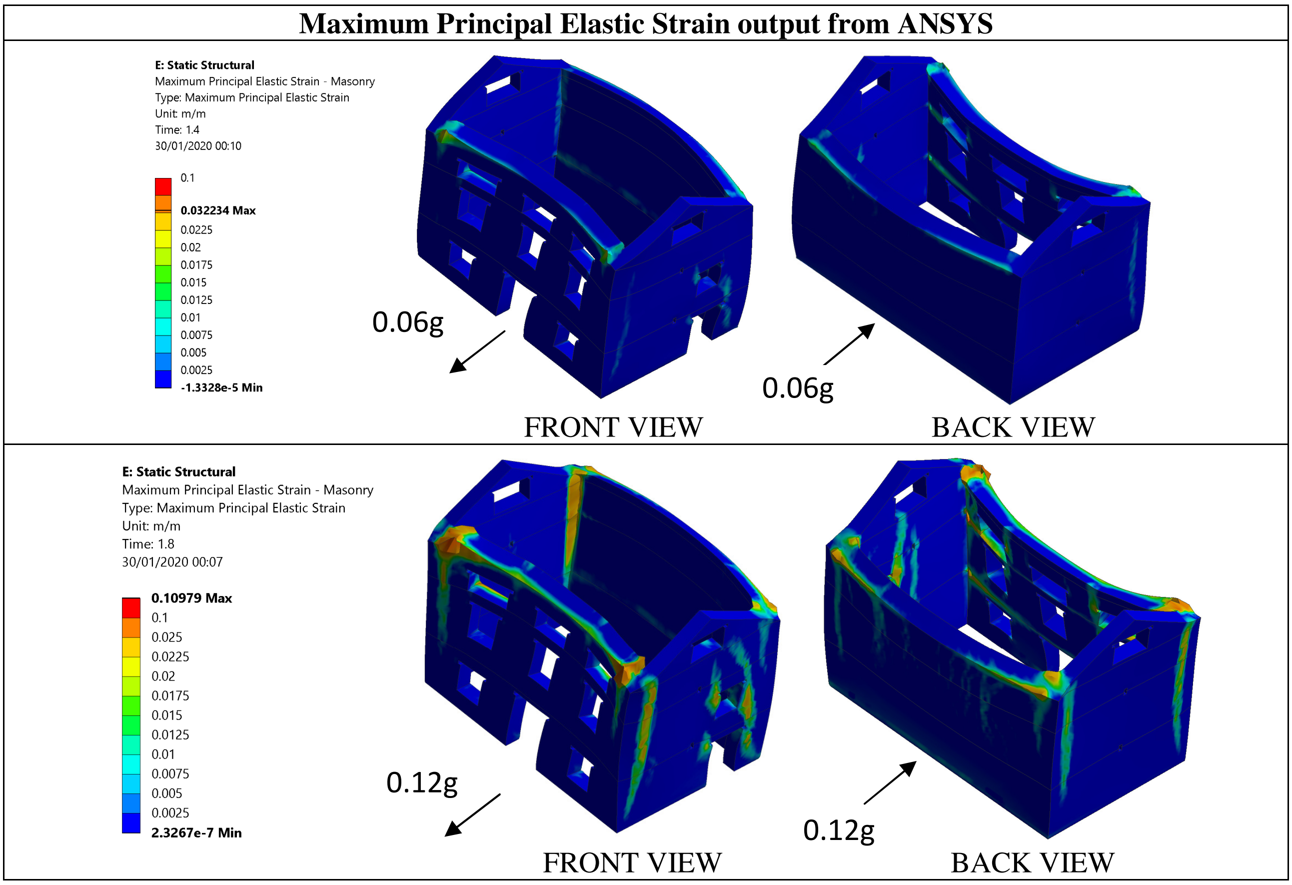


Fig. 7 – Damage Patterns in the Traditional House Model – Transverse Direction



The results presented above show that for both longitudinal and transverse horizontal accelerations, out-of-plane damage is predicted to occur first, characterised by vertical cracks forming at wall intersections and in the middle of walls experiencing bending forces (due to their orientation perpendicular to the horizontal acceleration). The FEA models show that the gable ends and end walls are particularly susceptible to this type of damage, indicating that they are likely to be the first part of the buildings to suffer damage at 0.06g, and would most likely collapse at 0.12g. At 0.12g there is also a big difference in strain predicted in the opposite faces of end walls, which could result in delamination of the outer wythe of the wall separating from the rest of the wall.

