SEISMIC VULNERABILITY OF MONASTERY TEMPLES OF STONE MASONRY

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ABSTRACT
Buddhist monasteries share a rich history in the culture and tradition of the Sikkimese Himalayan region. These monasteries, some dating back to 300 years, have played a significant role in portraying and conserving the architectural style of the Tibetan and Chinese construction. A typical monastery consists of a spacious courtyard, a main temple and living quarters for the monks. The temples are simple one to three-tiered structure on rectangular plan with reduced floor area for upper stories. The exterior walls are in stone masonry, mostly random rubble (R/R), enclosing an inner multi-level timber frame structure of column and beam system supporting the wooden floors. The double pitched pagoda-style timber roofs are usually covered with corrugated metal sheets.

The extensive damages caused to some of these monasteries in earlier earthquakes reveal their seismic vulnerability. A large number of these old monasteries also suffered varying degree of damages in the recent M6.9 Sikkim earthquake of September 18, 2011. Post-earthquake ambient vibration measurements of main temple were made at three monasteries, which establish them as short period structures with the fundamental period ranging from 0.23 to 0.37 s. A finite element analysis of one of the temples was carried out to study its dynamic behaviour and predict its seismic vulnerability. Response spectrum analysis and static lateral load analysis were performed which identified the wall openings as critical areas with tensile stresses exceeding the permissible value, which was supported by the observed damage.

KEYWORDS: monastery, seismic vulnerability, stone masonry, ambient vibration

INTRODUCTION
The Himalayan state of Sikkim is dotted with monasteries, some dating back to the 17th century and serve as centres of meditation and institution for learning Buddhist philosophy. These monastic structures showcase intricate timber carvings, life-like frescos of hoary Buddhist legends and illustrative paintings which are of great historical significance and act as a symbol of the rich cultural heritage.A typical monastery consists of a spacious courtyard, a main temple or shrine hall and dwellings or schools for the monks as shown in Figure 1. The shrine halls are simple one to three-tiered structure having symmetrical plan and reduced floor area on the upper stories. The temple is constructed traditionally using stone masonry walls on the exterior and

timber beam-column frame on the inside to support the timber floor diaphragm. The hipped timber roofs covered with corrugated metal sheets are provided at one or more levels. The overhangs extend up to 2 to 3 m protecting the exterior walls from the rain.

Sikkim lies in the seismic zone IV of IS 1893 [1] with an expected shaking intensity of VIII (MSK scale). The M6.9 earthquake hit Sikkim on September 18, 2011 at 6:11 PM IST with its epicentre located near Nepal-Sikkim border. The event caused widespread destruction and affected these historical structures causing varying degree of damage mostly to their exterior walls [2]. To understand the dynamic behaviour of the temple, ambient vibration measurements were made at three monasteries and subsequently, finite element analysis was performed for dynamic and static lateral loads to predict the expected seismic demand and its vulnerability.

