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Mapping the seismic safety of RC "Template Schools" in Nepal

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Abstract

Seismic risk assessment of a template school across Nepal is carried out to evaluate the suitability of a single design structure across an area with variable seismic hazard. Many non-governmental organisations (NGOs) use template designs for buildings as a method of reducing design costs; this results in a single seismic design used over a large area. Such an approach is particularly popular in Nepal as the government requires that all school designs obtain approval from the Department of Education. Probabilistic seismic hazard analysis (PSHA) studies have shown a varying seismic hazard across Nepal with the PGA a 475-year return event ranging from 0.05 g to 0.73 g. Pahar trust, a charity that builds schools in Nepal, provided the design of a reinforced concrete (RC) structure with masonry infills. Finite element (FE) models and fragility curves are developed with and without the masonry infill as to explore the effect of masonry infills on the seismic risk. The hazard and the fragility are then integrated to find the seismic risk of the template school across Nepal. The damage distribution is then evaluated for the template school under a 475-year and 2475-year earthquake. The probability of the school exceeding damage limitation (DL) and life safety (LS) has significant geographic variation, with a 475-year earthquake causing the school to exceed DL in most locations and a 2475-year return earthquake causing the school to exceed LS in the majority of locations. The structural contribution of infills brings the risk level below the threshold typically employed as reference in Europe.

Key Words: Nepal, School, Fragility, Gorkha, NGO, RC with infill

1. Introduction

Both non-government organisations (NGOs) and intergovernmental organisations play a crucial role in the reconstruction of schools after a large earthquake; this has been seen in Haiti 2010 ([World Bank, 2013](#)) and Nepal 2015 ([Unicef, 2018](#)). Structural characteristics of schools, such as large rooms with large openings (doors and windows) as well as a high concentration of people result in them being especially vulnerable ([Gentile et al., 2019a](#)). It is common practice to use a "template design", where one design is used at many sites to reduce design time and cost.

The Nepalese Department of Education, in collaboration with the Asian Development Bank and the Japanese International Cooperation Agency, (JICA) have set out strict guidelines on school designs ([Nepal Department of Education, 2016](#)). This was set up in response to many schools in Nepal not being designed in adherence with seismic codes even when they are in place ([D'Ayala et al., 2020](#)). Once a design has been developed and satisfies these criteria, it must then be approved by the Nepalese Department of Education. On top of ensuring that the schools meet the required guidelines, the Nepalese department of education is also responsible for ensuring that all school buildings are designed in accordance with the Nepalese building code ([Jones et al. 2016](#)). This can be a costly and time-consuming process. Therefore, many NGOs use a template school design to avoid having to repeat this process

multiple times. These designs may have minor alterations at different sites based on the availability of materials (Pahar Trust, 2019). Applying a single design across a country with non-homogeneous seismic hazard leads to a varying seismic risk of the structures. Furthermore, updated probabilistic seismic hazard analysis (PSHA) studies have peak ground acceleration (PGA) values reflecting the updated knowledge on the seismic sources. This study uses the results of recent PSHA studies in combination with the results of nonlinear time history analyses to assess the seismic risk of a template school across Nepal, and, therefore, to evaluate the variability of the “implicit risk” in the template school practice all around the country. The implicit risk is defined as the unavoidable seismic risk that a building will exceed a given limit state (Building Seismic Safety Council, 1988) and in this context represent the result of direct quantification of the risk of a structure designed according to code standard assessed explicitly through direct integration of hazard and fragility for different limit states (e.g., Iervolino et al. 2018).

Regional-scale seismic safety assessments are carried out on critical infrastructure such as schools as it allows local governments to address the issue of seismic safety using methods specific to their infrastructure and seismic hazard (Grimaz et al, 2016). Regional-scale seismic risk assessment has previously been applied to RC schools with masonry infills in Italy as part of a seismic loss assessment for the Italian school building stock (O’Reilly et al, 2019, Perrone et al, 2020a).

PSHA is the methodological approach underpinning most of modern design codes such as Eurocode 8 (EN 1998-1, 2004) and the unified building code (UBC-97, 1997). PSHA studies can be limited by availability of data for the tectonic faults in an area, fault modelling techniques and ground motion prediction equations (GMPEs) available. Many PSHA studies have been carried out on Nepal with widely varying results. PGA values for Kathmandu with a 475-year return period varied from 0.35g-0.55g for PSHA studies carried out prior to the Gorkha 2015 earthquake (Chaulagain et al. 2015a, Ram and Wang, 2013). More recent studies, benefitting of the additional evidence provided by the 2015 earthquake, have put the PGA for Kathmandu over 0.65g when considering a 475-year return period (Stevens et al. 2018, Pokhrel et al. 2019). These high levels of variation in PSHA results will have a significant impact on the risk of template schools.

Pahar Trust is a charity based in the UK and founded in 1991 by Tom Langridge MBE and Chandra Bahadur Gurung. Their work is split into construction work and educational improvements. The construction work involves the construction of four to six reinforced concrete (RC) schools per year and the educational improvements involve teacher training and resource improvements (Pahar Trust, 2020). Pahar Trust has provided the design of a template school for this study. It is a six-classroom school set across two stories made of RC with masonry infills. The structure has a seismic gap to reduce the torsional effects caused by the off-centre staircase.

A fragility assessment will be used to assess this template school which will then be combined with the hazard curves across Nepal to assess the variation of risk across the country when considering a single template design. A fragility curve relates a measure of ground shaking (i.e Intensity Measure, IM) to the probability of exceeding some level of response or damage state. Fragility curves can be obtained empirically, using historical data, or analytically by performing structural analysis on the building (Choi et al, 2004). Cloud analysis is a method to obtain a fragility curve by using linear regression in log-space with the results of nonlinear time history analysis (NLTHA) with a range of ground motions (e.g., Jalayer et al. 2014). A critical demand parameter or capacity limit is set based on the limit state and a cumulative probability function is

then obtained. Integration is then used with the hazard curves provided by Stevens et al (2018) to evaluate the mean annual frequency (MAF) of exceeding various limit states.

This paper addresses the suitability of template schools across a region of varying seismic hazard and assesses the influence on risk assessment associated with the explicit consideration of masonry infill contribution in the structural model. A comparison of the efficiency of different intensity measures is carried out for the considered structures considering as IM the peak ground acceleration (PGA) and the average spectral acceleration (AvgSA) (Kohrangi et al. 2017, De Luca et al. 2019).

