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INVESTIGATIONS ON THE RESTORATION AND SEISMIC ENHANCEMENT OPTIONS FOR THE JAISEDEWAL TEMPLE AFTER THE GORKHA EARTHQUAKE IN NEPAL

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Abstract: *During the strong Gorkha Earthquake that hit the Kathmandu Valley, Nepal, on April 25, 2015, the high vulnerability of the historical monuments of the region was revealed. In particular, the World Heritage brick masonry multi-tiered Nepalese Pagodas suffered extensive damages and numerous temples collapsed. Although a significant amount of work has been done to understand the seismic vulnerability of such historic temples, little work has been done on understanding the effectiveness of low-cost seismic retrofit and repair options. The aim of this paper is to investigate the restoration and seismic enhancement options for the Jaisedewal temple in Nepal which collapsed after the Gorkha Earthquake. A three-dimensional numerical model of the Jaisedewal Temple has been developed. Use was made of the finite element method of analysis which was implemented in the commercial code ABAQUS. The developed finite element model was based on the macro-modelling approach and aids in understanding of the seismic response of this structure as well as to give insights to the performance of the seismic enhancement options to safeguard Nepalese temples from future earthquake events.*

Introduction

On April 25, 2015, the Gorkha Earthquake struck close to the city of Kathmandu in Nepal. It is estimated that during this earthquake approximately 9,000 people were killed, many thousands were injured, 2.8 million people were displaced, and more than half a million structures were either damaged or collapsed (Sharma et al., 2018). On top of that, dozens of Kathmandu heritage buildings suffered severe damages while others partially or totally collapsed. The financial losses after the Gorkha Earthquake have been estimated to be around 7 billion US dollars. A side-effect of this earthquake was the severe drop in the influx numbers of tourists due to the damages inflicted to the monuments of the Kathmandu region, entailing a significant drop in the income of the country. The significance of the conservation of the monuments of a country for the economic growth highlights the need for future-proofing actions in terms of restoration, rehabilitation and strengthening.

The excessive damage inflicted on the cultural heritage of Nepal urged researchers to investigate further the monuments that survived from the Gorkha earthquake and assist in the documentation of existing damages. Shrestha et al. (2017) utilized digital recording and non-destructive techniques to assess the structural condition of historical temples, while Dhonju et al. (2017) developed a geo-crowdsourcing and web-mapping platform incorporating both web and mobile based data collection with the intention to invite community participation. This end-to-end system

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creates an online virtual community encouraging public engagement and raising awareness about heritage ownership. It also provides valuable sources for cultural heritage exploitation, management, education, and monitoring over time.

Several researchers collected detailed assessment data in order to characterize the material properties and the overall state of the temples in the aftermath of the Gorkha earthquake. Thus, updated information on the seismic vulnerability of the temples was acquired, essential for any potential rehabilitation solutions.

Jaishi et al. (2003) obtained ambient vibration measurements on three Nepalese temples for the calibration of the corresponding numerical models concluding in an estimation of modulus of elasticity for the mud-mortar in the brick masonry multi-tiered temples in the range of 400 to 800 MPa and of the damping ratio between 1% and 6%. Additional numerical studies were also carried out on seven more Nepalese temples finding that the first natural period of the temples is less than 0.6 s.

UNESCO commissioned the development of a finite-element model in order to get insight into the seismic vulnerability of the Gopinath temple in Hanuman Dhoka's Durbar Square (Arce et al., 2018). Though initially a value of 800 MPa was used for the modulus of elasticity for the mud-mortar, the calibration of the model based on the ambient vibration measurements showed that a value of 150 MPa, well below the one originally used, is more realistic, and the fundamental period of the structure was found to be equal to 0.5 s.

Despite the considerable amount of work related to understanding the seismic vulnerability of such historic temples in Nepal, little work has been done on the effectiveness of low-cost repair and strengthening options. The aim of this paper is to study the seismic response and investigate seismic enhancement options, such as strengthening of floors and timber columns, improvement of connections, replacement of existing materials with higher strength materials etc, for ancient temples in Nepal. For that purpose the three-tiered Jaisedewal Temple, a typical example of the Nepalese pagoda architectural style, was chosen as a case study. A three-dimensional finite-element model of the Jaisedewal Temple has been developed in the commercial code ABAQUS based on the macro-modelling approach and subjected to nonlinear analysis.

