Seismic vulnerability of monastery temples of stone masonry in Sikkim Himalaya

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Buddhist monasteries in the Sikkim region have conserved and portrayed the art of Tibetan and Chinese architectural style through centuries. These historic structures have sustained varied degrees of damage due to earlier earthquakes. Their performance in the recent Sikkim earthquake of *M* 6.9 on 18 September 2011 shows their high seismic vulnerability. A quick seismic assessment using certain simplified indices suggests higher vulnerability of damage for these heritage structures. A post-earthquake ambient vibration test established these monastery temples as short-period structures with fundamental period of 0.23 to 0.37 s. A finite element analysis of one of these temples has been done to study its dynamic behaviour. The response spectrum and static nonlinear pushover analysis highlighted vulnerable portions of stone masonry walls and provided useful insights for proper retrofitting to mitigate damage in future earthquakes.

Keywords: Ambient vibration, finite element analysis, monasteries, seismic vulnerability, stone masonry.

THE state of Sikkim has a long history of diversified influence from China, Tibet, Bhutan, Nepal and India, and this is reflected in its local culture and architecture. The monasteries portray this tradition and culture of Sikkim with their majestic art and architecture. These monasteries, some of which date back to as early as the 17th century AD, are present all over Sikkim and serve as Buddhist meditation and learning centres.

The monasteries in Sikkim are of three types: (a) Takphu, a rock-cave or a rock hermitage; (b) Gompa, a place for learning the finer aspects of Tibetan Buddhist culture, often built in inaccessible areas, and (c) Mani Lakhangs, which are administered by few monks and do not have any schools. A typical monastery consists of a central temple surrounded by schools and dwellings for the monks, with a spacious courtyard (Figure 1). These central temple or shrine halls are one to three storey, pagoda-style structures enclosing the chapel for daily prayer and a library of holy scriptures on the upper storeys; their inner walls are marked with paintings and frescos about Buddhist legends. The floor area of the temple is generally square and symmetrical in plan and gets reduced in the upper levels. They are provided with pitched roof at the top and also at the lower levels, with the overhangs extending up to 2–3 m. The schools and dwellings around the courtyard in the monastery complex are single-storey buildings in traditional Ikra style or reinforced concrete (RC) frames with masonry infills. These monasteries are ancient and built with locally available material with primitive knowledge of earthquake-resistant structures. Most of these monasteries are built with rubble stone masonry, mud mortar and timber. These structures have seen many earthquakes and have performed poorly, sustaining varied degrees of damage.

The *M* 6.9 earthquake hit Sikkim on 18 September 2011 at 6 : 11 p.m. IST with its epicentre located near Nepal–Sikkim border. The event caused widespread destruction and affected these historical structures, damaging mostly the bulky exterior walls[1]. This led to irreparable damage to the frescos, paintings and other artefacts of historical and religious significance (Figure 2). Therefore, the retrofitting scheme has to be carefully designed which requires careful evaluation of seismic vulnerability of these heritage structures. To understand the dynamic behaviour of the temple structure, ambient vibration measurements were made at three monasteries. Moreover, finite element (FE) analyses were performed on a structure under dynamic and static lateral load to predict the expected seismic demand and its vulnerability.

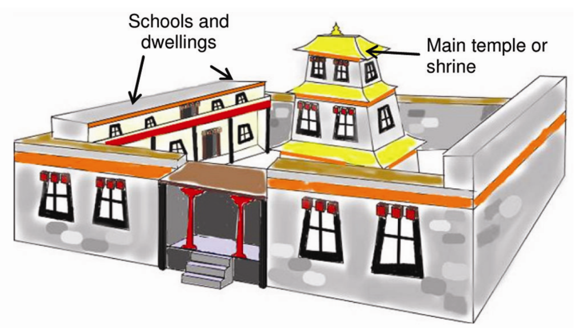

Figure 1. Layout of a typical monastery showing the courtyard, main temple and school.

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Figure 2. Damage caused to the frescos during the 18 September 2011 earthquake.