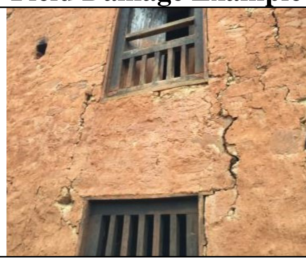
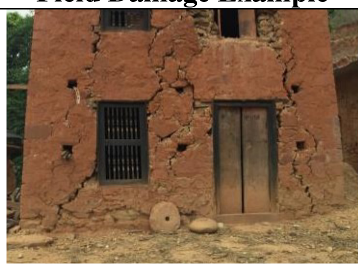
In-plane failure, characterised by diagonal cracks propagating away from openings at an angle of 45 degrees, occurs as a secondary mechanism in the side walls, a long time after significant out-of-plane damage has occurred. For a horizontal acceleration of 0.06g no in-plane damage is observed in the models, while there is significant out-of-plane damage. For a longitudinal acceleration of 0.12g small diagonal cracks are beginning to form in the side walls at the corners of openings, while for a transverse acceleration of 0.12g larger diagonal cracks are predicted in the end walls (also concentrated around openings). This is likely due to the fact that the in-plane forces will be more concentrated in the transverse direction as the building is oriented in its weak direction.

These results show that the floor diaphragms in traditional SMM buildings are very flexible and are not transferring horizontal forces efficiently between out-of-plane walls and in-plane walls. This is particularly evident for the end walls, which experience more out-of-plane damage than the side walls even though they are shorter and thus should experience lower bending forces. This is most likely due to the fact that the floor joists span between the central timber beam and the side walls, and do not restrain the end walls.

4.2 Examples of Field Damage from 2015 Gorkha Earthquake

The field damage examples in Table 3 begin with the most commonly observed damage patterns, and this order of damage mechanisms also provides an approximate order in which the damage would occur in an earthquake.

Table 3 – Observed Damage Mechanisms in the Field

1: Gable Wall Collapse	**Field Damage Example**
Much of the damage that was observed in traditional houses following the 2015 earthquake was concentrated in the gable walls at second floor/attic level. These walls are typically a continuation of the stone masonry walls up to the roof ridge level and they are usually not well connected to the timber roof frame or floors. Furthermore, as they are located at the top of the buildings they are subject to the highest acceleration forces during an earthquake, which means that they are often the first structural elements to fail.	
2: End Wall Separation	**Field Damage Example**
The next most common failure mechanism observed in the field was the formation of vertical cracks at wall intersections, ultimately leading to the collapse of the end walls away from the main building. The authors believe that the stresses from the seismic forces are concentrated in the wall corners and the cracks that form create a “pinned” condition which makes the end walls more flexible. Furthermore, the floor joists span the shorter axis of the building which leaves the end walls unrestrained, so seismic forces result in out-of-plane failure mechanisms, with vertical cracks beginning to form in the middle of these walls.	
3: Side Wall Out-of-Plane Cracking	**Field Damage Example**
Damage to the side walls is often the next observed failure mechanism with vertical cracks forming in the middle of the wall between openings, where out-of-plane bending forces are at a maximum. These cracks are often observed at the top of the building, in the unbraced parapets. This is because the timber roof imparts additional bending stresses into these cantilevered parapets. These cracks can be observed in the FEA model output for a horizontal acceleration of 0.12g in the transverse direction.	
4: Wall Delamination	**Field Damage Example**
Often the walls are constructed of inner and outer wythes, with the middle portion filled with rubble and mud. Sometimes through stones are installed which span from the inner face to the outer face, connecting the two layers, however this is not common as stones of this size are harder to find, shape and install. Thus, with very few bonding elements between the interior and exterior wythes, wall delamination occurs when the wall starts to crack and bend which can lead to subsequent collapse.	
5: Diagonal Cracking	**Field Damage Example**
This failure mechanism results in diagonal cracks forming in the walls, especially around openings, and propagating away at an angle of 45 degrees. This type of failure is associated with in-plane failure and is not observed very often in traditional houses as they are much more likely to experience out-of-plane failure and collapse before in-plane failure occurs. This is because these structures have flexible floor diaphragms which are not able to transfer lateral forces effectively to the in-plane walls.	