Finite element models are developed as to obtain a fragility curve for the template school. This fragility curve can be used to assess how the structure will most likely respond to a given ground motion. A hazard curve is used to identify what level of intensity of earthquake is expected in a given time period. The information from the hazard curve and fragility curves can then be combined to calculate what the risk is for the structure. This risk is measured in terms of probability of the structure exceeding a specific limit state in a given time. These risk levels can finally be compared to reference levels from design codes to assess whether these template schools have an acceptable level of seismic risk.

2. Structural Modelling

Pahar Trust provided the design of a template school which is a two-storey building with six classrooms. The total height of the building is 5.8 m, and dimensions in plan are 4.9 m and 20.3 m, see Figure 1. The template school is a reinforced concrete frame with masonry infill which is one of the structural typologies which is commonly used for schools in Nepal (De Luca et al., 2019). The design included the beam column layout, shown in Figure 1c, as well as the layout of the steel reinforcements. The steel class for the rebar was specified as FE500 and the concrete was specified as being M20 concrete. FE500 steel has a yield stress of 500 MPa and a Young's modulus of 210 GPa, as set out by the India standards (IS 1786, 2008). M20 concrete has a characteristic strength of 20 MPa and a Young's modulus of 22.3 GPa, as set out by Indian standards (IS456-2000, 2000). The masonry infill was specified as "fired brick masonry" with a mortar of a cement sand ratio of 1:6. Due to the nature of construction practices in Nepal, the materials and the modelling techniques used there are inherent uncertainties which could have a significant effect on the final risk assessment. These can be reduced with a high level of onsite quality control. These uncertainties are not accounted for in this study with the focus of this work being on the variability of risk caused by the hazard. It can be noted that the quality of construction practices in projects carried out by NGOs is typically higher with respect to the general population. This is often due to the presence of a qualified engineer onsite. The structure had a 102 mm (4 inch) seismic gap, as shown in Figure 1c between gridline 3a and 3b, to avoid torsion caused by the off-centre staircase. The beam and column section properties can be seen in Figure A1 in the appendix. As there is no structural interaction across the seismic gap as shown by the probabilistic assessment of pounding in Cross et al. (2019), the building was split into two separate finite element (FE) models.

Figure 1. Pahar Trust template RC with infill school: (a) 3D architectural rendering; (b) example of realised project; (c) beam and column layout of the template school (units in mm) (Pahar Trust, 2020).

Based on the structural drawings provided, a nonlinear finite element model in OpenSees was developed. In particular, two models were developed. The first model only accounted for the RC beams and columns and did not take into account the structural contribution of the masonry infills. This is an approach consistent with the design practice adopted by the Nepalese National building code (NBC 201, 1994) and it is also the standard design practice according to Eurocode (e.g., Dolšek and Fajfar 2001). This model will hereafter be referred to as the "bare model". The second model will consider explicitly the structural contribution of the masonry infills using a triple strut model, as described in the following. This model will hereafter be referred to as the "infilled model"

2.1 Element Classification

To assess the modelling details of the Reinforced Concrete (RC) elements, a preliminary ductility classification is carried out to ensure that maximum shear as a result of flexural behaviour (V_{flex}) does not exceed the shear capacity (V_{shear}) of the element. Such assessment is carried out classifying the elements by employing the shear capacity model proposed by the Eurcode 8 part 3 (EN 1998-3, 2005). The approximate classification assumed here as implemented by De Luca and Verderame (2013) allows classifying elements as ductile elements, limited ductility elements and brittle elements. The maximum shear as a result of flexural behaviour can be calculated using equation 1. Such classification is a preliminary assessment of the seismic detailing of the RC elements in the structure.

$$V_{flex} = \frac{1}{L_v} \left[\frac{Nh}{2} (1 - v) + f_y A_s (d - d') \right] \quad (1)$$

In equation 1, N denotes the axial load applied to the element based on seismic gravity loads, L_v is the shear length, h denotes the height of the section, v denotes the normalised axial force, f_y is the yield strength of steel, A_s is the cross-sectional area of longitudinal steel reinforcement, d is the effective depth and d' is the depth to the compression reinforcement. The normalised axial force, v (also known as the axial load ratio) is the ratio of the applied axial load to the axial load capacity defined as the product of the design concrete strength and the cross sectional area. The shear capacity is calculated from Eurocode 8 part 3 through equation 2, where μ_{pl}^A denotes the ratio of the plastic part of the cord rotation, γ_{el} is a factor set to 1.15 for primary seismic elements, x is the compression zone depth, A_c is column cross sectional area, f_c is concrete compression strength, ρ_{tot} is total longitudinal reinforcement ratio and V_w is the contribution of transverse reinforcement to shear resistance. For $V_{shear,min}$, μ_{pl}^A is assumed to take a value of 0, meaning there is no plastic rotation and, for $V_{shear,max}$, it is assumed to take the max value of 5 as imposed by Eurocode 8.

$$V_{shear} = \frac{1}{\gamma_{el}} \left[\frac{h-x}{2L_v} \min(N, 55A_c f_c) + \left(1 - 0.05 \min\left(5, \mu_{pl}^A\right)\right) \cdot \left[0.16 \max(0.5, 100\rho_{tot}) \left(1 - 0.16 \min\left(5, \frac{L_v}{h}\right)\right) \sqrt{f_c} A_c + V_w \right] \right] \quad (2)$$

For each element V_{flex} , $V_{shear,min}$ and $V_{shear,max}$ is calculated, and the criteria set out in **Table 1** was used to classify the ductility of the element. Each element must also be classified for the negative flexure demand, if the reinforcement in the section is asymmetric (i.e., typical case for beams).

Table 1. Classification of RC element behaviour

Criteria	Classification
$V_{shear,max} > V_{flex} < V_{shear,min}$	Ductile
$V_{shear,max} > V_{flex} \geq V_{shear,min}$	Limited Ductility
$V_{shear,max} \leq V_{flex} > V_{shear,min}$	Brittle

The element classification performed on the school building led to an expected ductile behaviour for all elements which then influenced the modelling approach implemented for beam and columns (see next section).

The shear capacity of the columns is also checked under the Nepalese Standard (NBC 110, 1990) which uses the shear capacity model from the Indian Standard (IS 456, 1978). The shear capacity of the column is calculated using **equation 3** where τ'_c is the shear strength given by Indian Standard (IS 456, 1978) developed by Taylor (1972) as a function of the concrete grade and the percentage longitudinal reinforcement. The f_y denotes the yield strength of the steel, A_{sv} denotes the total cross-sectional area of shear steel, d denotes the effective depth and s_v is the spacing of the shear reinforcement. Using the Indian Standard the peak shear demand over capacity ratio is found to be 0.74 in the analysis in this study, showing that the column will not experience shear failure.

$$V_u = \tau'_c b d + \frac{0.87 f_y A_{sv} d}{s_v} \quad (3)$$

2.2 Bare Model

A fibre model was created of the RC frame of the structure. This was implemented in OpenSees using the "Hinge Radau" function (Scott et al. 2006) which uses two-point Gauss-Radau integration over the hinge length. The length of the plastic hinge (L_{pl}) is calculated in accordance with Eurocode 8 part 3 (EN 1998-3, 2005), where L_v is the shear length assumed as half length of the element (see Lombardi, 2019).