Characteristics of the Nepalese Pagodas

The performance of heritage structures during the Nepal Earthquake of April 25, 2015, was reported in KC et al. (2017). One of the observations made was that the tiered temples founded on a wide plinth base generally performed better that the temples on a narrow plinth base. The plinth is usually of low-quality workmanship and limited cohesion, thus of inferior performance than the supported temple. According to Rits-DMUCH (2012), the Nepalese temples were placed on plinths in order to improve their seismic response against the wave amplification and to avoid the resonance with the soil layer. Although the structure could have been considered as an integral system consisting of the upper structure and the massive plinth base, the contribution of the latter has been neglected in previous studies (Shakya et al., 2013; Arce et al., 2018) but was deemed of paramount importance to include it in the numerical model developed in this work.

During the Gorkha Earthquake the pathologies of the Nepalese temples were revealed. The main cause of damage to these unreinforced masonry (URM) buildings is due to: (a) strength deterioration of the construction materials; (b) lack of regular maintenance; (c) lack of proper connections in the corners; (d) excessive weight in the upper part of the building and (e) discontinuous loading paths along the height of the structure.

Different interventions have been suggested for upgrading the seismic behaviour of these temples (Nienhuys, 2003). One of these is to reduce the seismic demand by reducing the mass of the structure. For example, new materials can be used for the construction of the roof system and the soil layer placed below the tiles (Figure 3b) can be removed in order to obtain a lighter structure. In addition, after discussion with local engineers and architects working for the restoration of the temples in Nepal it was highlighted that wooden connections along the temples could be reinforced and additional timber elements could be installed in areas of high concentration of stresses. Wooden diaphragms can also be stiffened since this can be a low-interference intervention which could improve the overall seismic behaviour of the structure. The foundation of the wooden pillars to the plinth base can be reinforced in order to avoid the premature rupture of the extremely vulnerable pin connections. In particular, the connection of the wooden pillars to the foundation below has been proved to be quite susceptible to seismic loading. The pillars at both edges end up to a pretty thin cross-section called pin (Figure 1a). The rupture of the pin of the main wooden columns was a common observation during the Gorkha Earthquake, which confirms its vulnerability (Figure 1b). Such local failure led to the loss of support of the overlying peripheral wall which could be the main reason of collapse of some temples.

Finally, the performance of the temples to earthquake loads could be affected since the top masonry walls of the temples does not directly sit on the underlying masonry core. This type of connection creates a discontinuous loading path. There were numerous examples of temples which although survived the earthquake, their top part collapsed (Figure 2).

Jaisedewal temple

The focus of this study is the assessment of the seismic response of the Jaisedewal temple. The structure is a typical example of the Nepalese architecture in the Kathmandu Valley. The temple atJaisedewal is a three-tiered temple based on a massive brick plinth, the dimensions of which at the bottom are 19.8 m × 19.8 m and at the top 7.6 m × 7.6 m. The height of the plinth is 6.1 m while the height of the upper structure is 16.6 m. The whole structure is characterized by a box-type configuration. In Figure 3 an architectural cross-section of the structure as well as a detail of the roof system and its elements (tiles, soil layer and timber elements) are presented. As seen in the cross-section (Figure 3a), the structural system of the temple consists of a thick core of masonry walls in addition to a peripheral external wall sitting on wooden pillars. At the top of the temple there is an internal core with thinner cross-section, which does not sit on the main core of the structure but is simply connected with some crossing timber elements.

Figure 1: (a) Detail of the connection of the wooden pillars with the rest structural elements (Arce et al., 2018), (b) typical failure of the vertical timber elements supporting the peripheral walls, and (c) strengthening of timber pins with stainless steel warping (Arce et al., 2018).

Figure 2: Different cases of collapse of the top part of the Nepalese temples during the Gorkha earthquake that hit Nepal in 2015, (a) (Endo and Hanazato, 2019), (b) and (c) (Shrestha, 2015).