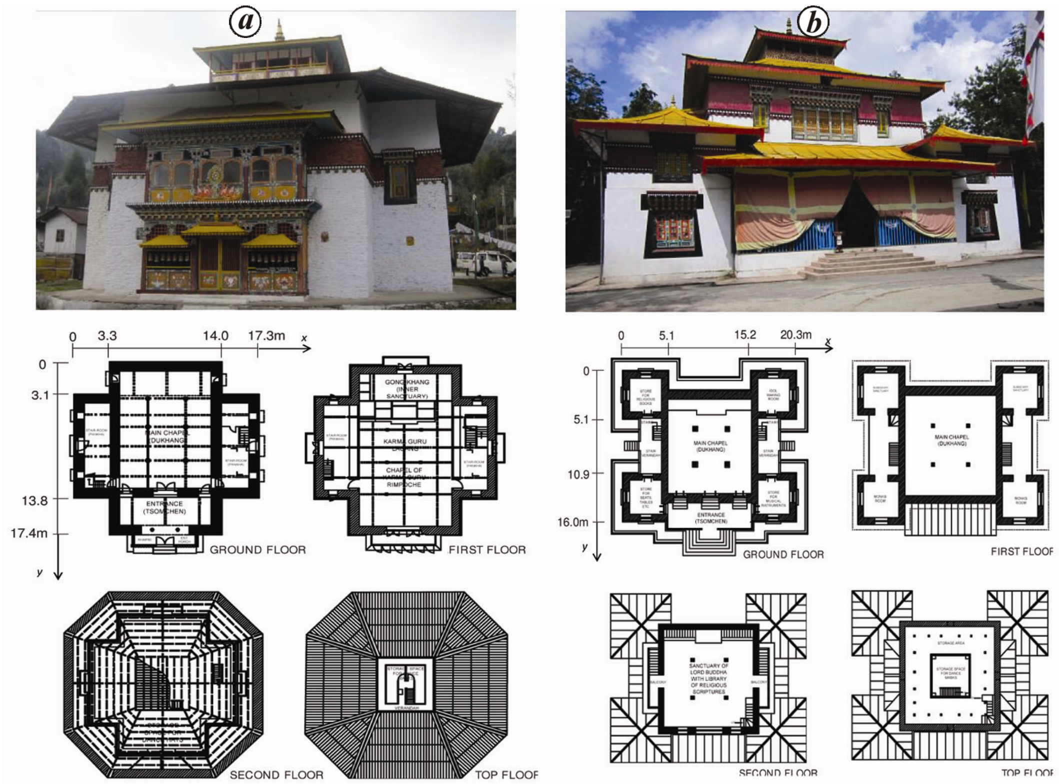

Figure 3. A view of the main temple and floor plans at different levels: ***a***, Labrang monastery; ***b***, Enchey monastery[3].

Structural system of the main temple

The construction practices of these monastic temples have evolved over centuries, and the old, traditional construction materials have been replaced by new ones. The older monasteries for example, Labrang (1826), Dubdi (1701) and Enchey (1909), have been constructed using exterior random rubble stone masonry walls and an interior timber frame along with timber floor diaphragms. In comparatively newer monasteries, or those that have been retrofitted, the timber frame is replaced by RC frame, such as in the Phodong monastery. The monasteries built around 1920s, like Tsuklakhang in Gangtok, the exterior walls have been constructed using dressed stone masonry. In the more recently built monasteries, the random rubble or dressed stone masonry wall has been replaced by concrete block masonry, for example, in Rumtek monastery built in 1960s in Gangtok. The rubble masonry exterior walls in the older monasteries are about 0.5–1 m thick at the bottom, tapered on the outside surface and the mortar used in these walls is mud, lime or cement mortar. The windows and doors are present on the three sides of the temple, excluding the side opposite the main entrance where the deities and scriptures are placed.

These openings are large in number and size, which greatly reduces the lateral strength of the wall.

Figure 3 shows an exterior view and floor plans at different levels of Labrang and Enchey monasteries in Sikkim. The timber frame is present in the central portion and has poor connection with the exterior walls. The beams run parallel to the main entrance Tsomchhen, supporting wooden rafters over which the floor made up of wooden planks rests[2]. The beam–column joints have 2–3 layers of capitals demonstrating intricate carvings (Figure 4 *a*). The columns are tapered and discontinued at each storey level and are constructed such that their centre line is maintained throughout all the storeys. The timber frame, diaphragms, joists, rafts, windows and doors have been built with hard wood like chestnut and walnut[3]. Figure 4 *b* shows the hipped timber roofs covered with corrugated galvanized iron sheets resting on trussed rafters.

In many monasteries certain appendages such as pavilions and extended prayer halls are built to accommodate the expanding congregation, for example, in Phodong monastery. Also, the unprofessional and unsystematic retrofitting on various occasions introduces RC, block/brick masonry or RC components in several monasteries. Most of the times, these interventions prove ineffective (Figure 5 *a*), and sometimes may even be detrimental to the structure as well as the artefacts (Figure 5 *b*)[4].

Seismic performance during the 2011 Sikkim earthquake

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 flexible diaphragms and undergo excessive deflection, pushing the walls outwards. Such heavy walls also attract a large amount of inertial forces and are easily overwhelmed by forces and displacement demands imposed on them. During the 18 September 2011 event, several monasteries suffered varying degrees of damage 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 in Ringhem Choling at Mangan (Figure 6 *a*), delamination of walls and cracks in Samten Choling temple, Lachung (Figure 6 *b*).

Figure 4. Typical details: ***a***, Wooden timber frame and floor diaphragm supported on rafters; ***b***, hipped roof construction.

Figure 5. ***a***, Steel frame in Labrang monastery for retrofitting, which proved ineffective during the September 2011 earthquake event. ***b***, Walls damaged by supporting steel structure, Old Ngadak Gompa, Namchi, South Sikkim[4].

Figure 6. Common types of damages observed during the earthquake: ***a***, Partial collapse of wall of Ringhem Choling monastery; ***b***, Shear cracks and delaminated wall painting at Samten Choling monastery.