The grade of steel rebar is specified by Pahar Trust as "FE500". This is implemented into the fibre model using the constitutive nonlinear hysteretic steel model as developed by Menegotto and Pinto (1973) and extended by Filippou et al. (1983). For FE500 steel, a yield strength of 500 MPa is used, and Young modulus of 210 GPa is used in accordance with Indian standard (IS 1786, 2008).

The strength class of concrete is specified by Pahar trust as being M20. This is the lowest permissible strength class of concrete for reinforced concrete by the Indian

standard (IS456-2000, 2000). This concrete is formed of a characteristic 28-day crushing strength of 20 MPa and a target (mean) strength of 30 MPa. M20 concrete has a cement:sand:aggregate ratio of 1:1.5:3 and has an elastic modulus of 22.3 GPa (IS456-2000, 2000). Other studies within Nepal have assumed the same concrete properties (Chaulagain et al, 2015b).

The Kent-Scott-Park (Scott et al., 1982) model is used for the response of the concrete in the FE model. The additional strength of the concrete due to the confinement was found using the Kent and Park model (1971) that was modified (Park et al., 1982), resulting in a unique confined concrete strength value for each section due to the varying ratio of confining steel.

2.3 Infilled Model

The masonry infills are implemented in the finite element (FE) model using a triple strut model. The method of using struts to replicate the effect of the masonry infill was first proposed in the 1960s (Polyakov 1960). In this study, the three-strut model is used as it has been implemented in many studies of RC with masonry infill (Perrone et al., 2020b) and has been verified with experimental data (Blasi et al., 2019). Since this method was initially proposed there have been several improvements; however some failure mechanisms exhibited by masonry infills are not accounted for (Stavridis et al., 2010).

The individual hysteretic properties for each strut are modelled using a piecewise linear load-displacement behaviour. The shape of this hysteretic response is calculated according to Panagiotakos and Fardis (1996) using the overstrength factor for solid bricks (i.e., 1.55) calibrated by Blasi et al. (2019). This method is commonly used in literature and has been verified using experimental data (e.g., Noh et al., 2017) based on the experimental tests calibrated by Blasi et al. (2019).

To implement this model three material properties are required for the masonry wall, these are the Young modulus of the masonry infill (E_{inf}), cracking shear strength of masonry infill (τ_{cr}) and the shear modulus of the masonry infill (G_w). Both E_{inf} and τ_{cr} are taken from the experimental work carried out by Shahzada et al (2012). This experimental work carried out a diagonal compression test on a masonry specimen typical of Pakistan where the construction techniques are similar to those in Nepal (Varum et al., 2016). This masonry specimen was formed of burnt clay bricks and mortar with a cement-to-sand ratio of 1:6. This is the same type of brick and sand-to-cement ratio as specified by Pahar Trust. The E_{inf} value is taken to be 1,227 MPa, and the τ_{cr} value is taken to be 0.05 MPa. The shear modulus of the masonry (G_w) which can be taken as 40% of the elastic modulus according to Eurocode 6 (EN 1996-1-1 2005). The final material property that is required is the Young modulus of the concrete (E_c); this is taken to be 22.3 GPa, as outlined in section 2.2. These material properties are summarised in Table 2.

Table 2. Material properties for model of masonry infill

Property	Symbol	Value	Reference
Young modulus of masonry infill	E_{inf}	1,227 MPa	(Shahzada et al, 2012)
Cracking strength of masonry	τ_{cr}	0.05 MPa	(Shahzada et al, 2012)
Shear modulus of masonry	G_w	491 MPa	(EN 1996-1-1 2005)
Young modulus of concrete	E_c	22.3 GPa	(IS456-2000, 2000)

The hysteretic response of the struts is also a function of the geometry of the wall, which varies across the structure. The three dimensions of the infill are t_w , L_w and h_w , which are the thickness, length and height of the infill wall, respectively. Using these dimensions, the diagonal length of the wall (d_w) can be calculated. The angle of the diagonal dimension of the panel (θ), shown in [Figure 2](#), is also required. The final geometric parameter required is the moment of inertia of the RC column (I_c).

Table 3. Equations used to model the hysteretic response of the triple strut model in accordance with Panagiotakos and Fardis (1996) and Blasi et al. (2019)

Parameter	Symbol	Equation
Cracking force	f_{cr}	$f_{cr} = \tau_{cr} L_w t_w$
Peak shear strength	f_{max}	$f_{max} = 1.55 f_{cr}$
Residual shear strength	f_{res}	$f_{res} = 0.1 f_{max}$
Relative panel-to-frame stiffness	λ	$\lambda = \sqrt[4]{\frac{E_{inf} t_w \sin(2\theta)}{4 E_c I_c h_w}}$
Width of the equivalent truss section	b_w	$b_w = d_w 0.175 \lambda h_w^{-0.4}$
Initial stiffness of strut	K_1	$K_1 = \frac{G_w t_w L_w}{h_w}$
Post cracking stiffness	K_2	$K_2 = \frac{E_{inf} t_w b_w}{d_w}$
Softening stiffness	K_3	$K_3 = -0.01 K_1$

The peak shear strength (f_{max}) is taken as $1.55 f_{cr}$, as shown in [table 3](#), as is specified for solid clay bricks in Blasi et al. (2019). The residual shear strength (f_{res}) of the struts is taken as $0.1 f_{max}$ based on the experimental results of Panagiotakos and Fardis (1996). The softening stiffness is taken as $-0.02 K_1$ based on the results of experimental work (Crisafulli, 1997).

The total strength and stiffness is distributed between the three struts with the central strut having 50% of the total strength and stiffness and the offset struts having 25% of the total strength and stiffness, as suggested by Chrysostomou (1991), shown in [figure 2b](#). More recent modelling techniques use a single strut while using superposition of vertical and horizontal forces on the RC frame to represent the internal shear forces caused by the masonry infill (Gentile et al, 2019b). The offset distance of the non-central struts (Z_c and Z_b) are calculated using the equation provided by Al-Chaar (2002) and shown in [Figure 2a](#), where θ is the inclination angle of the strut and C_{od} and C_d are the stiffness distribution coefficients for the off-centre and central strut respectively.

