SEISMIC FRAGILITY ANALYSIS OF STONE MASONRY MONASTIC TEMPLES IN SIKKIM HIMALAYAS

Anu Tripathi¹ and Durgesh C. Rai²

ABSTRACT

The Buddhist monasteries of Sikkim portray the history and culture of the place with their intricate art and architecture. These monasteries, located in the regions of high seismic activity in the Himalayas, have suffered varied degrees of damages in the past earthquakes. Proper restoration programs require knowledge of the seismic performance of these random-rubble stone masonry structures. This paper presents the seismic fragility analysis of four representative study monastic temples, namely Labrang, Enchey, Phodong and Pubyuk. The seismic fragility analyses of these temples were performed based on their pushover curves, using the capacity spectrum method, which accounts for the variability in the seismic loading. The analyses provided the probability of exceedance of a damage state for the earthquake levels expected in seismic zone IV of Indian seismic code. In this study, the damage levels of immediate occupancy (IO), life safety (LS), and collapse prevention (CP) were considered. A range of material properties were used in the model to study the effect of masonry properties on the fragility of the structure. These results highlighted the indispensability of strong masonry for better performance, as well as the detrimental impact of replacing timber with concrete frames. These curves can be used for suitable choice of retrofitting measures.

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The Buddhist monasteries of Sikkim portray the history and culture of the place with their intricate art and architecture. These monasteries, located in the regions of high seismic activity in the Himalayas, have suffered varied degrees of damages in the past earthquakes. Proper restoration programs require knowledge of the seismic performance of these random-rubble stone masonry structures. This paper presents the seismic fragility analysis of four representative study monastic temples, namely Labrang, Enchey, Phodong and Pubyuk. The seismic fragility analyses of these temples were performed based on their pushover curves, using the capacity spectrum method, which accounts for the variability in the seismic loading. The analyses provided the probability of exceedance of a damage state for the earthquake levels expected in seismic zone IV of Indian seismic code. In this study, the damage levels of immediate occupancy (IO), life safety (LS), and collapse prevention (CP) were considered. A range of material properties were used in the model to study the effect of masonry properties on the fragility of the structure. These results highlighted the indispensability of strong masonry for better performance, as well as the detrimental impact of replacing timber with concrete frames. These curves can be used for suitable choice of retrofitting measures.

Introduction

The Buddhist monasteries in Sikkim region constitute important group of architectural and cultural heritage. These random rubble (RR) stone masonry structures, some of which date back to 17th century, serve as meditation and learning centres for the Buddhist monks. Located in seismic zone IV of the Indian seismic zoning [1], the maximum shaking intensity of VIII on MSK scale is expected in the region. These structures have sustained varied levels of damage in the past earthquakes, ranging from total collapse to no damage [2], [3]. Their performance in the recent earthquakes emphasizes the urgency of determining their seismic vulnerability and taking appropriate retrofitting measures.

Detailed FE analyses of four archetypal monastic temples, namely Enchey, Labrang, Phodong, and Pubyuk, were performed to obtain the seismic performance and the lateral load-displacement

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behavior of these structures. The typical monastic temples, which are one to three storey pagoda style structures with square floor plan and pitched roof, were built with locally available material with primitive understanding of earthquake resistant structures. These temples have bulky exterior stone masonry wall with large openings and interior timber or concrete frames. The linear and non-linear finite element (FE) analyses of these temples identified the critical areas in the structures which matched well with the damages observed in recent earthquakes [4].

The seismic vulnerability of such structures can be assessed suitably using fragility analysis, which provides the probability of exceedance of a damage state under a given earthquake loading. An empirical fragility curve can be obtained based on building inspection data after earthquakes [5], [6]. Such fragility curves incorporate uncertainties in both, seismic demand and capacity. However, this method requires thorough inspection of substantial number of similar structures and does not lead to an insightful idea for retrofitting. On the other hand, an analytical fragility curve can be developed using results from numerical or experimental analyses. The material uncertainties can be incorporated in fragility curves using Monte Carlo simulations [7], or other sampling techniques such as Latin Hypercube Sampling [8]. The epistemic uncertainties resulting from modelling can also be similarly incorporated by varying geometric and mechanical details of the model [9]. The statistical uncertainties present in the seismic demand can be modelled based on incremental dynamic analyses (IDA) using multiple earthquake time histories [10], [8]. These analyses provide accurate results, however at a large computational cost, and are unsuitable for large models. For detailed FE models, nonlinear static analyses can be used to obtain fragility curves using capacity spectrum method (CSM) [11]. The CSM uses mean and standard deviation of the response spectrum of ground motions to account for the variability in seismic loading.