Figure 7. Damage observed on the front wall of the third storey at Enchey monastery.

Figure 8. Cracks on the exterior wall of Labrang monastery.

Figure 9. ***a***, Phodong monastery, interior RC frame with exterior stone masonry wall. ***b***, Damaged column due to short-column effect induced by the presence of deep haunches.

Moderate damage was seen in many monasteries, including Enchey, one of the most important monasteries, built in 1909 near Gangtok. It was retrofitted after the 2006 Sikkim earthquake, but suffered moderate damages during the 2011 event[5]. The out-of-plane failure around the opening was observed in the unretrofitted portion of the third-storey wall (Figure 7), which is similar to the damage observed to the exterior wall built in stone laminates at Labrang monastery (Figures 5 *a* and 8). The Labrang monastery was built in 1826 on a hilltop near the village of Tumlong and retrofitted after the 2006 earthquake[3]. The collapsed bamboo 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.

The Phodong monastery, built in the early 18th century and retrofitted later, is a mixed construction of RC frames and load-bearing masonry walls. These load-bearing exterior walls did not suffer damage during the recent earthquake (Figure 9 *a*). However, the RC columns were severely affected because of the short-column effect due to the presence of deep haunches at the beam-column junctions (Figure 9 *b*). 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 the columns.

In contrast to such large-scale damage in various monasteries, some of the monastic temples suffered no damage during the earthquake. The Tsuklakhang, also referred to as the royal chapel, is a hallmark of excellent workmanship and building technology of Buddhist architecture. The main temple is a two-storey structure constructed in dressed stone masonry with a regular plan area. No significant damage was observed to its structural components during the 18 September 2011 earthquake.

The floor and roof diaphragms of these monasteries did not experience any damage. Also, there was no damage to the interior timber frames in the older monasteries, but the RC frames in the new monasteries experienced severe damage, mainly due to their faulty detailing. Damage to the random rubble masonry walls was widespread, including modes like out-of-plane bulging and collapse, vertical shear cracks and corner cracks. Table 1 summarizes the various types of structural components in the seismic load path of these structures and the observed damage pattern.

Post-earthquake ambient vibration test

Ambient vibration measurements of the main temple were made at three monasteries, namely Enchey, Labrang and Phodong, to obtain the dynamic properties of these structures. The sources of excitation are natural

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	Ringhem Choling, Samten Choling, Rikzing Choling, Enchey, Pubyuk and Labrang 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
Horizanantal load resisting system	Diaphragms/ floors	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 damage	Monasteries retrofitted after the 2006 earthquake; Labrang monastery

Figure 10. Experimental set-up consisting of seismometer and data acquisition system at Enchey and Labrang monasteries.

Table 2. Frequency measurement of the ambient vibration test

Monastery	x-direction (Hz)	y-direction (Hz)
Enchey	4.1	4.4
Labrang	3.1	3.5
Phodong	3.0	2.7

vibrations caused due to wind, and various local random and periodic sources (traffic and human activity). The vibration measurements were made in two directions, i.e. parallel to the main entrance (x-direction) and perpendicular to the main entrance (y-direction) using the seismometer. Figure 10 shows the experimental set-up of sensors and data acquisition system at Enchey and Labrang monasteries.

The measurements were 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. Figure 11 shows the recorded time-histories at the top floor in both directions and their corresponding Fourier spectra at the three temple sites. The data are filtered and re-sampled to remove the high frequency content. Table 2 lists the fundamental frequencies of the temple sites observed from the ambient vibration test. It can be observed that all the three temples are short-period structures and fall in the acceleration-sensitive region of the seismic design response spectrum. In addition, the floor vibrations were measured by placing the seismometer in the upright position and creating vibration by tapping the floor. The fundamental frequencies of the floor slabs were found in the range 9–18 Hz, and used to model the floor thickness for FE analyses.

Seismic vulnerability assessment

Preliminary vulnerability assessment

A quick seismic vulnerability assessment of the structure can be performed using various indices utilizing the geometric properties of the structure. The simplified indices proposed by Lourenco *et al.*[6] for masonry buildings have been utilized to assess the seismic vulnerability of four monastic temples in Sikkim. This simplified method is reported to yield best results if the structure is regular and symmetric with rigid diaphragm, and the in-plane shear failure is the dominant failure mechanism. However, the old monasteries do not satisfy requirement of rigid diaphragm, but these indices will provide an important insight