The struts are implemented into OpenSees with the truss elements and the uniaxial *pinching4* material. The material is set to have a near-zero tensile stiffness resulting in a compression-only material. It is shown in Blasi et al (2019) that the *pinching4* material has a very similar response to the hysteretic model laid out by Panagiotakos and Fardis (1996). The hysteretic behaviour of *pinching4* are defined by three parameters α , β and γ . These values were taken from the calibration of Blasi et al. (2019) as $\alpha = 0.15$, $\beta = 0.1$ and $\gamma = 0.8$ which were the most representative values based on the comparison of experimental results of RC infilled frames.

An opening factor ($\lambda_{opening}$) is applied to the width of the strut to reduce the stiffness accounting for the openings in the infill panel. Using the structural drawings provided by Pahar trust (2019) it is found that the infill wall opening percentage (α_w) is 25%. The empirical equation given in Equation 4 was first proposed by Al-Chaar et al. (2002) and further verified as being the most accurate equation available in literature using experimental data. Equation 4 is only applicable where the infill wall opening percentage is less than 60%. The opening factor calculated for Pahar trust template school is 0.638 so it is considered to be applicable of the case.

$$\lambda_{opening} = 1 - 1.6\alpha_w + 0.6\alpha_w^2 \quad (4)$$

Figure 2. (a) Visual representation of triple strut model. (b) Backbone curve of individual struts.

2.4 Modal properties

A modal analysis of the four structural models was performed. For the modal analysis, the school can be considered as two structures as the sections of the structure at either side of the seismic gap have no structural interaction. They will subsequently be referred to as *structure 1* and *structure 2*, where structure 1 has the staircase. The modal analysis assumes the stiffness of each material is equal to the initial stiffness (i.e., uncracked). However, as both our material models for concrete and steel exhibit degradation, the modal properties of the structure will change throughout the nonlinear time history analysis. Table 4 shows the first six periods for each of the structures for both the infilled and bare models. The stairs are modelled as a single inclined beam following the modelling approach suggested in Lombardi et al. (2019). The modal participation masses can be found in Table A1 to Table A4 in the appendix for the first six modes of each of the four structures and the modeshape for the first two modes can be seen in figure 3. It can be seen from figure 3 that for both the infilled model without the staircase and the bare model without the staircase the first two mode shapes are translational in the x and y directions, with the modal participation factors above 90% in the respective direction. The third mode shape is rotational about the z axis with a modal participation above 90% for both the infilled and bare model. The mode shapes are not as distinct in the structures with off-centre staircase as the off-set centre of stiffness causes a higher torsional component in the first modes, this can be seen in the participation masses in Table A2 and A4 in the appendix and in figure 3. As is expected due to the additional stiffness caused by the masonry infills the infilled structure has a significantly lower period for the first vibrational modes.

Figure 3. Mode shapes of: (a) without stairs, bare model (b) with stairs, bare model (c) without stairs, infilled model (d) with stairs, infilled model

Table 4. Modal properties of the bare and infilled structures at either side of the seismic gap, Structure 1 is the model with the staircase (w-s), Structure 2 is the model without the staircase.

	Bare		Infilled	
	Structure 1 (w-s)	Structure 2	Structure 1 (w-s)	Structure 2
Mode 1 (s)	0.329	0.352	0.082	0.092
Mode 2 (s)	0.321	0.329	0.066	0.066
Mode 3 (s)	0.189	0.307	0.058	0.064
Mode 4 (s)	0.099	0.103	0.029	0.033
Mode 5 (s)	0.097	0.100	0.024	0.024
Mode 6 (s)	0.042	0.093	0.022	0.023

3. Seismic Hazard and Ground Motion Selection

3.1 PSHA

To complete this study, a seismic hazard map is required for Nepal. Several PSHA studies have been carried out with varying results (Chaulagain et al. 2015a, Ram and Wang, 2013, Stevens et al. 2018, Pokhrel et al. 2019). For this study the PSHA study carried out by Stevens et al. (2018) is used, as it was carried out using an up-to-date characterisation of the Main Himalayan Thrust (MHT) as single thrust based on the evidence from the 2015 earthquake (Elliott et al. 2016, Stevens et al. 2018)

Stevens et al. provide hazard maps for the 2% and 10% probability of exceedance in 50 years. These maps are shown in Figure 4. Stevens et al (2018) also provided the complete hazard curves for five locations shown in Figure 5: Bharratpur, Janakpur, Kathmandu, Pokhara and Rara Lake as indicated in the maps in Figure 4.

Figure 4. Maps generated using data from Stevens et al (2018) (a) PSHA study for 2% probability of exceedance in 50 years (2475-year return period). (b) PSHA study for 10% probability of exceedance (475-year return period) in 50 years.

Figure 5. Hazard curves for five locations across Nepal as provided in Stevens et al. (2018)

For full integration of risk at all locations, the full hazard curve is needed at each geographic point. Data from Figure 4 provides only two points for the curve at each geographic point; thus, a curve must be fitted based on those two known points. For brevity Stevens et al. (2018) provides the full hazard curve for five locations across Nepal as shown in figure 5, these curves and the two PGA values given at each location can be used to interpolate a full hazard curve for each location. Interpolation is often used in areas where data is scarce. At first, linear regression in log-space was used with the two known points (see Figure 6a). The error was calculated by evaluating the difference in the area beneath the hazard curves provided by Stevens et al. (2018) and the fitted line. The error between the linear approximations and the five known curves (Figure 5) was evaluated leading to a mean error of 31.2%. This is not considered an acceptable level of error and a second order approximation was used, as proposed by Vamvatsikos (2014). To allow a unique solution of a 2nd order polynomial, three initial conditions are required. For each geographic location, provided by Stevens et al. (2018) two points on the hazard curve are provided, and for the third point, an earthquake with a PGA of 0.01 g or over has 100% probability of occurrence in 50 years is assumed. This assumption can be seen in Figure 6a and holds true for all hazard curves provided as can be seen in Figure 5. The second-order approximation in log space can be seen in equation 5.

$$H(s) = k_0 \exp(-k_2 \ln^2(s) - k_1 \ln(s)) \quad (5)$$

k_0 , k_1 and k_2 are coefficients that can be found to have unique values for a second-order polynomial that passes through three known points. When this method is used to approximate the hazard curve, it results in a probability of greater than 1 for low PGA events (Figure 6a). Given the first quadratic fit, the third point, that was previously assumed to be a 100% probability of a 0.01 g event in 50 years, is moved to a greater PGA value whereby the quadratic has a maximum at 1. The result can be seen in Figure 6b. Figure 6b shows that this results in a drop at smaller PGA values. This is avoided by setting the probability to a constant level of 1 before the peak. This final method of approximation can be seen in Figure 6c. This is shown to be an accurate approximation with a mean error of 4.1%. This method of approximating the hazard curve can then be applied to all geographic points using the two input values.