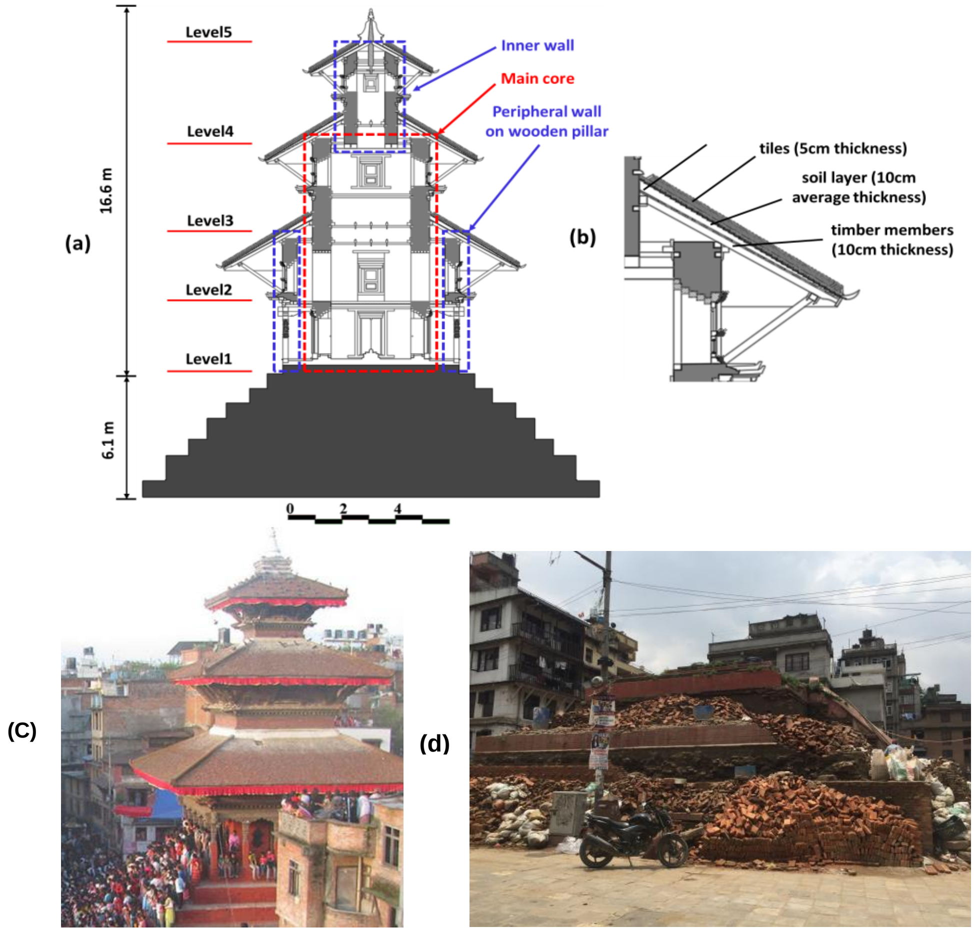

Figure 3: Jasidewal temple: (a) cross section (illustration of foundation is indicative); (b) detail of the roof system; (c) the temple before the earthquake; and (d) the temple after the Gorkha earthquake.

Seismic records

On April 25, 2015, the Gorkha earthquake (7.8-magnitude) hit the Kathmandu Valley. This tremendous ground motion was followed by a 7.3-magnitude earthquake on May 12, 2015. The epicentre was near the Gorkha region, 80 km from the Kathmandu Valley. Seismic records came from the rock site (KTP station) as well as from the sedimentary sites TVU, PTN, THM, and KATNP (Takai et al., 2016). The recorded waveforms produced significant spectral accelerations in the range of short periods (up to 1 s) but also for longer periods (i.e. up to 6 s). The largest maximum peak ground acceleration (PGA=0.25g) was recorded at the EW component of the KTP station. The response spectra of the main records of the Gorkha Earthquake are presented in Figure 4a.

Another destructive characteristic of the earthquake was its long duration containing multiple pulses (Figure 4b). As will be shown below, many vulnerable Nepalese Pagodas reached their capacity in the very first pulses of the excitation, while the next pulses pushed the structure even further making the total collapse inevitable.

The record coming from the EW component of the TVU station (Takai et al., 2016) was chosen for the time-history analyses performed in this work (Figure 4b). The distance between the TVU station and the Jasidewal temple is 3km. The chosen record has a PGA of 0.23 g and more importantly, it produces spectral accelerations of up to about 0.8 g at the period of 0.7 s, which coincides with the fundamental period of the case-study structure.