When the finite element results in Section 4.1 are compared with the crack patterns and failure mechanisms observed in traditional houses in Nepal following the 2015 earthquake, a strong correlation can be identified. The types of damage which were most commonly observed in the field following the earthquake were also the damage mechanisms that occurred first and were the most significant in the finite element models.

4.3 Pushover Curves for Existing SMM Buildings

Reviewing the damage pattern output from the finite element models of the traditional house and the field damage examples it is clear that the model is accurately reflecting the behaviour of traditional houses. This output however, does not provide the seismic capacity of the entire building as a system. For this a pushover analysis generating a capacity curve provides more insight into how the building is responding to the seismic forces. Fig. 8 shows the capacity curves from the ANSYS model of a traditional building with drift % plotted against acceleration in the transverse and longitudinal directions. A similar study, also using laboratory testing results from Build Change, but adopting a simplified micro-modelling approach, found a similarly low lateral capacity of 0.08-0.12g at 0.4% drift for traditional SMM buildings in Nepal. [7] It is interesting to note that the ANSYS output shows that the existing building is weaker in the longitudinal direction, even though this is the strong axis of the building, due to the failure of the unrestrained end walls. The capacity curves from that study are also included in Fig. 8 for comparison.

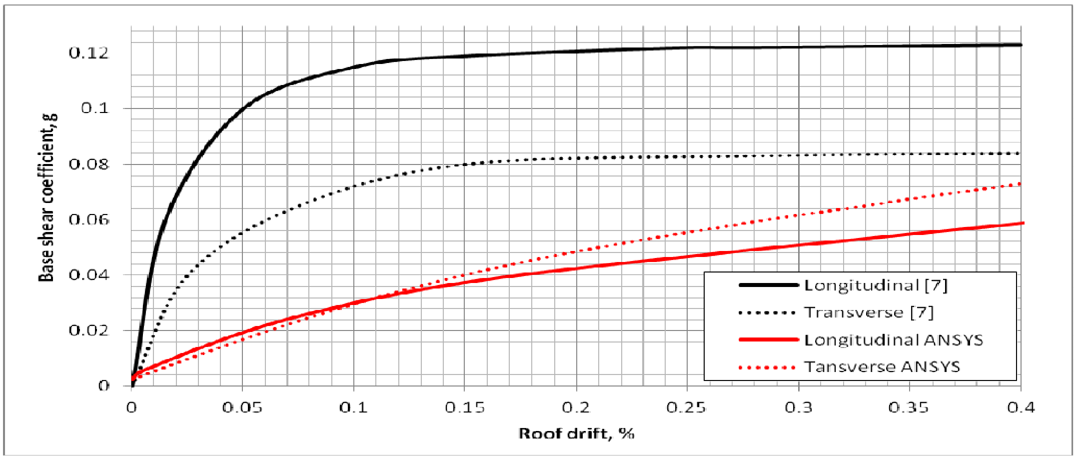


Fig. 8 – Nonlinear Pushover Curves for Existing SMM buildings.

4.5 Damage Patterns in the Retrofitted Building FEA Model

The finite element model of the traditional house was then “retrofitted” which involved adding various RCC elements to tie the building together more effectively, as described in Section 2.1. The RCC members were added as beam elements and were connected back to the existing building at “through concrete” locations. The retrofitted model also used the updated constitutive material model for the SMM walls to reflect the increased tension capacity from the addition of the cement render and through concrete elements.

The FEA of the retrofitted model showed a significant reduction in damage at low lateral loads, compared to the traditional SMM building model and the model showed a different order to the damage patterns as well. Whereas the damage to the traditional building model had been governed by out-of-plane damage, with in-plane damage occurring as a secondary failure mechanism, the retrofitted building model exhibited the opposite order of predicted damage. This shows that the retrofit design is acting as a box and that the stiffened floor diaphragms are allowing for the development of a global failure mode, rather than collapse of individual parts of the structure. The damage patterns predicted in the retrofitted building model at different horizontal accelerations are presented in Fig. 9 (longitudinal direction) and Fig 10. (transverse direction). Higher horizontal accelerations were used in this analysis to highlight the increased seismic capacity of the retrofitted models.