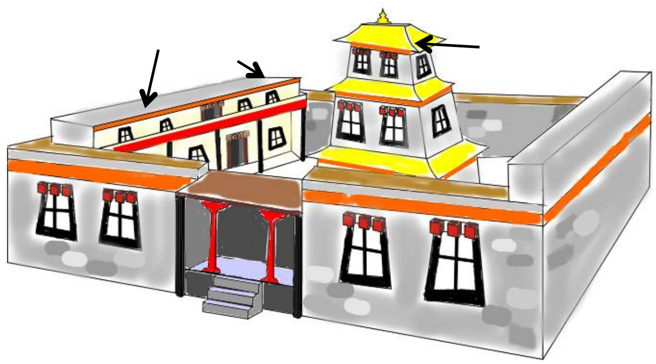
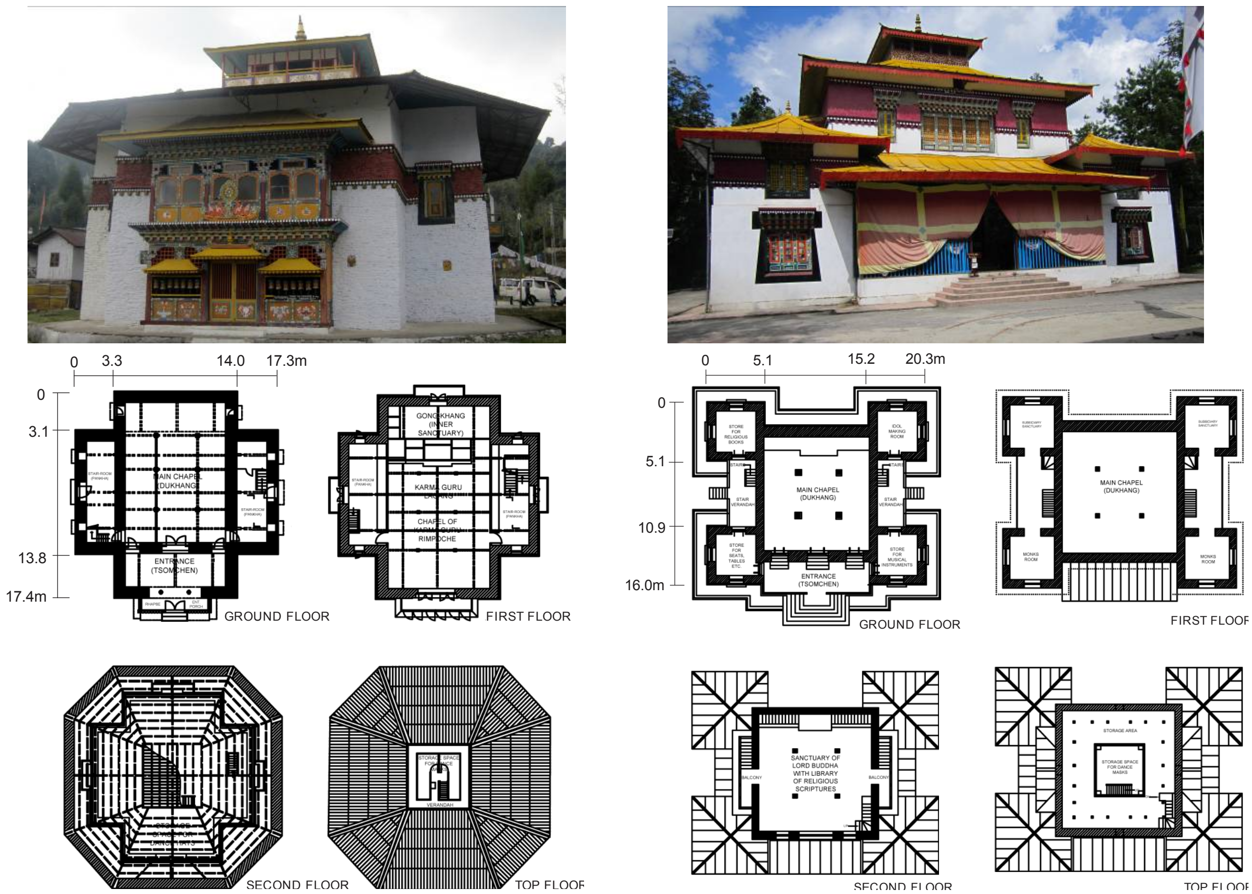


Figure 1: (a) Layout of a typical monastery showing the courtyard, main temple and school

STRUCTURAL SYSTEM OF THE MAIN TEMPLE

The main temple is a two or three storey structure with load bearing exterior walls and an inner timber frame which supports the wooden diaphragm. Figure 2shows an exterior view and floor plans at different floor levelsof two important monastic temples in Sikkim. Both temples have symmetric plan with rectangular opening at the centre which serve as an assembly hall. The exterior walls are thick, tapered and built using dressed or semi-dressed or random rubble (R/R) stone masonry laid in mud or lime or cement mortar. The thickness of the wall varies from 0.5 to 1 m which gradually reduces in the upper stories. Doors and windows are an important part of the Buddhist construction from both functional and religious point of view. These openings are large in size and number, and significantly influence the overall strength of the wall.

The timber framing system comprises of grid of columns and beams arranged in the central portion of the room. The main beam runs from one wall to the other in the direction parallel to the main entrance or *Tsomchhen*. The columns are not continuous from bottom to top in a multi-level frame but they are constructed in such a manner that their centre-line is maintained at all the floor levels. These columns are tapered and made from solid timber logs which extend from the base of the floor to the capital (Figure 3a). The capitals known as bows have elaborate carvings and are placed in either two or three layers depending upon the height of the floor. The floor is made of wooden planks placed on wooden rafters supported between the main beam and the wall [3]. Hipped timber roofs covered with corrugated galvanised iron sheet are provided at one or multiple levels resting on trussed rafters as shown in Figure3b.



(a) (b)

Figure 2: A view of the main temple and floor plans at different floor levelat (a) Labarang Monastery and (b) Enchey Monastery [4]

(a) (b)

Figure 3: Details of (a) wooden timber frame and floor diaphragm supported on rafters and (b) hipped roof construction.