In this study, the static nonlinear analyses of the study temples were used to obtain the fragility curves using CSM. The modelling uncertainties were neglected considering the quality of available geometric data. In the absence of material data, the curves were developed for different sets of material properties, each accounting for demand uncertainties using the mean and standard deviation of the design response spectrum. The paper provides a detailed description of the parameters and steps in obtaining the fragility curves and interpretation of the fragility curves of the study monastic temples.

**Non-linear Finite Element Analyses of the Study Monastic Temples**

The 3D FE models of the central temples of the four study monasteries were developed in the software ABAQUS. These stone masonry walls were modelled using thick shell element S4R and the frames were modelled using 3D linear brick element C3D8R. The lack of experimental data on the connections led to model all the connections as monolithic connections using tie constrains. Pushover analyses of numerous stone masonry monuments have shown considerable agreement with actual observations in case of symmetric structures with high natural frequencies. This study assumes a dominant first mode behaviour of the structures based on the results of response spectrum analysis considering first 5 modes [4].

Non-linear pushover analyses of the study monastic temples were performed to understand their structural behaviour under seismic loads in greater details. These analyses were performed by
applying constant gravity loads, and monotonically increasing lateral loads, which were uniformly distributed across the whole height of the structure. In absence of the experimental characterization of the materials, a thorough survey of literature was carried out to obtain their nonlinear mechanical properties. Table 1 provides the average, lower, and upper bound of the mechanical properties of stone masonry considered in the study. The exponential constitutive model proposed for uniaxial compression by [12] and linear stress-strain model proposed by [13] were chosen for modelling stone masonry. The M20 (characteristic cube strength of 20 MPa) concrete in Phodong monastic temple was modelled with its compressive behaviour described using the [14] model and tensile behavior using the tension softening model by [15]. The elastic modulus and tensile strength were calculated as per [16]. The compressive and tensile strength of timber used in the interior floors and frames in Enchey, Labrang and Pubyuk monastic temples was taken as 40 MPa and 80 MPa, based on values reported in literature [17], [18] and it was modelled as a linear material.

Table 1. Stone masonry compressive strength ($f_m$), elastic modulus ($E$) and tensile strength ($f_t$)

<table>
<thead>
<tr>
<th>Properties</th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_m$</td>
<td>0.5</td>
<td>1.0</td>
<td>1.5</td>
</tr>
<tr>
<td>$E/f_m$</td>
<td>75</td>
<td>200</td>
<td>350</td>
</tr>
<tr>
<td>$f_t/f_m$</td>
<td>0.1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The seismic analysis of each monastic temple was performed for nine sets of masonry properties, under loading in $x$- and $y$-directions. Fig. 1 shows the exterior view, FE model, and pushover curves for Enchey, Labrang, Phodong and Pubyuk monastic temples for masonry property $f_m=1$ MPa. The pushover curves also show the drift corresponding to immediate occupancy (0.15%), life safety (0.25%) and collapse prevention (0.5%) damage states and the base shear demand imposed by design level earthquake in seismic zone IV (PGA = 0.24g) as per [1].

**Fragility Analyses**

The seismic vulnerability of the study monastic temples was assessed using fragility analyses. The fragility of a structure is defined as the probability that the structure would exceed a specified damage state under a given earthquake loading. The level of earthquake loading can be measured using the parameter called intensity measure, which was taken as peak ground acceleration (PGA) to assess the performance of the structures in the design level seismic event.

A fragility curve is developed for a damage state, which is the indicator of the performance level of the structure in terms of the physical damage and the rehabilitation requirements of the structure. In this study, the discrete damage states specified in [19] were utilized to assess the seismic performance of the monastic temples. These damage states included Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). The degree of damage to the structure and the damage states are expressed in terms of an easily measurable physical quantity called the damage measure. The damage measure considered in this study is the transient roof drift ratio whose value was obtained at each intensity measure. The damage state drift limits were obtained using literature survey of experimental investigation of the in-plane cyclic behaviour of stone masonry [20], [21], [22].