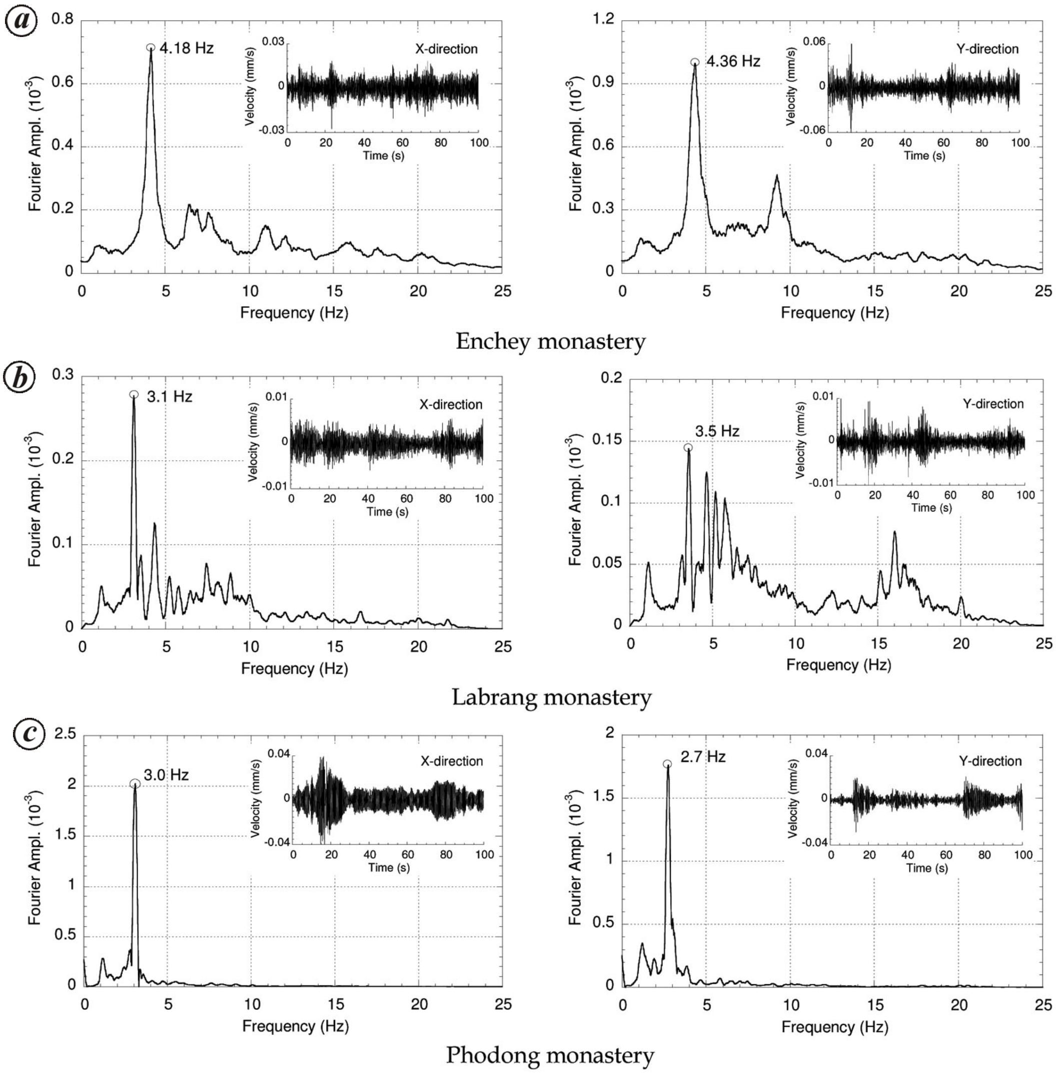

Figure 11 ***a–c*****.** Recorded time-history and Fourier spectra at three monasteries in the x-direction parallel and y-direction perpendicular to the main entrance.

Table 3. Value of five indices for four monasteries

	Index 1 (%)		Index 2 (m^2/MN)		Index 3		Index 4	Index 5
Monastery	$\lambda_{1,x}$	$\lambda_{1,y}$	$\lambda_{2,x}$	$\lambda_{2,y}$	$\lambda_{3,x}$	$\lambda_{3,y}$	λ_4	λ_5
Enchey	9.4	10.2	3.1	3.4	0.83	0.90	25.9	0.13
Labrang	6.0	7.9	2.3	3.1	0.67	0.88	25.76	0.13
Phodong	2.7	3.6	2.1	2.8	0.62	0.81	36.47	0.09
Pubyuk	4.2	4.1	3.7	3.6	0.75	0.73	28.19	0.12

Underlined values violate the recommended limits for seismic safety in regions of high seismicity with peak ground acceleration (PGA) ≥ 0.25 g.

to the seismic vulnerability of the monastic temples. There are three in-plane and three out-of-plane indices. The following indices (eqs (1)–(3)) have been proposed as in-plane indices

Index 1: In-plan area ratio, $\lambda_{1,i} = A_{w,i}/A_p$, (1)

Index 2: Area to weight ratio, $\lambda_{2,i} = A_{w,i}/W$, (2)

Index 3: Base shear ratio, $\lambda_{3,i} = V_{c,i}/V_D$, (3)

where $A_{w,i}$ is the area of the earthquake force-resistant wall in the i direction, A_p the total floor plan area, W the weight of the structure, V_D the total seismic demand, and