In the newly constructed monasteries the traditional timber frames are replaced by RC frames. In addition, appendages to the existing structure such as, pavilions, extended prayer halls, corridors etc., are constructed in reinforced concrete and brick/block masonry to accommodate the expanding congregation and also occasionally to support the weight of the aging structure in several monasteries.

SEISMIC PERFORMANCE

The monasteries have poor lateral load resistance capacity as the stone masonry walls have low in-plane and out-of-plane strength. In addition the timber floors under lateral loads act as a flexible diaphragm and undergo excessive deflection, pushing the walls outwards. Moreover, such heavy walls attract a large amount of inertial forces and are easily overwhelmed by such forces and displacement demands imposed on them.

In the September 18, 2011 event several monasteries suffered varying degree of damages ranging from cracked walls to total collapse. Heavy damages were observed to exterior walls at several monasteries, e.g., total collapse of a village temple at Lachung, partial collapse at Ringhem Choling monastery at Mangan (Figure 4a), delamination of walls and cracks in Samten Choling monastery temple, Lachung (Figure 4b).

(a) (b)

Figure 4: Common damages observed (a) Partial collapse of wall of Ringhem Choling monastery and (b) Shear cracks and delaminated wall painting at Samten Choling monastery.

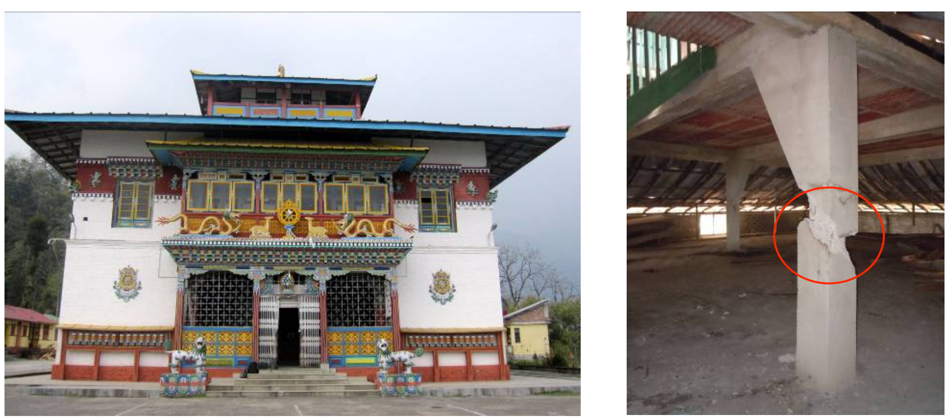
Enchey monastery at Gangtok, retrofitted after the 2006 Sikkim Earthquake also suffered moderate damages in the 2011 event [5]. The out-of-plane failure around the opening was observed in the unretrofitted portion of third-storey wall as shown in Figure5. Similar damage to the exterior wall built in stone laminates was also observed at Labrang monastery (Figure6). After the 2006 earthquake, the collapsed roof was replaced by a truss roofing system supported on steel columns erected around the shrine room. Steel joists connected to columns were also inserted below the timber floor to relieve the load on the timber beams and walls.

Phodong monastery which is a mixed construction of RC frames and load bearing masonry walls suffered no damages to the exterior walls (Figure 7a). However, the RC columns were severely affected because of short-column effect due to the presence of deep haunches at the beam-column junctions as shown in Figure 7b. The haunches were supposedly provided to increase the lateral load resisting capacity of the frame, which in turn created a short-column effect causing the brittle shear failure of columns.

Figure 5: Damages observed on the front wall of third storey at Enchey monastery

(a) (b)

Figure 6: (a) Cracks on the exterior wall of Labrang monastery and (b) Exposed stone laminates during the recent damage and steel columns and joists used in the retrofit



(a) (b)

Figure 7: (a) Phodong monastery, interior RC frame with exterior stone masonry wall and (b) Damaged column due to short-column effect induced by the presence of deep haunches

Tsug-lakhang also referred to as the royal chapel, a two storey building with a regular plan area, constructed in dressed stone masonry, is a hallmark of excellent workmanship and building technology of the Buddhist architecture. No significant damage was observed to its structural components during the Sept. 18, 2011 earthquake. Table 1 summarizes the various types of structural components in the seismic load path of these structures and the observed damage pattern.