Fig. 9 – Damage Patterns in the Retrofitted House Model – Longitudinal Direction

Maximum Principal Elastic Strain output from ANSYS

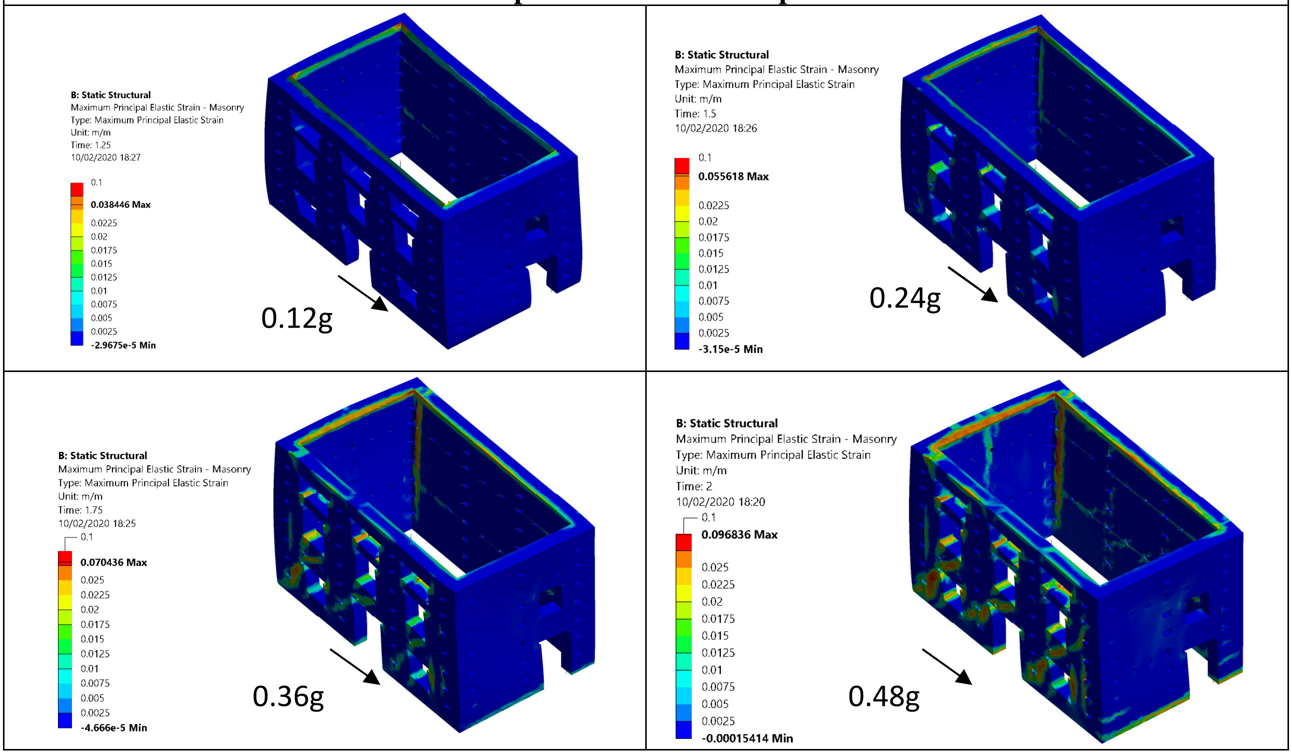
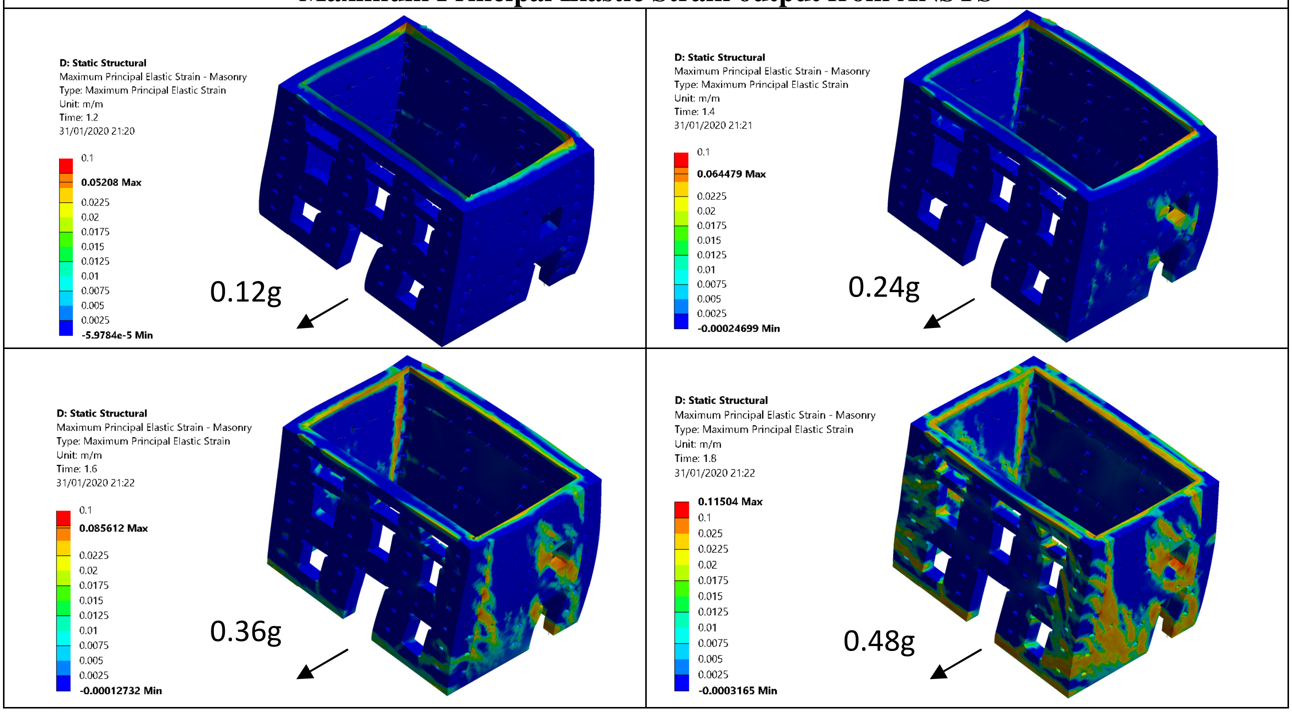