The drift limits for the damage states IO, LS and CP were taken to be the average drift at crack
initiation ($d_{ci}$), yield drift in the bilinear idealized backbone curve ($d_e$), and drift at maximum lateral load capacity ($d_{Hmax}$), respectively. A range of drift limits for each damage state was proposed to consider the variability in these limits from average – $SD$ to average + $SD$. Table 2 shows the average drift limits and their ranges for each damage state.

Figure 1. Exterior view, FE model, and pushover curves for (a) Enchey, (b) Labrang, (c) Phodong, and (d) Pubyuk
### Table 2. Average and range of drift limit for the damage states

<table>
<thead>
<tr>
<th>Damage States</th>
<th>Average Drift Limit (%)</th>
<th>Drift Range (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>IO</td>
<td>0.15</td>
<td>0.1 - 0.2</td>
</tr>
<tr>
<td>LS</td>
<td>0.25</td>
<td>0.2 – 0.3</td>
</tr>
<tr>
<td>CP</td>
<td>0.50</td>
<td>0.3 – 0.7</td>
</tr>
</tbody>
</table>

**Capacity Spectrum Method Based Fragility Curves**

The fragility curves of the study monastic temples were developed based on their pushover analyses using capacity spectrum method (CSM). CSM is a nonlinear static analysis procedure to evaluate the seismic performance of a structure. It uses the capacity spectrum, obtained using pushover curve, and the nonlinear response spectrum to find their intersection, the performance point. The guidelines provided in [23] and [24] were followed to carry out the method.

The capacity spectrum of a structure represents the capacity curve of the equivalent single degree of freedom system for structure in acceleration-displacement response spectrum (ADRS) format. The conversion of base shear and roof drift in pushover curves to spectral acceleration and spectral displacement provides the capacity spectrum. The spectral acceleration is obtained from the base shear using Eqs. 1 and 2 and the roof displacement is converted to spectral displacement using Eqs. 3 and 4.

\[
S_a = \frac{V}{W} \alpha_1
\]  
\[
\alpha_1 = \left[ \frac{\sum_{i=1}^{N} (w_i \phi_i) / g}{\sum_{i=1}^{N} w_i / g} \right]^{2}
\]  
\[
S_d = \frac{\Delta_{\text{Roof}}}{(PF_1)(\phi_{\text{Roof},1})}
\]  
\[
PF_1 = \left[ \frac{\sum_{i=1}^{N} (w_i \phi_i) / g}{\sum_{i=1}^{N} (w_i \phi_i^2) / g} \right]
\]

where, $PF_1$ is modal participation factor for the first natural mode, $\alpha_i$ is the modal mass coefficient for the first mode, $w_i$ is the mass of the $i^{th}$ floor, $\phi_i$ is the amplitude of mode 1 at level $i$, $N$ is the number of storeys in the structure, $V$ is the base shear of the structure, $W$ is the weight of the building, $\Delta_{\text{Roof}}$ is the roof displacement, $S_a$ is the spectral acceleration, and $S_d$ is the spectral displacement.

The modal mass coefficient, $\alpha_i$ is the ratio of the equivalent modal mass in the mode $i$ and the total mass of the system. The values of the participation factor and the equivalent modal mass used for the first mode were acquired from the modal analysis in ABAQUS. These parameters emphasize the significance of the fundamental vibration mode. It was observed that these values don’t change with material properties of the structural member. The participation factors and
modal mass coefficients for the monastic temples are given in Table 3. The capacity curves of
the monastic temples obtained in chapter four are utilized here to obtain the capacity spectrum.