Table 1: Seismic Load Path and Observed Damage Patterns

Structural components			Damage pattern	Affected monasteries
Vertical load resisting system	*Walls*	Random rubble masonry	• Out-of-plane bulging and collapse • Vertical and shear (sliding) cracks • Local damage and failure to wall corners due to excessive thrust from timber joist • Shear cracks at the corner of door and window openings	Ringem choling, Samten choling, Rikzing choling, Enchey, Pubyuk and Labarang monastery
		Concrete block masonry	Diagonal and shear cracks due to poor quality of blocks	Rumtek monastery
	Frames	Timber beams and posts	No damage	All old monastery constructions
		RC frames	Shear and flexural failure in columns due to inadequate/poor seismic detailing	Phodong monastery
Horizontal load resisting system	*Diaphragms floor*	Timber floor	No damage	All old monastery constructions
		RC floor	No damage	All new monastery constructions
	Pitched roof	Timber joist or frame and CG sheets	No damage	All old and new monastery constructions
		Steel frames and CG sheets	No damages	Monasteries retrofitted after 2006 earthquake; Labarang monastery

POST-EARTHQUAKE AMBIENT VIBRATION TEST

Ambient vibration measurements of main temple were made at three monasteries namely Enchey, Labrang and Phodong to obtain the dynamic properties of these structures. The vibration measurements were performed in two directions, i.e., parallel to the main entrance (x-direction) and perpendicular to the main entrance (y-direction), using SS-1 Ranger seismometer (Kinemetrics, USA). The set up of sensor and data acquisition system at Enchey monastery and Labrang monastery are shown in Figure8.

The measurement was taken at every floor and at different locations, chosen depending on the ease of placing the sensor. The sampling frequency was 2000 Hz which was recorded for about 100 s each time. The recorded time-history in both the direction and its corresponding Fourier spectrum at three temple sites is shown in Figure9. The data are filtered and re-sampled to remove the high frequency content. It was observed that all the three temples were short period structures and fall in the acceleration sensitive region of the seismic design response spectrum. In addition the floor vibrations were also measured by placing the seismometer in upright position and creating vibration by tapping the floor.

Figure 8: Experimental setup consisting of seismometer and data acquisition system at Enchey monastery and Labrang monastery

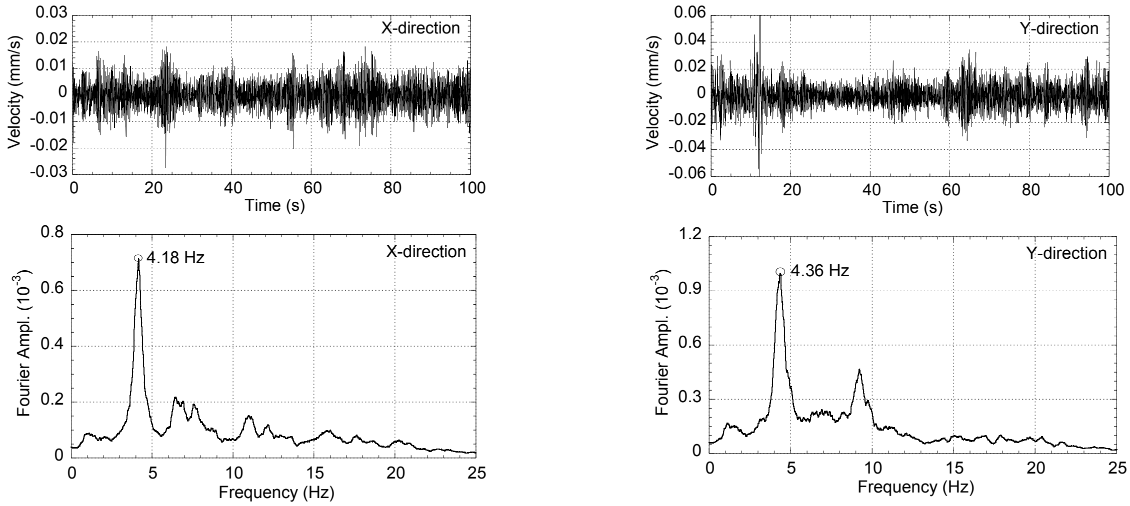
SEISMIC VULNERABILITY ASSESMENT

The seismic vulnerability assessments of monastic temples were performed using the simplified method of analysis as described by Lourenco and Roque [6].Due to the symmetric and regular plan configuration of these structures, a simplified method will be a good indicator for their possible seismic performance. The following simplified safety indexes are analyzed [6]:

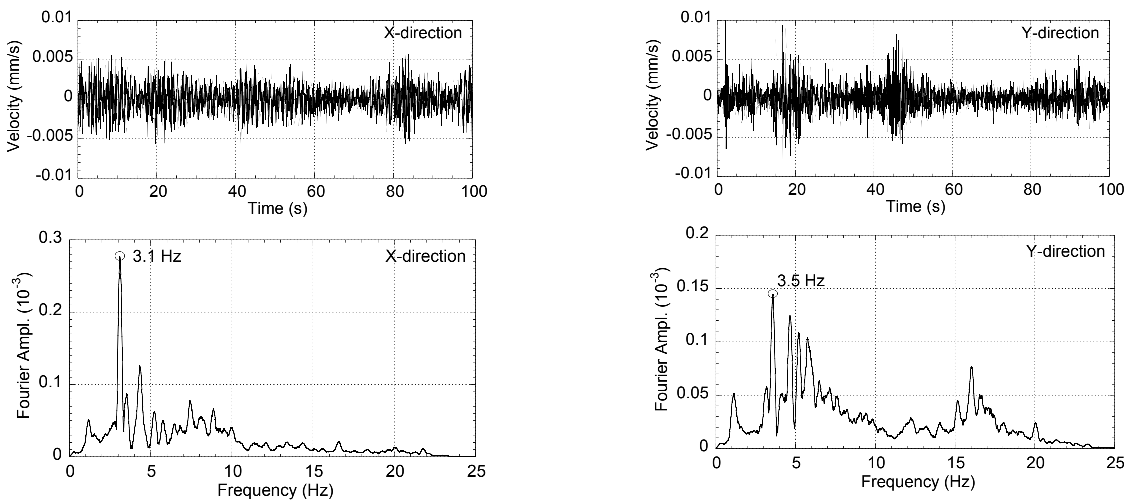
- Index 1: In-plan area ratio, $\lambda_{1,i} = A_{wi} / A_p$
- Index 2: Area to weight ratio, $\lambda_{2,i} = A_{wi} / W$
- Index 3: Base shear ratio, $\lambda_{3,i} = V_{Ci} / V_D$

where A_{wi} is the area of earthquake resistant walls in direction 'i', A_p is sum of the floor plan areas for all floors in a building and Wrepresents total seismic weight. V_D is the total shear demand for seismic loading and can be estimated as per IS 1893($V_D = \alpha_h \times W$), where α_h is an equivalent seismic coefficient related to the design ground acceleration [1]. V_{Ci} is the shear capacity of structure in direction 'i' and can be obtained from shear strength of wall $V_{Ci} = \Sigma A_{wi} \times f_v$, where, according to Eurocode 6 [7], $f_v = f_{v0} + 0.4\sigma_d$. Here, f_{v0} is the cohesion, which can be assumed equal to a low value or zero in the absence of more information, σ_d is the design value of the normal stress and 0.4 represent the tangent of a constant friction angle [6]. Substituting the values of V_D and V_{Ci}: $\lambda_{3,i} = A_{wi} / A_w \times 0.4/\alpha_h$, where, A_w is the total plan area of earthquake resistant walls.

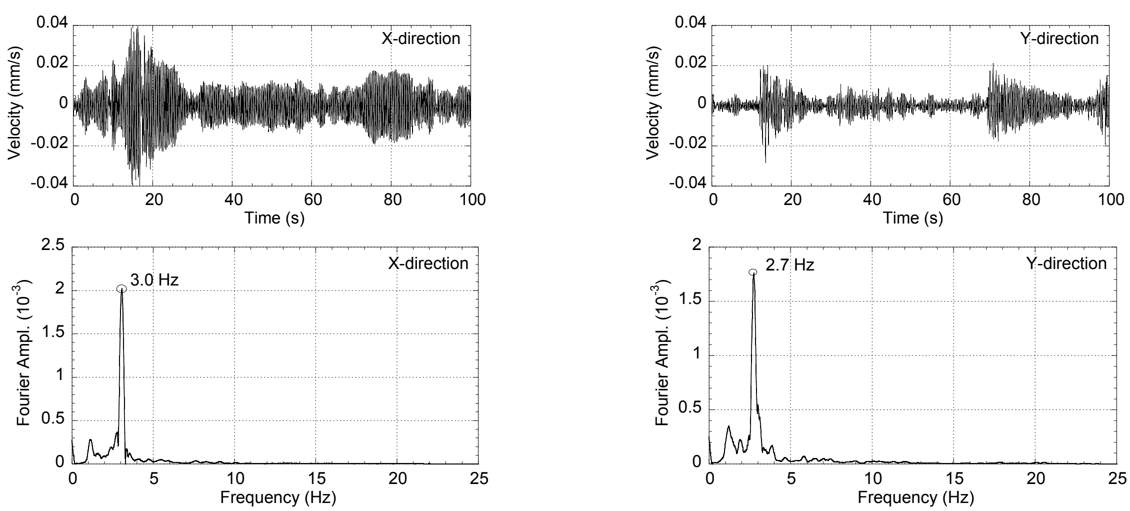
In high seismicity region, minimum value of 10% and 1.2 m^2/MN is recommended for indexes$\lambda_{1,i}$and $\lambda_{2,i}$ for historical masonry buildings, respectively.The third index $\lambda_{3,i}$ consider the seismicity of the zone and a minimum value equal to one is acceptable. Table 2 gives the values of three indexes for Enchey and Labarang monastery temples (Figure 2). The seismic coefficient $\alpha_h$was equal to 0.3 for historical structures in Zone 4 according to Indian seismic code [1]. Except for index 2 and value of $\lambda_{1,y}$ for Labarang monastery, both temples violates the limiting values for Index 1 and 3. The minor to moderate damage observed in both monastery temples during the current 2011 Sikkim earthquake comply with the results of simplified indexes. These indexes reveal that monastery temples are highly vulnerable to damage during the seismic event and require remedial measures or, at least, detailed investigation. The results of index 2 conflicts with the observation of other two indexes and indicate that its threshold values needs revision.