Figure 6. Approximation of hazard curve. (a) Linear approximation and second order approximation using third known point as 100% probability of a 0.01 g event in 50 years (b) Second order approximation with the third point set so the maximum is equal to 1. (c) All PGA values lower than the third known point are set equal to 1.

3.2 Ground Motion Selection

To perform a nonlinear dynamic analysis of the buildings presented in section 2, a ground motion record selection was needed. This analysis was carried out with two ground motion sets. The first ground motion set is the far-field ground motion set suggested by FEMA P695 (2009). This ground motion set contains 22 pairs of horizontal motions and is available from the PEER NGA West 2 (2014) ground motion database. Ground motion set 1 (GM 1) is referred to as the "far-field" ground motion set by FEMA P695 and is used for collapse assessment. All these ground motions have a fault rupture distance greater than 10 km and have magnitudes that range from 6.5 M_w to 7.6 M_w . The ground motion catalogue can be found in Table A5 in appendix and the 5% elastic response spectrum is shown in Figure 7a.

Indian standards (IS 1893 Part 1, 2002) do not specify a ground motion selection method but state that NLTHA "shall be based on an appropriate ground motion and shall be performed using accepted principles of dynamics". Many other seismic codes use a ground motion selection method for NLTHA that is based on seismic hazard at the site of the structure (ASCE 7 10, 2010, FEMA P-58-1, 2018). As the template school is implemented across an area of varying seismic hazard, it is not possible to apply these methods for this analysis. FEMA P695 presents 22 far-field ground motion sets and 28 near-field ground motion sets. These ground motions are independent of the site and are considered efficient to assess record-to-record variability. The far-field ground motion set was used as these are the ground motions recommended by FEMA P695 (2009) for collapse assessment. FEMA P695 (2009) states that near fault motions should not be used for collapse assessment due to "unresolved issues concerning the characterization of near-fault hazard and ground motion effects". These ground motion effects include forward directivity effect, fling-step effect, basin effects, surface wave effect and soft soil effect (Chen et al, 2019).

The second ground motion set was formed of five pairs of horizontal recordings from the Gorkha 2015 earthquake. Four of these recordings were made available by the Faculty of Engineering, Hokkaido University, Japan (Rupakhety et al. 2017) with the final one being made available by USGS (2016). The Gorkha 2015 earthquake which had a moment magnitude of 7.8 M_w , had an epicentre 80 km west-northwestst of the Centre of Kathmandu and destroyed 6000-8200 schools (Giordano et al, 2020). Figure 7b shows that all the ground motions with the exception of KTP have amplification in the high period range, this is due to the sediments and the topography of the Kathmandu Valley (Asimaki et al., 2017; Gilder et al. 2020). The KTP station does not experience this amplification as it positioned on a rock outcrop near the western edge of the basin. This amplification resulted in relatively low damage in low-rise structures and high levels of non-structural damage to tall buildings (Brando et al., 2017). The ground motion catalogue for GM 2 can be found in Table A6 in the appendix.

Two ground motion sets are used as ground motion set 1 offers exposure to a wide range of far-field ground motions with a moment magnitude ranging from 6.5 to 7.6 and a PGA ranging from 0.21 g to 0.82 g. The Gorkha ground motion set allows us to highlight how the Gorkha earthquake may have affected these school buildings in 2015, and this can further be used as a point of comparison with other local schools. It can be seen from Figure 7 that the response spectra have different shapes and different intensities (i.e. GM1 is significantly higher with respect to GM2). In particular, the variation in PGA is greater for GM 1 (COV = 0.42) compared to GM 2 (COV = 0.25) and the variation in terms of spectral acceleration at the first fundamental period of the bare structure without the staircase ($S_a(T_1)$), assuming $T = 0.352s$, is also greater in GM 1 (COV = 0.43) compared to GM 2 (COV = 0.28). The first period of the infilled

structure is very low ($T=0.082s$) so the spectral acceleration at the first period can be considered equal to the PGA. This differences between the two ground motion sets are expected as GM1 accounts for the inter-event variability that is not accounted for in GM2 (i.e., all ground motion records from the same event).

Figure 7. 5% damped elastic response spectrum for (a) "far-field" ground motion set by FEMA P695 (b) Gorkha 2015 ground motions

Parts of Kathmandu could potentially experience earthquakes up to Mw 9.2 (Stevens and Avouac, 2016) which is higher than either of the ground motion sets. Jalayer et al (2017) set out a number of criteria for ground motion selection to meet, for the fragility curves to be considered reliable. It is shown in section 4.2 that these criteria are met and hence the fragility curve can be reliable at extreme IM measures.

4. Structural assessment

4.1 Non-linear time history analysis

Non-linear time history analysis is carried out using the model outlined in section 2 and the ground motion selection outlined in section 3. Rayleigh damping is implemented using a critical damping ratio of 2% (Dolšek et al, 2001) and the first and third fundamental periods to evaluate the mass proportional damping coefficient and stiffness proportional damping coefficient (Chopra, 2012). P-Delta Coordinate Transformation is used as to account for second order effects caused by the lateral loads.

Figure 8 shows the shear demand for the 1994 Northridge earthquake and the shear capacity of a column with section c1 (shown in figure A1 in the Appendix) according to the Indian standard and Eurocode using equations 3 and equation 2 respectively from section 2.1. As the Eurocode 8 Part 3 (EN 1998-3, 2005) calculation is a function of the length of the member the shear check is carried out for the squat and slender column as defined by the infill structural modelling. The squat column is defined as the column from the floor to the off-centre strut and the slender column is defined as the columns from the lower to upper off-centre column, as can be seen in figure 8. Northridge 1994 earthquake (RSN 953) is used as it has the highest peak shear demand of all the ground motions and section C1 is used as it has the lowest amount of shear reinforcement. This further shows that the shear capacity is not exceeded.

Figure 8. Shear demand for Northridge 1994 earthquake (RSN 953) from GM 1 with shear capacity of section C1 according the Indian Standard (IS 456, 1978) and Eurocode 8 Part 3 (EN 1998-3, 2005) (a) Slender column (b) Squat column. Shear demand (in red) shown for structure 1 infilled model.

4.2 Fragility Analysis

A fragility curve is defined as the relationship between the conditional probability of a structure exceeding a certain limit state and a seismic load intensity. In this study two limit states will be considered; damage limitation (DL) and life safety (LS).