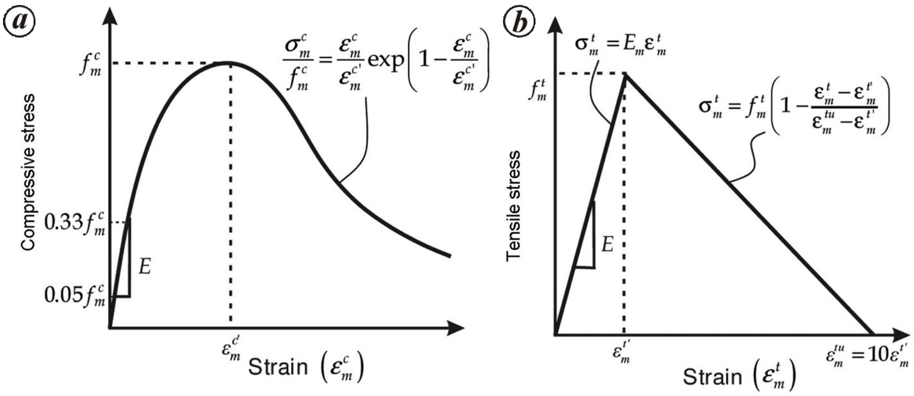

Figure 12. Constitutive model of stone masonry with weak mortar: ***a***, compressive stress–strain curve; ***b***, tensile stress–strain curve.

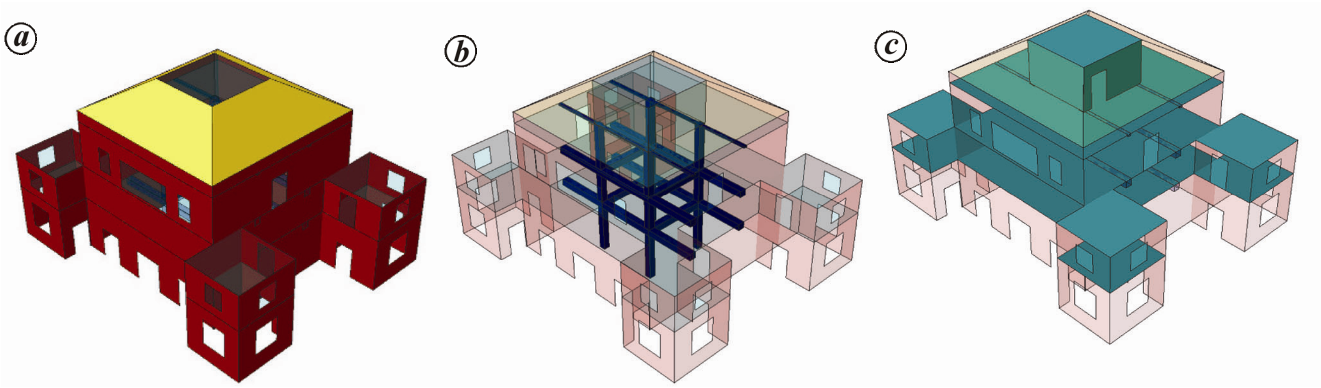

Figure 13. View of finite element models: ***a***, Exterior masonry walls and pitched roof of GI sheet; ***b***, Frame system of the structure comprising wooden beams and column; ***c***, Roof and floor diaphragms.

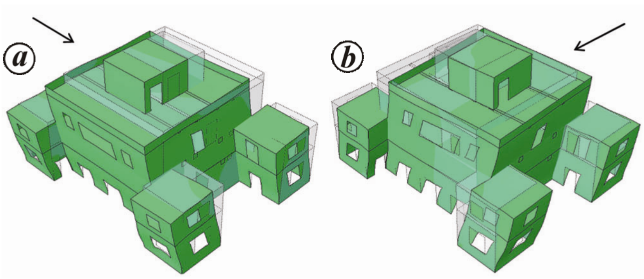

Figure 14. Mode shapes for natural frequency: ***a***, *x*-direction; ***b***, *y*-direction.

$V_{c,i}$ is the seismic capacity in the i direction. V_D can be calculated using Indian seismic code IS 1893 (ref. 7) as $V_D = \alpha_h \times W$, where α_h is the equivalent seismic coefficient. The seismic capacity of the masonry building is estimated using the following equation

$$V_{c,i} = \sum A_{w,i} \times f_v, \tag{4}$$

where according to Eurocode 6 (ref. 8), $f_v = f_{v0} + 0.4\sigma_d$. Here, f_{v0} is the cohesion in the masonry wall, which is assumed as zero due to absence of relevant information and σ_d is the design value of the normal stress.

The in-plan area ratio $\lambda_{1,i}$ indicates that a minimum area of earthquake resisting walls in the i direction is required for a given plan area and its threshold value for the structure to be deemed as safe increases with peak ground acceleration (PGA) value. In case of high seismicity or PGA $\geq 0.25g$, the minimum value of this non-dimensional index $\lambda_{1,i}$ is recommended as 10% for historical masonry buildings[6]. The area to weight ratio $\lambda_{2,i}$ indicates that a minimum wall area is required for a given weight of the structure, which indirectly takes into account the height of the building. Its threshold value increases with the expected PGA level and minimum value should be 2.5 m^2/MN for PGA = 0.25 g. The base shear ratio $\lambda_{3,i}$ indicates that the shear demand should be less than the shear capacity of masonry walls and is constant for different seismic zones, as it considers the effect of seismicity. Index 3 resembles traditional seismic safety approach for structural design and its threshold value is equal to 1.