Enchey monastery



Labarang monastery



Phodong monastery

Figure 9: Recorded time-history and Fourier spectrums at three monasteries in *x*-direction parallel and *y*-direction perpendicular to the main entrance

Table 2: Value of three indexes for Enchey and Labarang monastery

Monastery	Index 1 (%)		Index 2 (m^2/MN)		Index 3	
	$\lambda_{1,x}$	$\lambda_{1,y}$	$\lambda_{2,x}$	$\lambda_{2,y}$	$\lambda_{3,x}$	$\lambda_{3,y}$
Enchey	8.3	7.8	3.6	3.4	0.72	0.68
Labarang	7.7	10.2	4.2	5.5	0.67	0.88

To further understand the behaviour of the monastery temples and also to predict their seismic vulnerability, the main temple at Enchey monastery was studied using Finite Element (FE) analysis in Abaqus environment [8]. Modal analysis and response spectrum analysis (RSA) was carried out to know the dynamic properties of the structure and determine the seismic demands in terms of stresses in the various structural components. In addition, static horizontal loads proportional to the weight of structure were applied in the two orthogonal directions which showed higher stresses in the areas around the opening where the damage was observed

Enchey monastery is one of the most important monasteries of the Nyingma sect and was built by Sikyong Tulku in 1909 [4]. The main temple is a four-tiered construction with semi-dressed stone masonry walls on the outside and roof at three levels as shown in Figure 2b. The plan is symmetric with a rectangular opening at the centre and four square sections connected at the four edges as shown in Figure 2b. The frame comprises of four columns arranged in a rectangular grid with main beams running parallel to the entrance and a transverse secondary beams in the perpendicular direction between the two columns.

A three dimensional FE model was developed for Enchey monastery using the general purpose program Abaqus [8]. At this stage, no material characterization could be possible due to insufficient data on the strength of the masonry walls. The properties of the stone masonry, corrugated GI sheets and timber used for analysis are mentioned in Table 3. The typical value of Young's modulus for stone masonry rangingfrom 200 to 1000 MPa has been reported by Tomaževič [9]. A modulus of 300 MPa near lower values of the range was chosen considering the age, quality and type of masonry construction.The masonry walls and timber frame were modelled as 3D solid elements while the floor and roof components were modelled as 3D shell elements. The second and third storey floors were modelled as isotropic shell of thickness 145 and 210 mm to match the frequencies of two floors (9.4 Hz and 13.5 Hz) as obtained by the ambient vibration test in the vertical direction. The discretization of masonry walls and timber frame was achieved by 4-noded linear tetrahedron elements (C3D4) whereas 3-noded triangular shell elements (S3) of the Abaqus element library were used for roof and timber floors. Monolithic connection was assumed between the column and beam joints and between frame and diaphragm. A view of the FE model is shown in Figure10.

Table 3: Mechanical properties of materials used in the FE modeling

Material	Density (kN/m^3)	Young's Modulus (GPa)	Poisson's Ratio
R/R stone masonry	20.0	0.30	0.20
GI sheet	78.5	210	0.30