The intensity measure (IM) is used to characterise the ground shaking related to seismic events. In literature commonly used IMs are PGA and spectral acceleration at the first mode ($S_a(T_1)$) (Tothong and Luco, 2007). An efficient IM is one that, when unchanged, gives low levels of variability in the given demand capacity ratio (DCR) (i.e., a low level of standard deviation in log space). The demand capacity ratio (DCR) is defined as a ratio of an engineering demand parameter, such as interstorey drift or chord rotation, to the critical demand parameter, such as a code specified interstorey drift limit. The dispersion can be calculated using Equation 6 where a and b are two constants for that are found in the linear regression for that specific fragility function (Bayat et al. 2015), Y is the DCR and N is the number of samples.

$$\beta_{Y|IM} \cong \sqrt{\frac{\sum (\ln(Y_i) - \ln(b \ln(IM) + \ln(a)))^2}{N - 2}} \quad (6)$$

A sufficient intensity measure is one that results in the $P[DCR|IM]$ to be unaffected by other ground motion properties such as magnitude. The more sufficient an IM is, the more results in the DCR are mainly independent of the ground motion selection (e.g., Ebrahimian et al., 2014).

Shome et al. (1998) demonstrated that the 5% damped $S_a(T_1)$ is a more efficient IM than the PGA of the ground motion. More recently, it has been shown that AvgSA is a more efficient IM than $S_a(T_1)$ (Bayat et al. 2015, Kohrangiet al. 2017). AvgSA is the geometric mean of the spectral acceleration values between periods $c_1 T_1$ and $c_n T_1$ where c_1 and c_n are lower and upper bound factors. In previous work it has been shown 0.2 T_1 and 3.0 T_1 are appropriate choices for the upper and lower bound with a logarithmic spacing between, as this has shown strong sufficiency and efficiency (Eads et al. 2015).

$$Sa_{avg}(c_1T_1, \dots, c_nT_1) = \left(\prod_{i=1}^N Sa(c_iT_1) \right)^{\frac{1}{N}} \quad (7)$$

This study uses a modified version of AvgSA where the upper limit is equal to $3.0T_{1,bare}$ where $T_{1,bare}$ is the first period of the bare structure. The Lower limit will be set to $0.2T_{1,infilled}$ where $T_{1,infilled}$ is the first period of the infilled structure. This intensity measure which accounts for the dynamic response of all structures considered (bare and infilled, with and without stairs) is referred to as $AvgSA_{mod}$. This allows the suitable comparison of the fragility curves of both structures as they will have a common IM. The two sets of records discussed in section 3.2 are employed to perform a cloud analysis. Their suitability as seismic input for cloud analysis is checked according to the general criteria set out in Jalayer et al. (2017) and provided in the following:

1. The record selection should have a large dispersion of Sa (or the relative intensity measure) as to reduce the level of standard error in the regression slope.
2. At least 30% of the nonlinear time history analyses should cause the demand capacity ratio (DCR) to exceed 1.
3. No more the 10% of the ground motion selection should be from the same event.

The ground motion selection presented in section 3.2 is compliant with the first of the criteria set out by Jalayer et al. (2017). The first of the Jalayer criteria is satisfied as $AvgSA_{mod}$ ranges from 0.14 g to 0.94 g with a standard deviation of 0.216 g. The second of the Jalayer criteria are also satisfied with the bare model having 78% of records yielding a DCR greater than 1 and the infilled model has 40% of records giving a DCR greater than 1. The third Jalayer criterion is satisfied for GM1; however, it is not met GM2 as all recordings are from the Gorkha 2015 earthquake. If the two sets are considered together still, the condition is not met as 18% of the records are from Gorkha earthquake. The linear regression is also completed without the Gorkha ground motions, as shown in Figure 9, to assess whether using a single event for five of the recordings has a large influence on the results. It can be seen that the linear regression results are very closely aligned.

Cloud analysis uses a linear regression-based probabilistic model with the results of nonlinear dynamic analysis to find a cumulative probability function for the occurrence of exceeding a critical demand parameter (CDP). A suitable critical demand parameter is first established for each of the limit states. In the analysis of RC structures without masonry infill a CDP of 0.5% interstorey drift ratio (IDR) for DL is used as set out in Eurocode 8 for buildings which have brittle non-structural elements (EN 1998-1, 2004). The DL criteria for the infilled model is defined as the IDR when all infills have exceeded peak load. This typically occurs between 0.1% and 0.2% (Ricci et al, 2016). The peak load of each infill varies as it is a function of the material properties and geometry of that specific masonry infill. The largest IDR at peak strength, equal to 0.19%, is assumed as capacity limit herein. This is used as the damage limitation criteria for the infilled model. The life safety limit state is 2% for both the infilled model and the bare model (ASCE/SEI 41-17, 2017). The demand capacity ratio (DCR) is

defined as the ratio of the peak demand to the structures at either side of the seismic gap, in the results of a dynamic analysis, to the CDP. This study assesses the fragility using the cloud method. This involves using linear regression in the logarithmic space of the IM against the DCR. This gives the statistical constants a and b shown in Equation 8, where Y is the DCR. This can then be used in combination with the standard deviation given in Equation 9 to find the fragility curve using Equation 10 (Jalayer et al. 2014).

$$E[\log(Y) | IM] = \log \eta_{Y|IM} = \log a + b \log IM \quad (8)$$

$$\sigma_{\log(Y)|IM} = \beta_{Y|IM} = \sqrt{\frac{\sum_{i=1}^N (\log Y_i - \log \eta_{Y|IM_i})^2}{N - 2}} \quad (9)$$

$$P(Y > 1 | IM) = P(\log Y > 0 | IM) = 1 - \Phi\left(\frac{-\log \eta_{Y|IM}}{\sigma_{\log(Y)|IM}}\right) = \left(\frac{\log \eta_{Y|IM}}{\sigma_{\log(Y)|IM}}\right) \quad (10)$$

The cloud analysis was carried out using both ground motion sets (presented in section 3.2) as a single one with both the bare model and the infilled model. The $AvgSA_{mod}$ was used as an IM as to allow comparison between the two structural models. Figure 9 shows the linear regression in logarithmic space for the two models while also showing the level of dispersion by showing the plus and minus one logarithmic standard deviation (calculated according to equation 9). Figure 9 also shows the linear regression for the FEMA P695 ground motion set and it shows that the results are not significantly effected by the Gorkha ground motion set where five recordings are from one earthquake.

Figure 10 shows the *robust* fragility curves and the plus and minus one standard deviation curves according to Jalayer et al. (2015), for both the bare and the infilled model. The infilled structure has a lower probability of exceeding LS or DL at any level of $AvgSA_{mod}$. The median (η) and logarithmic standard deviation (β) parameters of the four fragilities presented in Figure 10 are summarised in Table 5.