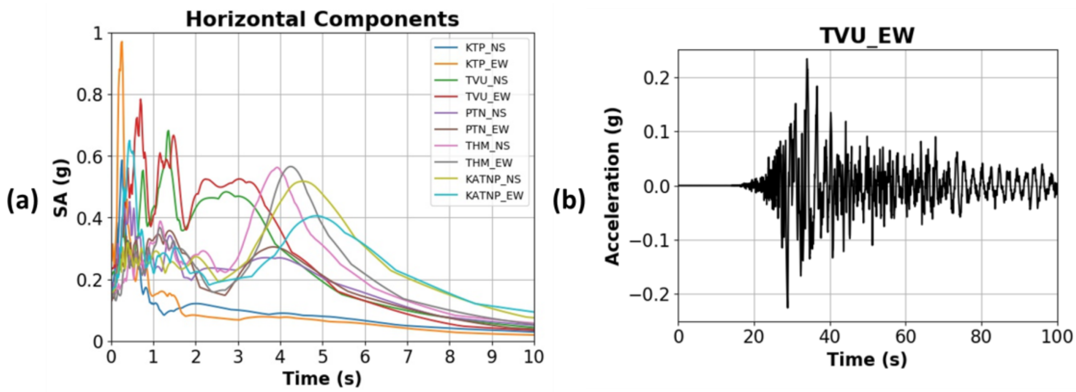

Figure 4: (a) The response spectra corresponding to the horizontal components of the main records of the Gorkha earthquake Nepal (April 25, 2015), and (b) the record used for the time-history analysis.

Development of the Numerical Model

A three-dimensional finite-element model (ABAQUS, 2014) based on the macro-modelling approach has been developed to analyse the seismic response of the Jaisedewal Temple. The structural members of the model include both timber and brick wall elements. The timber elements were modelled as linear elastic, while the masonry walls were modelled with nonlinear elements based on the Concrete Damage Plasticity constitutive law, already implemented in ABAQUS.

For the case of the brick walls, the modulus of elasticity was taken equal to 150 MPa. Moreover, the value of compressive and tensile strength for the brick walls were chosen as 1 and 0.05 MPa respectively (Arce et al., 2018). The ability of the masonry walls to carry tensile forces is rather limited. Thus, it is reasonable to assume the tensile strength as only 5% of the compressive strength. The material properties used for the development of the model are shown in Table 1. The density of the brick walls and the timber elements were selected as 1,800 kg/m^3 and 800 kg/m^3 respectively as proposed by Parajuli (2012) and Arce et al. (2018). The mass of the roof system (tiles, soil layer and timber members) was calculated in accordance to the suggestions made by Nienhuys (2003). In particular, a mass of 1,500 kg was assigned at the top of the structure to account for the decorative members at that level (Figure 3). The density of the tiles and the soil layer beneath was assumed 1,400 kg/m^3.

Two models were developed. The first one stands for the “initial condition” of the Jaisedewal Temple prior to the earthquake and the other one corresponds to the “retrofitted” structure. For the “initial condition” the wooden slabs were modelled as flexible diaphragms. The pin connection of the wooden pillars was taken into consideration in the model. The masses of the three tiers were incorporated in the model as extra masses at the top of the masonry walls where the roofs were mounted to. The timber pegs that connect the peripheral walls with the main masonry core were also included in the numerical simulation. The inner walls were connected with the outer walls with timber beams with long spacing among them. Moreover, the timber floors were modelled as flexible providing limited diaphragmatic action to the structure. The vertical timber elements connecting the peripheral outer wall to the foundation level were considered to remain elastic. Due to the symmetry of the structure loading was applied only along the horizontal (X-X’) direction (Figure 5).

For the “retrofitted” model interventions of low interference were implemented: the wooden slabs were stiffened in order to provide a diaphragmatic action, wooden planks were added to connect the internal core with the peripheral wall with the aim to exclude the concentration of stresses around the connections of the existing timber pegs, and the wooden poles at the end of the timber columns were wrapped with stainless steel to improve their connection with the foundation (Figure 1c).