Timber frame	8.0	7	0.12
Timber floor	8.0	12	0.12

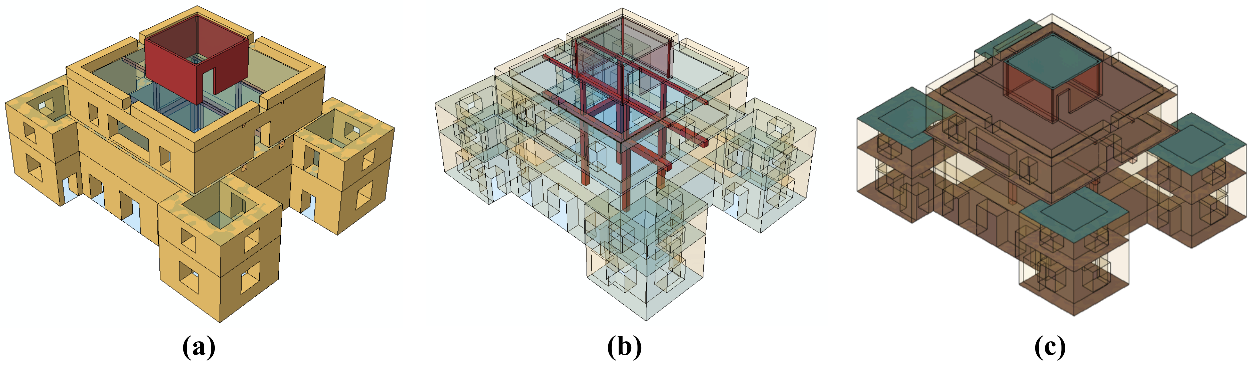


Figure 10: View of FE models (a) Exterior masonry walls and small wooden cabin (b) Frame system of the structure comprising of wooden beams and columns (c) Roof and floor diaphragms modeled using solid shell elements.

The modal analysis of the structure was performed by assuming fixed translational and rotational boundary conditions at the base of the structure. The frequency in the orthogonal direction: parallel (*x*-direction) and perpendicular (*y*-direction) to the entrance was found to be 4.5 Hz and 4.8 Hz, respectively as shown in Figure 11. These obtained frequencies are in a good agreement with the observed values of 4.2 and 4.4 Hz measured by ambient vibration tests.

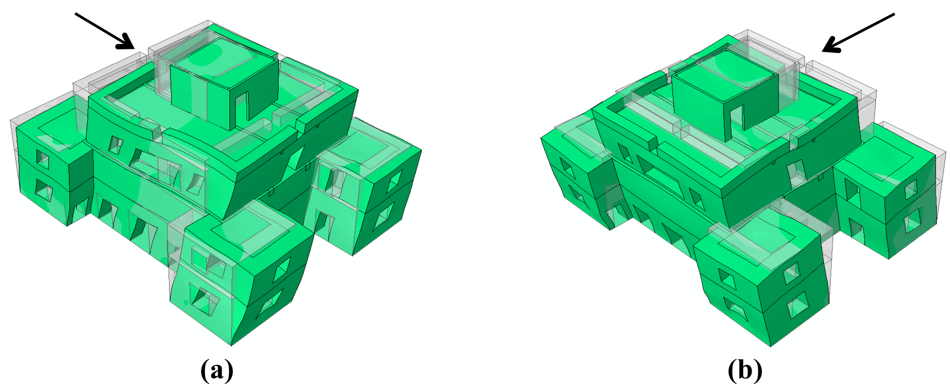
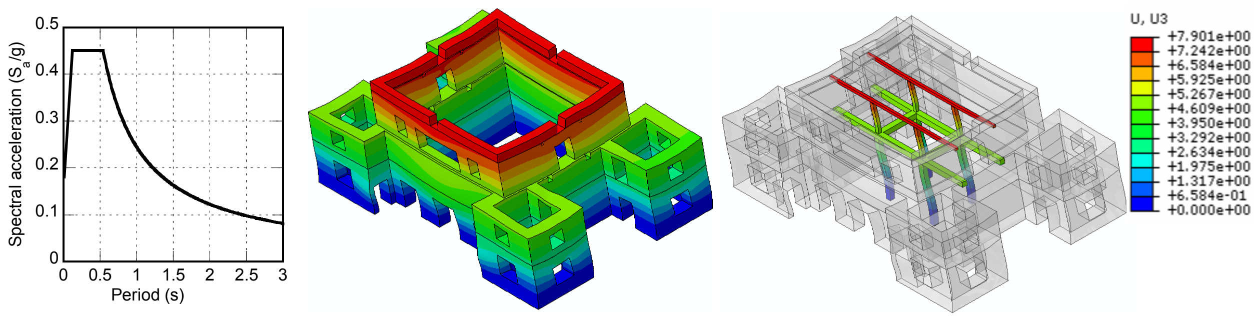


Figure 11: Mode shapes for natural frequency in (a) *x*-direction and (b) *y*-direction

Response spectrum analysis (RSA) considering all significant modes in both the direction was performed to predict the seismic demand for Zone IV (zone factor of 0.24g). For the design basis earthquake (DBE), 5% damped elastic design response spectrum for soil type II was scaled so that the zero period acceleration (ZPA) is 0.18g which includes a load factor of 1.5 (Figure 12a). The total base shear was calculated which was found to be 29% and 31% of the self weight in *x*- and *y*- direction, respectively. The obtained base shear was divided by the net wall area in both directions at the window level to estimate the average wall shear stress. Figure 12b shows the displacement contour for loading in *y*-direction as obtained from the response spectrum analysis.