Besides the above three in-plane indices, Lourenco *et al.*[6] proposed three indices for structural out-of-plane behaviour of columns and walls as given below

$$\lambda_4 = h_{\text{col}} / \sqrt{I_{\text{col}} / A_{\text{col}}}, \tag{5}$$

$$\lambda_5 = d_{\mathrm{col}}/h_{\mathrm{col}}, \quad (6)$$

$$\lambda_6 = t_{\mathrm{wall}}/h_{\mathrm{wall}}, \quad (7)$$

where h_{col} and d_{col} are the height and equivalent diameter of the columns respectively, I_{col} is the moment of inertia and A_{col} is the cross-section area of columns. t_{wall} and h_{wall} are the thickness and height of the perimeter walls respectively.

The indices λ_4 and λ_5 estimate the out-of-plane stability of columns present in the structure. The minimum value of these non-dimensional indices is recommended as 40 and 0.10 for PGA = 0.25 g respectively. λ_6 provides a check for out-of-plane stability of masonry walls based on its thickness to height ratio. However, its threshold limit is defined for good-quality brick work and dressed stone masonry, which may not be applicable for random rubble stone masonry, commonly used in perimeter walls of monastic temples. Thus, only indices λ_4 and λ_5 were evaluated for these temples.

According to IS 1893 (ref. 7), Sikkim lies in the seismic zone IV of India, for which the design PGA value is taken to be 0.24 g. Thus, the limiting values of indices 1–5 defined at a PGA value of 0.25 g are appropriate for the vulnerability assessment of stone masonry monastic temples in Sikkim. Table 3 provides details of the in-plane and out-of-plane indices of Enchey, Labrang, Phodong and Pubyuk monasteries. All the monasteries violate index 1, i.e. the in-plan area ratio λ_1 in both directions, except for Enchey monastery, which marginally satisfies it in the y-direction. Index 2, λ_2 is satisfied by all except Labrang and Phodong in the x-direction. The values of index 2 suggest that temples with larger wall area and lower floor height are seismically much safer. All monasteries without any exception violate index 3, λ_3, i.e. base shear ratio, indicating low masonry strength. All the monasteries satisfy the two out-of-plane criteria for columns, except Phodong, which violates index 5. The observed damage during the 2011 Sikkim earthquake in slender RC columns in Phodong monastery validates this result. The seismic performance of these monasteries during the recent earthquake substantiates these results, showing that the monasteries built during this period are seismically vulnerable.

Detailed assessment using finite element analysis

To further understand the behaviour of the monastery temples and also predict their seismic vulnerability, the main temple at Enchey monastery was studied using FE analysis in Abaqus environment[9]. Modal analysis and response spectrum analysis (RSA) were carried out to determine the dynamic properties of the structure, whereas nonlinear pushover analysis was performed to obtain the global force–deformation response of the temple and seismic demands in terms of stresses in the various structural components.

The Enchey monastery is one of the most important monasteries of the Nyingma sect. It was built by Sikyong Tulku in 1909 on a hilltop about 3 km from Gangtok[3]. The main temple is a four-tiered construction with semi-dressed stone masonry walls on the outside and roof at three levels (Figure 3 *b*). The plan is symmetrical, enclosing a square area as a prayer hall with four smaller square halls at the corner of the main hall. The frame comprises four columns arranged in a rectangular grid with main beams running parallel to the entrance and transverse secondary beams in a perpendicular direction between the two columns.

In the present study, physical tests were not conducted for material characterization; however, the mechanical properties of the masonry have been chosen based on the available test data (prism compression and diagonal tension tests) on multi-wythe stone masonry[10–15]. Table 4 shows the properties of the stone masonry, corrugated GI sheets and timber used for analysis. The typical value of Young's modulus for stone masonry ranging from 200 to 1000 MPa has been reported[11]. A modulus of 300 MPa near lower values of the range was chosen considering the age, quality and type of masonry construction. Compressive strength of the multi-wythe stone masonry with weak mortar was found in the 0.5–1.5 MPa range[10–15]. The tensile strength was taken as one-tenth of the compressive strength based on the available test data. Timber frames and floors are considered as linear elements due to relatively lower stresses obtained in these members during the analyses. This assumption is also justified as no damage was observed in these members during the 2011 Sikkim earthquake.