Fig. 10 – Damage Patterns in the Retrofitted House Model – Transverse Direction

Maximum Principal Elastic Strain output from ANSYS



4.6 Pushover Curves for the Retrofitted House

Comparing the damage pattern output from the finite element models of the traditional SMM building and the retrofitted building, it is clear that the latter experiences far less damage when subjected to the same accelerations. The increase in seismic capacity of the retrofitted building compared to the SMM building becomes apparent when reviewing their pushover curves, as shown in Fig. 11. The pushover curves from a similar study [7] have also been included for reference, although it should be noted that these curves relate to a single storey building which has horizontal RCC bands at regular intervals in the walls, so it is not a direct comparison.

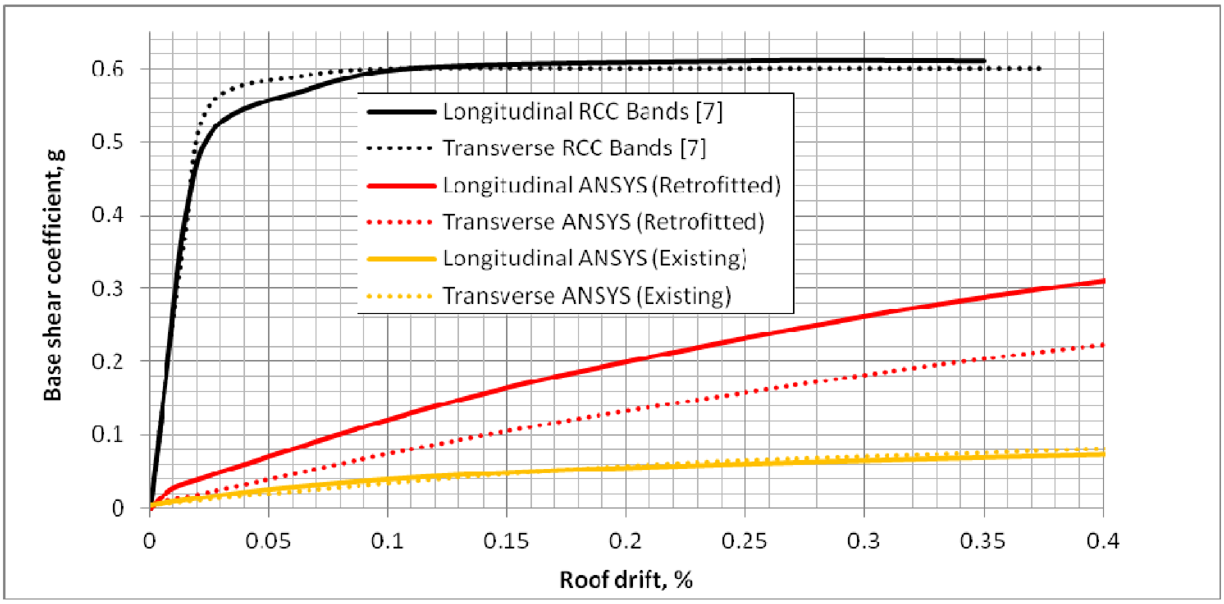


Fig. 11 – Nonlinear Pushover Curves for Existing and Retrofitted SMM buildings.

As is clear from Fig. 11, the slopes of the pushover curves for existing and retrofitted buildings are significantly different with the traditional building model exhibiting significant damage even for a very low horizontal acceleration of 0.06g. The slope of the linear part of the curves is also noticeably different with the retrofitted house model exhibiting a stiffer response to lateral forces. The retrofitted building model also experiences a gradual change from linear to nonlinear behaviour suggesting that failure mechanisms are controlled by the retrofit. This implies that it is unlikely that sudden brittle failure would occur, thus reducing the chances of collapse at low earthquake loads. These pushover curves clearly show that the retrofit design is operating as it has been designed to do, controlling sudden brittle out-of-plane failure mechanisms and increasing the ability of the building to withstand earthquake induced lateral loads.

The capacity curves for the retrofitted building model in both longitudinal and transverse directions have both increased, although it is worth noting that the longitudinal direction achieves a capacity of 0.3g while the transverse direction only achieves approximately 0.2g. This reflects the orientation of the building, with the longitudinal direction including longer in-plane walls. The RCC frame and the strengthening measures to the SMM walls have prevented the walls from undergoing out-of-plane failure and channelled the seismic forces into in-plane forces in the walls. Thus, the primary collapse mechanism in the retrofitted model is in-plane shear failure, while in the existing building models in-plane shear failure is a rarely occurring secondary failure mechanism. This means that the walls are being used much more efficiently as the forces are in their strong direction.