In order to identify the stress critical zone in the structure, a uniform static horizontal load taken as 20% of the weight as per the Indian seismic code was applied in both the directions. Table 4 summarizes the stresses obtained by response spectrum analysis and lateral load analysis. The maximum tensile stresses observed at the corner of openings exceeded the lower bound crackingvalue of 0.08 MPa [9] (Figure13). The location of maximum stresses was same as the location of damage observed during the 2011 earthquake (Figure 5).



(a) **(b)**

Figure 12: (a) Response spectrum for zone IV soil type II with 5% damping,(b) displacement contour for loading in *y*-direction as obtained by RSA for masonry walls and timber frame

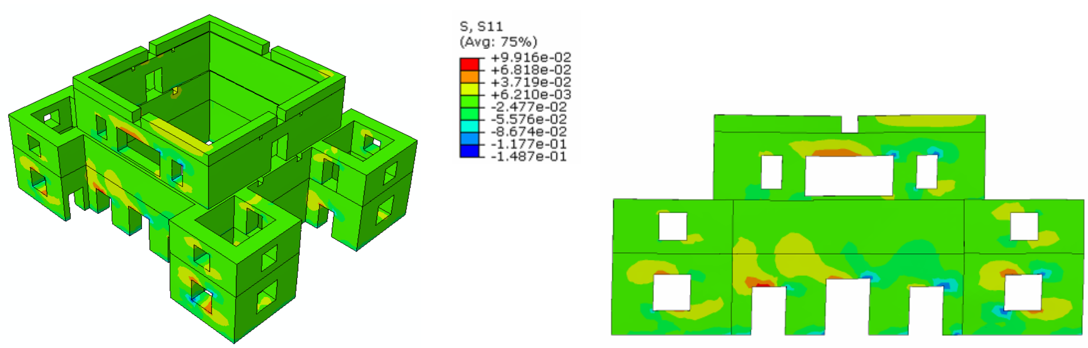


Figure13: Areas identified with critical stresses (*x*-direction)

Table 4: Results of the response spectrum and lateral load analyses for stresses in walls

Excitation direction	Shear stress from RSA (MPa)	Lateral Load Analysis			
		Tensile Stress (MPa)		Compressive Stress (MPa)	
		x-dir.	*y*-dir.	*x*-dir.	*y*-dir.
x-dir.	0.08	0.10	0.07	0.14	0.12
y-dir.	0.09	0.07	0.10	0.32	0.12

The FE analysis highlights the vulnerable portion of stone masonry walls at upper stories and especially around openings. The interior wooden frame of building is relatively robust which also in correlation with no damage observed during the earthquake. The short period and hence their behavior is dominated by acceleration response. In order to reduce/mitigate damage in future earthquake it is suggested to reinforce/strengthen the masonry around openings and control deformation of timber diaphragm which may induce out-of-plane deformation demands on perimeter stone walls. Moreover, as indicated by the simplified indices such as, in-plan area ratio and base shear ratio, the in-plane strength of masonry walls in these structures need to be enhanced to prevent any major in future seismic events.

CONCLUSIONS

The damages caused to the monasteries during the 2011 Sikkim earthquake highlighted the seismic vulnerabilities of these structures of cultural heritage. Major damages were observed in the exterior stone masonry walls due to their poor lateral load resisting capacity. The timber floor diaphragms and roof structures behaved satisfactorily with negligible damage. The ambient vibration tests performed on main temples at three monasteries showed that they were short period structures with the fundamental period ranging from 0.23 to 0.37 s. The FE model of the temple was able to simulate the observed frequencies and the dynamic behaviour, which was dominated by massive and relatively stiff masonry walls. The model was used to estimate seismic demands imposed on various components of the structure for a design level earthquake. The lateral load analyses showed that the tensile stresses around the wall openings exceeded the permissible values and are, therefore, susceptible to damage. The simplified safety indices were analyzed for the quick assessment of vulnerabilities under earthquake loads and stone masonry construction of monastery structures were found seismically deficient as per these indices. Further, thepredicted vulnerabilitiesfrom simplified safety indices and more refined FE analysescompared well with the damages observed in the recent earthquake, however, these preliminary results needs to be further supported with detailed analyses.

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