Figure 9. Linear regression in logarithmic space for DCR at DL and LS vs $AvgSA_{mod}$ a) bare model, DL state b) bare model, LS c) infilled model, DL d) infilled model, LS

Figure 10. Fragility curves for the DL and LS for both the infilled and the bare model using an intensity measure of $AvgSA_{mod}$

Due to the PSHA study described in section 3.1, providing the expected PGA values across Nepal, a fragility analysis must be carried out using PGA data. Although using PGA as an intensity measure has been shown to have lower sufficiency and efficiency than AvgSA (Eads et al. 2015) it allows the integration of the PSHA study and the fragility assessment. The DCR remains unchanged. Figure 11 shows the linear regression with figure 12 showing the fragility curves. Figure 9 and Figure 11 show that AvgSA_{mod} is a more efficient with the β value being 17% lower for the cloud analysis where AvgSA_{mod} is used relative to PGA. The η and β parameters of the four fragilities presented in Figure 12 are also summarised in Table 5.

Figure 11. Linear regression in logarithmic space for the DL and LS vs the peak ground acceleration a) bare model, DL b) bare model, LS c) infilled model, DL d) infilled model, LS

Figure 12. Fragility curves for the DL and LS for both the infilled and the bare model using an intensity measure of PGA

Table 5. Parameters of the fragility curves presented in Figure 10 and Figure 12

IM	Model	Limit State	η [g]	β
AvgSA _{mod}	Bare	DL	0.263	0.228
		LS	0.743	0.226
	Infilled	DL	0.371	0.188
		LS	1.067	0.193
PGA	Bare	DL	0.178	0.249
		LS	0.465	0.250
	Infilled	DL	0.245	0.205
		LS	0.644	0.211

4.1 Damage Distribution

Using the fragility curves derived previously and the PGA values at each location provided by Stevens et al. (2018), the damage distribution of the template school at each location across Nepal can be found. The damage distribution is defined as the relative probability of the structure exceeding a given limit state given a specific level of ground shaking, in this case PGA. Figure 13a shows that for the 2% in 50 years earthquake the bare model exceeds LS, with over 99% probability, in Rara Lake, Pokhara, Bharatpur and Kathmandu, with Janakpur having a 40% probability of exceeding DL and 60% probability of exceeding LS. This demonstrates how the varying seismic hazard causes results in the template school having a varying level of seismic risk. Figure 13b shows that for the 10% in 50 years earthquake the bare model

at Rara Lake, Pokhara, Bharatpur and Kathmandu either exceeds LS or DL, with Janakpur having a 30% probability of experiencing no damage. The damage distribution for the infilled model of the template school can be seen in Figure 14. Specifically, Figure 14a shows that for the 2% in 50 years earthquake that Rara Lake, Pokhara, Bharatpur and Kathmandu are expected to exceed LS and Janakpur is expected to either exceed DL with a 11% chance of exceeding LS. Figure 14b shows that for the 10% earthquake in 50 years Rara Lake, Pokhara, Bharatpur and Kathmandu all most likely exceed DL and Janakpur has a 82% probability of experiencing no damage.

Figure 13. Damage distribution of bare model of the template schools at five location across Nepal (a) for the 2% probability in 50-year (2475 year return) earthquake (b) for the 10% probability in 50-year (475 year return) earthquake

Figure 14. Damage distribution of infilled model of the template schools at five location across Nepal (a) for the 2% probability in 50-year (2475 year return) earthquake (b) for the 10% probability in 50-year (475 year return) earthquake

5. Risk assessment

Using the hazard curve at each point across Nepal and the fragility curves for each limit state the annual probability of exceeding each limit state can be found using Equation 11 where P_{LS} is the annual probability of exceeding a given limit state, $H(s)$ is the hazard curve, and $P_{Is/s}$ is the fragility curve. Numerical integration can be carried out by discretising the hazard curve into equal parts which are assumed to be piecewise linear; this is shown in Equation 12, where $P_{F/Scgi}$ is the probability of exceeding the given limit state at the mid-point of s_i and s_{i+1} . The results of this integration are represented in the maps shown in Figure 15 to Figure 18.

$$P_{LS} = \int_0^{\infty} P_{Is/s} \left(\frac{dH(s)}{ds} \right) ds \quad (11)$$

$$P_{LS} = \sum_{i=1}^n [H(s_i) - H(s_{i+1})] P_{ls/scgi} \quad (12)$$

Figure 15. Annual probability of exceeding DL for the bare model

Figure 16. Annual probability of exceeding LS for the bare model

Figure 15 shows that the annual probability of exceeding DL is in the range of 0.010 to 0.025 for the majority of Nepal. As this structure is a school, it would be considered to have an importance class III according to Eurocode 8. For buildings of importance class III for DL, the annual probability of exceedance should be lower than 0.030 according to Eurocode standards (Pinto et al. 2014) as shown in table 6. The annual probability of exceeding DL is lower than this for all locations across Nepal. Figure 15 also shows that the risk level to the template school varies significantly based on its

location. If the template school was located on the northern border, it would have an annual probability of exceeding DL of 0.003 which is almost ten times lower than for Bharatpur, where the annual probability of exceeding DL is 0.025. **Figure 16** shows that the annual probability of the bare model exceeding LS ranges from 0 to 0.0050 across Nepal. The threshold for LS is set to 0.0032 (Pinto et al. 2014).

Figure 17. Annual probability of exceeding DL for the infilled model

Figure 18. Annual probability of exceeding LS for the Infilled model

Figure 17 shows the annual probability of the infilled model exceeding DL ranges from 0 to 0.018. This is below the maximum allowable level of 0.03 (Pinto et al. 2014). It can be seen the annual probability of exceeding DL is significantly lower for the infilled model relative to the bare model. This is due not only to the structural contribution of the infills but also to the change of DL threshold recommended in structural engineering practice. **Figure 18** shows that the annual probability of the infilled model exceeding LS ranges from 0 to 0.0025 which is below the threshold of 0.0032 (Pinto

et al. 2014). This emphasises how the structural contribution of infills push down the risk below the critical thresholds set up according to European standards and can make the difference between a school being compliant or non compliant according to risk-based requirements of codes. The impact of this observation has implications on the realisation of masonry infills that, especially, in high-hazard areas of the country, need to be constructed with good care and a high level of quality control on mortar and bricks.

Table 5. Maximum annual probability of exceedance of DL and LS exceedance for the two models and corresponding thresholds (Pinto et al. 2014).