The pushover curves from the comparison study [7] are significantly higher although it should be noted that these related to a single story house built with reinfored concrete bands at regular centres through the walls, which significantly increases their in-plane capacity. Thus, it is not surprising that that study shows a much higher seismic capacity.

5. Conclusions and Future Work

From the results and discussions presented in this paper, the following main conclusions can be drawn.

- As proven by the results of validation studies, as well as through the comparison of actual and numerical failure modes presented in this study, FEA using a constitutive non-linear material model for SMM construction is an effective tool to model and study the seismic behaviour of traditional buildings in Nepal. The damage patterns predicted by the models analysed in this paper correspond well with the crack patterns and failure mechanisms which had been observed in traditional houses in Nepal following the 2015 earthquake.
- Collapse of the gable walls is often the first failure mechanism observed in traditional SMM buildings, followed by vertical separation cracks and corner failure triggering the collapse of end walls. The next observed damage mechanism is longitudinal wall out-of-plane cracking and in some cases delamination of outer wythes. In-plane failure of walls in existing SMM buildings is much rarer, and is only ever observed as a final failure mechanism.
- The proposed Build Change retrofit design has been shown to be very effective at preventing the sudden out-of-plane failure mechanisms most commonly observed in existing SMM buildings, by tying the whole structure together and transferring lateral earthquake forces into the in-plane walls, developing a global failure mode. This is achieved through strengthening the flexible timber floor diaphragms at each floor level and by providing adequate wall-diaphragm connections. The concrete render and through concrete elements also increase the in-plane strength of the walls.
- These improvements ensured that the building responded with box-like behaviour and sudden brittle out-of-plane flexural failure was replaced by controlled in-plane shear failure as the governing design condition, significantly improving the structural behaviour of the traditional house under seismic forces.
- Possible future work related to this paper could involve conducting a seismic performance assessment and derivation of fragility/vulnerability functions as demonstrated by [7].

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