The concrete damaged plasticity (CDP) model in Abaqus was used to simulate the inelastic behaviour of stone masonry. The model uses the concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behaviour. The CDP model is based on the non-associated flow rule and provides necessary control to dilatancy in modelling friction and quasi-brittle materials. A constitutive model proposed by Naraine and Sinha[16] was implemented to represent the compression behaviour of masonry. It was proposed that compression response

Table 4. Mechanical properties of materials used in the finite element modelling

Material	Density (kg/m^3)	Young's modulus (GPa)	Poisson ratio
R/R stone masonry	2000	0.30	0.20
GI sheet	7850	210	0.30
Timber frame	800	7	0.12
Timber floor	800	12	0.12

can be represented by an analytical formulation as given in Figure 12 *a*. A simplified tri-linear curve given by Akhaveissy *et al.*[17] was used to define the tensile behaviour of the masonry (Figure 12 *b*). Other material properties required for the CDP model were taken as default values: dilatancy angle = 30°, flow potential eccentricity = 0.1, ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress = 1.16, ratio of second stress invariant = 0.667, and viscosity parameter = 1×10^{-4}.

The masonry walls, floors and roof were modelled as 3D shell elements (S4R), while the timber frame members were modelled as 3D solid elements (C3D8R). The second and third storey floors were modelled as isotropic shell of thickness 145 mm and 210 mm to match the frequencies of two floors, 9.4 and 13.5 Hz respectively, as obtained by the ambient vibration test. Rigid connection was assumed between the column and beam joints and between the frame, wall and diaphragm. In the absence of connection details, a simple connection was assigned between various structural members using tie-constraints in the FE model. Fixed boundary condition was assumed at the base of the structure. The live load on the structure was ignored for the analyses, as it is negligible when compared to the dead load. Figure 13 provides a view of the FE model. Frequencies in the orthogonal direction: parallel (*x*-direction) and perpendicular (*y*-direction) to the entrance were found to be 4.1 and 4.3 Hz respectively (Figure 14). The obtained frequencies are in a good agreement with the observed values of 4.2 and 4.4 Hz measured by ambient vibration tests.

RSA considering all significant modes in both directions was performed to predict the seismic demand for zone IV (PGA of 0.24 *g*). For the design basis earthquake, 5% damped elastic design response spectrum for soil type-II was scaled so that the zero period acceleration is 0.18 *g*, which includes a load factor of 1.5 (Figure 15). The total base shear obtained from the analyses was 30% and 32% of the self-weight in the *x*- and *y*-directions 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. The average shear stresses were estimated as 0.11 and 0.12 MPa in the *x*- and *y*-directions respectively.

The pushover analysis of Enchey monastery was performed considering the lower and upper limits of available material properties, i.e. f_m^c was taken as 0.5 and 1.5 MPa. The pushover analysis was carried out by applying uniformly distributed lateral load across the height of the monastic temple. Similar approximation in lateral load application was assumed in available analytical studies on masonry monuments with complex geometric configurations[18]. Figure 16 shows the base shear versus roof drift curves obtained from the pushover analyses along both *x*- and *y*-directions. Due to similar geometric dimensions in the *x*- and *y*-directions, pushover curves are nearly the same in both orthogonal directions. These curves also mark the collapse prevention (CP) limit taken as 0.5% drift, which indicates the maximum design deformation capacity of the structure. The CP limit for stone masonry with weak mortar was obtained from past experimental studies. It ranges between 0.3% and 0.7% and an average value of 0.5% has been used in this study[19–22]. Figure 16 shows the design base shear demands (V_{D-IV} and V_{D-V}) corresponding to hazard levels of seismic zone IV and zone V. It can be seen that for masonry compressive strength of 0.5 MPa, the monastery is just safe for design base shear demand for zone IV (6504 kN), but exceeds the CP drift limit under base shear demand for zone V (9756 kN). This indicates a high probability of collapse of similar monasteries under design-level earthquakes for seismic zone V. However, for f_m^c = 1.5 MPa, the monastery does not exceed the CP limit for design earthquake loads in zones IV and V.

Figure 17 shows the contour plots of maximum in-plane stresses in both the *x*- and *y*-directions at the CP drift limit in masonry walls with compressive strength of 0.5 MPa (positive and negative stresses denote tensile and compressive stresses respectively). The tensile stresses observed at the corners of the openings exceed the lower-bound cracking value of 0.05 MPa, making them the most damage-prone areas of the structure (Figure 17). The locations of maximum stresses were the same as those of damages observed during the 2011 earthquake (Figure 7). The compressive stresses are maximum near the base of the structure and at corners the stresses are higher than the compressive strength. Regions of concentrated tensile stress are seen near the floor levels, which compare well with the observed damage at the connection between the diaphragm and walls. The high tensile regions in the upper storeys indicate the possibility of out-of-plane damage, similar to those observed in other monasteries, such as Ringhem Choling in Mangan (Figure 6 *a*).

The FE analysis highlights the vulnerable portion of stone masonry walls in the upper stories and especially

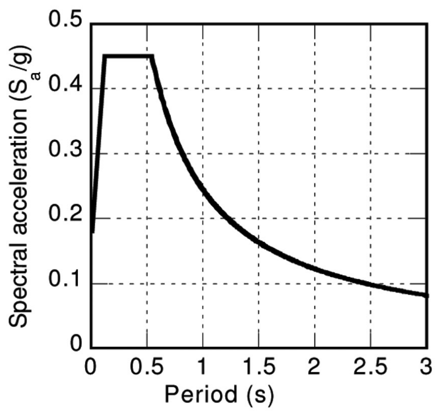

Figure 15. Response spectrum for zone IV soil type II with 5% damping.