Table 3. Participation factor ($PF_1$) and modal mass coefficient ($\alpha_1$) for first mode of the temples

<table>
<thead>
<tr>
<th>Monasteries</th>
<th>Participation factor ($PF_1$)</th>
<th>Modal Mass (kg)</th>
<th>Total Mass (kg)</th>
<th>Modal mass coefficient ($\alpha_1$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Enchey</td>
<td>1.78</td>
<td>1035000</td>
<td>1445000</td>
<td>0.70</td>
</tr>
<tr>
<td>Labrang</td>
<td>1.64</td>
<td>835000</td>
<td>1191000</td>
<td>0.71</td>
</tr>
<tr>
<td>Phodong</td>
<td>1.37</td>
<td>1892500</td>
<td>2573000</td>
<td>0.74</td>
</tr>
<tr>
<td>Pubyuk</td>
<td>1.51</td>
<td>244500</td>
<td>300000</td>
<td>0.82</td>
</tr>
</tbody>
</table>

The demand spectrum used in finding the performance point was reduced from the elastic
spectrum to consider the nonlinearity of the structure. The non-linearity in structure’s behaviour
causes an increase in the effective time period and effective damping which in turn changes the
demand. The initial time period was taken to be that obtained using the modal analysis and initial
damping as 5 percent. The procedure ‘A’ specified in [23] was used for determining the
performance point and bilinear idealization of the capacity spectrum. The effective time period
and damping were calculated using the bi-linear idealization of the capacity curve and result in
the reduced inelastic response spectrum. The spectral reduction factors $SR_d$ and $SR_V$ were
obtained using the effective time period and damping using [23]. $SR_d$ is applied at the constant
acceleration region of the spectrum and $SR_V$ at the constant velocity region of the spectrum to
give inelastic response spectrum. The procedure is repeated with the new position of the
performance point until its position converges.

The procedure to derive CSM based fragility curves require the mean and the mean + standard
deviation response spectra of considered ground motions [11]. The response spectrum specified
in [1] for medium soil type is taken as the mean response spectrum. The mean + standard
deviation response spectrum is obtained by amplifying the mean response spectrum by a factor
of 1.5. These mean and mean + standard deviation elastic response spectra are shown in Fig. 2a.

For each PGA value, two response spectra, namely mean and mean + standard deviation were
obtained which yielded two performance points corresponding to the mean and mean + standard
deviation seismic demand. The spectral displacement values for mean performance point are
referred to as $S_{d_{mean}}(PGA)$ and that for mean + standard deviation is referred to as
$S_{d_{mean+SD}}(PGA)$ (Fig. 2b).

The spectral displacement was reasonably taken as log-normally distributed, with the mean equal
to $S_{d_{mean}}(PGA)$ and standard deviation equal to $\sigma_d (PGA)$, which is the difference between
$S_{d_{mean+SD}}(PGA)$ and $S_{d_{mean}}(PGA)$. The parameters of two-parameter log-normal distribution of
spectral displacement, $c(PGA)$ and $\zeta(PGA)$ were obtained by solving Eqs. (7) and (8).

$$S_{d_{mean}}(PGA) = c(PGA) \exp \left[ \left( \zeta(PGA) \right)^2 / 2 \right]$$  \hspace{1cm} (7)

$$\left( \sigma_d(PGA) \right)^2 = \left( S_{d_{mean}}(PGA) \right)^2 \exp \left[ \left( \zeta(PGA) \right)^2 - 1 \right]$$  \hspace{1cm} (8)
The damage measure considered in this study is the transient roof drift ratio whose value was obtained at each PGA level. The drift ratio of the model obtained for a given PGA value was assumed to be log normally distributed due to the variability in earthquake loading. The probability of exceedance of the damage state of the structure is the same as the probability of the drift ratio exceeding the limit of the damage state. The parameters of log-normal distribution of the roof drift, $c'(PGA)$ and $\zeta'(PGA)$, were obtained from $c(PGA)$ and $\zeta(PGA)$ using Eqs. 9 and 10, respectively. Thus, the probability of exceedance of a damage state is given by Eq. 11.

$$c'(PGA) = c(PGA) \frac{(PF_{\phi}) (\Phi_{Roof,\phi}) 100}{H}$$  \hspace{1cm} (9)\\

$$\zeta'(PGA) = \zeta(PGA) \frac{(PF_{\phi}) (\Phi_{Roof,\phi}) 100}{H}$$  \hspace{1cm} (10)\\

$$P(PGA, d_{ij}) = P(S_d(PGA) \geq d_{ij}) = 1 - \Phi \left[ \frac{\ln(d_{ij}) - \ln(c'(PGA))}{\zeta'(PGA)} \right]$$  \hspace{1cm} (11)\\

where, $H$ is the total height of the monastic temple, $\Phi$ is the cumulative distribution function for standard normal distributions, and $P$ is the probability of exceedance of the damage states. The fragility curve is obtained by plotting the value of probability of exceedance obtained using Eq. 11 versus PGA.