Limit State	Bare	Infilled	Threshold
Damage Limitation	0.025	0.018	0.030
Life Safety	0.0050	0.0025	0.0032

6. Conclusion

This study developed a nonlinear finite element model of a template school in Nepal and carried out nonlinear time history analysis with two ground motion sets. The focus of this study was to assess how a template school design affected the seismic risk to schools in Nepal. The study did not assess other potential risks to the structures, including construction quality, landslides and liquefaction as cascading hazards for structures induced by earthquakes. The results of these analyses were then used to carry out a cloud analysis where a modified version of average spectral acceleration was used as it was a more efficient intensity measure relative to PGA and it incorporated the structural response of both models. The cloud analysis was then carried out using PGA values to allow integration with seismic hazard curves. Hazard curves were then developed from data provided by Stevens et al. (2018) for all points across Nepal through a simplified interpolation procedure. Finally, the fragility curves and hazard curves were integrated to find the annual probability of exceeding each limit state.

The fragility analysis was carried out for an FE model without masonry infill and a model with masonry infill. The results showed that an earthquake with a $AvgSA_{mod}$ of 0.6 g would give a 50% probability of exceeding DL and that an earthquake with a $AvgSA_{mod}$ of 1.1 g would give a 50% probability of exceeding LS, for the bare model. The infilled model had a 50% probability of exceeding DL for an earthquake with an $AvgSA_{mod}$ of 0.37 g and had a 50% chance of exceeding LS with an earthquake with a $AvgSA_{mod}$ of 1.07 g. The results of the cloud analysis also demonstrated that none of the the Gorkha 2015 ground motion recordings caused the template school to exceed LS or DL.

The cloud analysis was then carried out with an intensity measure of PGA as this allowed the fragility curves to be integrated with the hazard curves. Hazard curves were extrapolated from data provided by Stevens et al. (2018) and were then integrated with the fragility curves to find an annual probability of exceeding each limit state. The bare structure has a maximum annual probability of exceeding DL of 0.025 and a maximum annual probability of exceeding LS of 0.0050. The infilled structure has a maximum annual probability of exceeding DL of 0.018 and a maximum annual probability of exceeding LS of 0.0025. With the exception of the bare model for LS, all of these annual probabilities of exceedance values are lower than the threshold values set out according to European regulations (Pinto et al. 2014). This is a very

encouraging result for a template school as it complies with risk thresholds levels for school buildings in Europe setting a very high standard in terms of accepted societal risk.

This study shows that using a single design for a structure across an area of varying seismic hazard results in a highly varied seismic risk. It was shown that the varying hazard resulted in the template school having a significantly higher level of risk around Pokhara and Kathmandu than along the northern border. For the bare model, the risk threshold for a school building was exceeded however this was not the case on the infilled model, showing that accounting for the structural contribution of infills can be very important to have an accurate picture of the implicit risk within a specific template design across a large area. On the other hand, this also emphasises how quality controls on template schools have to include also non-structural elements (i.e., mortar and bricks) to make sure that the additional seismic performances that they can provide are effectively guaranteed. The properties of infills have been assumed from regional South-Asian studies to be realistic and avoid overestimations. One critical aspect is for example the cement to sand ratio that needs specific control during the construction to make sure that infills structural contribution can be relied upon in case of an earthquake event. A further action to contain losses during the life of the school template could be to have implement a gap between the non-structural infills and the reinforced concrete elements to decrease the damage for frequent earthquakes, increase the drift capacity for damage mitigation and reduce potential repair costs.

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Appendix

Figure A1. Beam and column section properties of Pahar Trust template school (Pahar Trust, 2019)

Table A1. Modal participation masses of the bare structure with no staircase

Mode	MX [%]	MY [%]
1	0.000	90.951
2	92.402	0.000
3	0.000	0.463
4	0.000	8.499
5	7.598	0.000
6	0.000	0.087

Table A2. Modal participation masses of the bare structure with off-centre staircase

Mode	MX [%]	MY [%]
1	60.622	16.067
2	31.484	30.076
3	0.007	38.892
4	7.853	0.002
5	0.033	0.352
6	0.001	14.611

Table A3. Modal participation masses of the infilled structure with no staircase

Mode	MX [%]	MY [%]
1	0.000	92.624
2	93.631	0.005
3	0.373	0.569
4	0.000	6.740
5	5.413	0.007
6	0.582	0.055

Table A4. Modal participation masses of the infilled structure with off-centre staircase

Mode	MX [%]	MY [%]
1	0.501	81.080
2	92.223	0.068
3	1.346	14.049
4	0.015	4.603
5	5.172	0.006
6	0.742	0.196

Table A5. Main characteristics of FEMA P695 far-field earthquakes

Earthquake	Year	Epicenter Distance (km)	PGA (g)	Magnitude	Duration (s)
Northridge	1994	13.3	0.52	6.7	29.99
Northridge	1994	26.5	0.48	6.7	19.99
Duzce, Turkey	1999	41.3	0.82	7.1	55.90
Hector Mine	1999	26.5	0.34	7.1	45.31
Imperial Valley	1979	33.7	0.35	6.5	99.92
Imperial Valley	1979	29.4	0.38	6.5	39.035
Kobe, Japan	1995	8.7	0.51	6.9	40.96
Kobe, Japan	1995	46	0.24	6.9	40.96
Kocaeli, Turkey	1999	98.2	0.36	7.5	27.085
Kocaeli, Turkey	1999	53.7	0.22	7.5	30.00
Landers	1992	86	0.24	7.3	44.00
Landers	1992	82.1	0.42	7.3	27.965
Loma Prieta	1989	9.8	0.53	6.9	39.955
Loma Prieta	1989	31.4	0.56	6.9	39.945
Manjil, Iran	1990	40.4	0.51	7.4	53.52
Superstition Hills	1987	35.8	0.36	6.5	40.00
Superstition Hills	1987	11.2	0.45	6.5	22.30
Cape Mendocino	1992	22.7	0.55	7.0	36.00
Chi-Chi, Taiwan	1999	32	0.44	7.6	90.00
Chi-Chi, Taiwan	1999	77.5	0.51	7.6	90.00
San Fernando	1971	39.5	0.21	6.6	28.00
Friuli, Italy	1976	20.2	0.35	6.5	36.345

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Table A6. Main Characteristics of the five ground motion recordings from Kathmandu Valley of the Gorkha 2015 earthquake

	Orientation	Latitude	Longitude	Location
KATNP	000, 090	27.71307	85.3161	Kanti Path
KTP	230, 320	27.68182	85.27261	Kirtipur Municipality Office
THM	090, 180	27.68072	85.3772	University Grant Commission Office, Bhaktapur
TVU	000, 090	27.68145	85.28821	Central Department of Geology
PTN	090, 360	27.68082	85.31897	Engineering College, Pulchowk

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