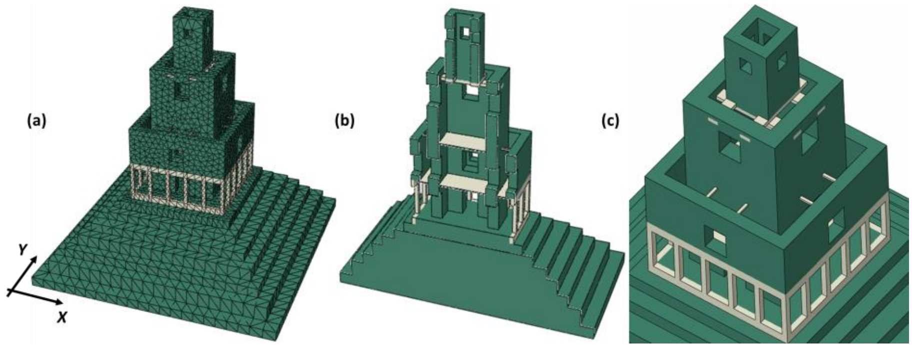

Figure 5: The developed 3D numerical model: (a) meshing, (b) cross-section of the model, and (c) detail of the model of the upper structure. Brickwork with mud mortar is shown with green colour and timber elements in grey colour.

	E (MPa)	v	d (kg/m³)	f_c (MPa)	f_t (MPa)
Mud-mortar brick masonry	150	0.24	1.8	1	0.05
Timber	12500	0.3	0.8	-	-

Table 1: Material properties used for the numerical models developed.

Numerical analysis: Original vs retrofitted temple

a) Modal Analysis

First the dynamic characteristics of the temple were derived by performing modal analysis. Figure 6a presents the first ten periods of the temple. The first two modes were translational in the two horizontal directions corresponding to a period of 0.76 s. The third and fourth modes were translational too, while the fifth mode was rotational, all of a period of 0.34 s. The rest of the modes were local (Figure 6a). Figure 7a shows the first mode of the initial structure. It is evident that the peripheral walls tend to deform out-of-plane (OOP) probably due to the limited capacity of the wooden beams that tie the peripheral walls to the main masonry core. Consequently, this connection is reckoned to be flexible. The periods of the first ten modes for the retrofitted structure are given in Figure 6a. The direction and the type (translational, rotational or local) of the modes were similar to the "original" structure. However, in Figure 6b it is evident that the strengthening options implemented herein reduce the periods of the structure by up to 30%. The first two periods of the retrofitted structure were calculated as 0.58 s signifying a 24% reduction. The beneficial impact of the interventions can be noticed in the first mode shape of the retrofitted structure (Figure 7b). The peripheral wall moves along with the main core of the structure and the tendency for OOP deformations observed in the initial structure has now been diminished.

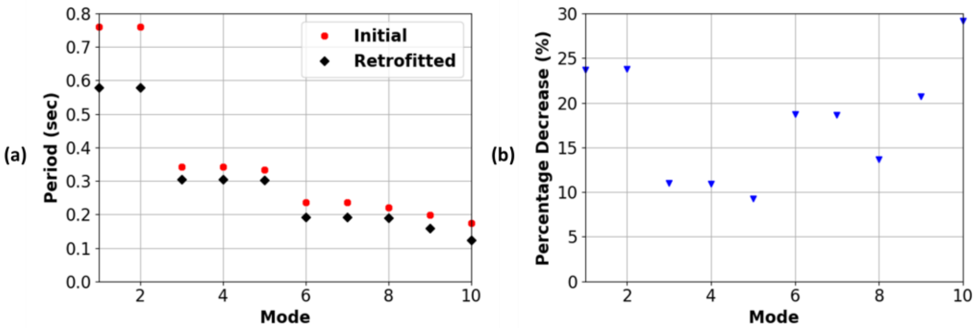

Figure 6: (a) The first ten periods of the structure, as calculated from the numerical model for the initial and retrofitted structure, and (b) the percentage decrease of the periods after the implementation of the retrofitting interventions.

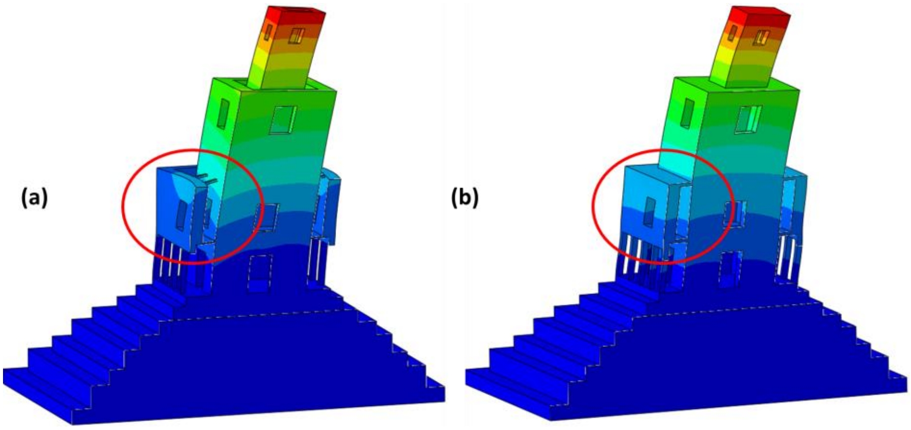

Figure 7: Cross-sections of the structure presenting the first mode shape at the (a) initial and (b) retrofitted state.