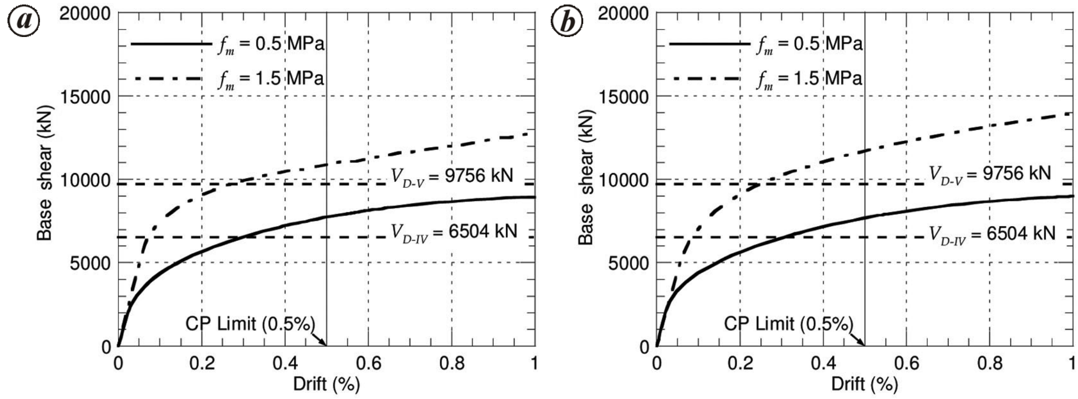

Figure 16. Pushover curve for the Enchey monastery for different masonry properties: ***a***, *x*-direction; ***b***, *y*-direction.

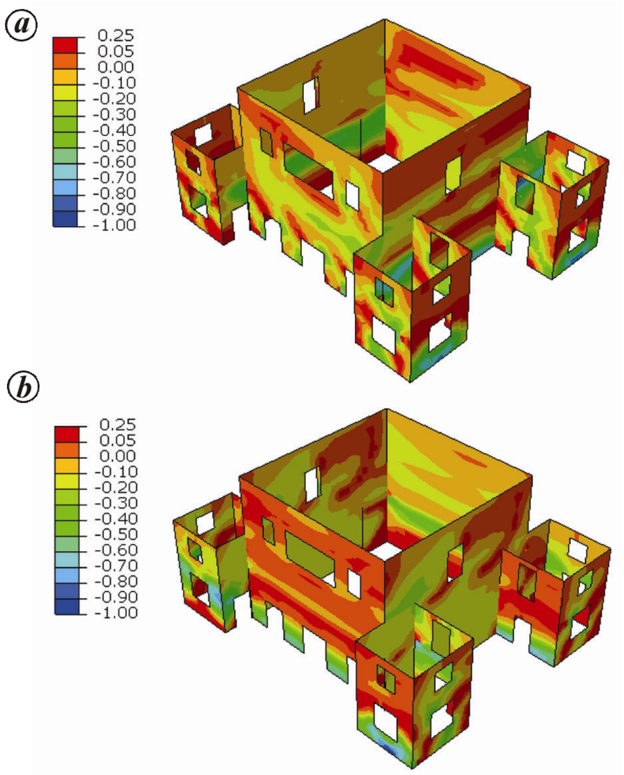

Figure 17. In-plane stress contour plots which identify areas of critical stress: ***a***, *x*-direction; ***b***, *y*-direction.

around openings. The interior wooden frame of buildings is relatively robust, which is also in correlation with no damage observed during the earthquake. These are short-period structures and hence their behaviour is dominated by acceleration response. In order to reduce/mitigate damage in future earthquakes, 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 peripheral 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 needs to be enhanced to prevent any major damage in future seismic events.

Conclusion

The damage caused to the studied monasteries during the 2011 Sikkim earthquake highlights the seismic vulnerabilities of these structures of cultural heritage. Major damage was 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 the main temples at the three monasteries showed that they are 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 nonlinear pushover analyses showed that the tensile stresses around the wall openings exceeded the permissible values and are, therefore, susceptible to damage. The pushover analysis highlighted that for a design-level earthquake in seismic zone V, the monastic temple could be subjected to drift levels higher than collapse prevention limits. The simplified safety indices were analysed for the quick assessment of vulnerabilities under earthquake loads and stone masonry construction of monastery structures were found seismically deficient according to these indices. Further, the predicted vulnerabilities from simplified safety indices and more refined FE analyses compared well with the damages observed in the recent earthquake. However, these preliminary results need to be further supported with detailed analyses. The vulnerability assessment using simplified safety indices and nonlinear FE analyses shows that the monastery structures which have survived the design-level earthquake shaking during the 2011 Sikkim earthquake are vulnerable to partial or complete collapse under stronger shaking and thus need urgent retrofitting plan to safeguard them against future seismic events.

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