**Results and Discussion**

The fragility curves for the four study monastic temples were developed for each of the nine sets of stone masonry material properties mentioned in Table 1. Fig. 3 shows the fragility curves for the study monastic temples for the weakest and the strongest masonry properties. A summary of the expected damage state of the structures with weakest, average and strongest masonry properties are given in Table 4. Each plot shows the fragility curves for the damage states IO, LS and CP. Due to variability in the drift limits of these damage states, the fragility curves are drawn for the average, lower, and upper bounds of damage state drift limits. The shaded areas between the fragility curves shows the zone where the corresponding fragility curve would be present.
depending upon the cyclic in-plane pushover behaviour of the stone masonry. The expected response of the structure under a PGA level is taken to be the drift level with 50% probability of exceedance.

Figure 3. Fragility curves using weakest ($f_m = 0.5$ MPa and $E/f_m = 75$) and strongest ($f_m = 1.5$ MPa and $E/f_m = 300$) masonry properties for (a) Enchey, (b) Labrang, (c) Phodong, and (d) Pubyuk
Table 4. Expected damage states of the study monastic temples with weakest, average, and strongest masonry properties under design level earthquake in zone IV

<table>
<thead>
<tr>
<th>Monastic Temple</th>
<th>$f_m$ (MPa)</th>
<th>$E/f_m$</th>
<th>Expected Damage State</th>
</tr>
</thead>
<tbody>
<tr>
<td>Enchey</td>
<td>0.5</td>
<td>75</td>
<td>CP</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>200</td>
<td>No Damage</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>350</td>
<td>No Damage</td>
</tr>
<tr>
<td>Labrang</td>
<td>0.5</td>
<td>75</td>
<td>CP</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>200</td>
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<td>75</td>
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</tr>
<tr>
<td></td>
<td>1.0</td>
<td>200</td>
<td>CP</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>350</td>
<td>No Damage</td>
</tr>
<tr>
<td>Pubyuk</td>
<td>0.5</td>
<td>75</td>
<td>CP</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>200</td>
<td>No Damage</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>350</td>
<td>No Damage</td>
</tr>
</tbody>
</table>

Under zone IV design PGA=0.24g, all the monasteries would reach CP damage state if the masonry properties are comparable to the weakest, i.e. $f_m = 0.5$ MPa and $E/f_m = 75$. However, if the masonry properties are comparable to the strongest, i.e. $f_m = 1.5$ MPa and $E/f_m = 350$, Enchey, Labrang and Pubyuk would not show any damage, but only Phodong would reach IO damage state under PGA=0.24g. Similarly, under seismic loading demand of zone V, the temples of Enchey, Labrang, and Pubyuk would show no damage for strongest material properties, whereas Phodong would reach LS damage state. It was seen that the Phodong monastic temple shows higher expected damage state than Enchey and Labrang for all the material properties. This can be attributed to the nonlinearity introduced in the frames due to replacing timber with reinforced concrete, which leads to high drift demand on the stone masonry. These curves highlight the importance of strong masonry in the global response of these structures. These curves can be used to estimate the required masonry properties for desired probability of a damage state under given loading.

Conclusion

The fragility curves for the four archetypal monastic temples were developed for each of the nine sets of stone masonry material properties based on their pushover analyses using CSM. These fragility curves provided the vulnerability of these temples associated with IO, LS, and CP damage states. It was observed that the monastic temples with the weakest masonry properties ($f_m=0.5$ MPa and $E/f_m=75$) showed unacceptably high probabilities of collapse under the design level earthquakes in seismic zone IV, whereas those with stronger material properties ($f_m=1.5$ MPa and $E/f_m=350$) showed no damage or IO damage state. Phodong monastic temple showed higher expected damage state than other monastic temples due to higher drift, attributed to replacing timber columns with reinforced concrete. These results highlighted the indispensability of strong masonry for better performance of the structures and can be used for proper allocation of retrofitting measures.

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