b) Pushover analysis

In addition, non-linear static analysis with incrementally increasing horizontal load was performed. The load was applied proportionally to the mass of the structure attempting to imitate the horizontal forces of the seismic excitation. The horizontal load was increased until the structure attained its capacity and the failure modes were fully activated.

Figure 8 shows the failure patterns for the initial and the retrofitted structure as obtained from the pushover analysis. The results are presented for drift 0.5%. This value of drift corresponds to the yield limit for URM structures as defined in EN-1998-1 (2005). For the case of the initial structure (Figure 8a) concentration of damages occurs around the timber pegs which connect the main core to the peripheral wall. Due to the lack of diaphragmatic action at the level of the timber floors, the structure experiences extensive damage at the corners of the peripheral walls. Furthermore, the failure of the timber joist connections lead to OOP deformations in the middle of the peripheral wall, responsible for the development of cracks at these areas. Consequently, it can be inferred that the damages on the initial structure originate from local effects.

For the case of the retrofitted structure (Figure 8b) the main damages manifest as diagonal cracks on the main core. The tendency of the peripheral wall for OOP deformations has been eliminated and the cracks at the corners of the peripheral walls are limited. A drawback appearing at the retrofitted structure is that the wooden pillars participate more in carrying horizontal loads, thus higher stresses are transmitted to their foundation causing the formation of cracks. After the retrofit of the temple, the core of the structure is the main structural element that attracts the horizontal loads and local failure modes are limited. However, any possible strengthening of the wooden pillars needs to be done in combination with interventions on their foundation.

c) Time history analysis

During the time-history analysis the initial structure suffers excessive damage from the very first seconds of the motion (Figure 9a). Damage occurs to almost the entire structure and accumulates during the earthquake until collapse. The top of the structure is extremely vulnerable to earthquake shaking since the walls of the structure do not sit on the main masonry wall but are supported by the timber beams. Such failure mode has been observed in other temples as well during the Gorkha earthquake (Figure 2). It is noted that the pushover analysis failed to capture this type of failure which highlights the neseccity to perform time-history analysis in order to reproduce more accurately the seismic response of the investigated structure.

The deformed shape of the retrofitted model at the end of the time-history analysis is shown in Figure 9b. It is noted that even after the retrofit has taken place, the inner wall at the top of the structure deforms excessively and a collapse of that part of the structure seems highly likely. Nevertheless, the response of the rest of the structure has been enhanced. In particular, the structure presents a box-type behaviour with all the structural elements to follow the motion of the main core. Therefore, concentration of stresses and OOP deformations are less pronounced and the cracks caused by these local failure modes are rather limited. The structure exhibits regions of tensile failures which implies that cracks should be expected on the structure after a similar seismic excitation.

Figure 10a presents the drift values of the main core for the initial and the retrofitted structure. The results are given for the strong part of the record, that is from 20 to 50 s. Note that the analysis with the initial structure is terminated for t=46 s due to excessive damage inflicted on the damage bringing the structure close to collapse. Until that point the maximum recorded drift on the main core for the initial structures reaches the value of 1%, while the corresponding value for the retrofitted system is limited to 0.4%. Another point that the retrofitted structure outperforms the initial one is the OOP deformation of the peripheral wall. In particular, the relative deformation between the main core and the peripheral wall at the level3 of the structure (Figure 3a) is given in Figure 10b. While for the retrofitted case the OOP deformation is zero, for the initial structure accumulation of OOP displacement is noticed. Therefore, toppling of the peripheral for the temple at its initial state is extremely likely.

Consequently, it can be stated that the recommended interventions fail to tackle the collapse of the inner masonry core at the top of the structure. Nonetheless, the rest of the retrofitted structure behaves better than the initial structure. The total collapse of the structure seems to be prevented even though crack regions along the temple should be expected. This behaviour can be considered favourable for ordinary buildings during a catastrophic earthquake as the Gorkha one.

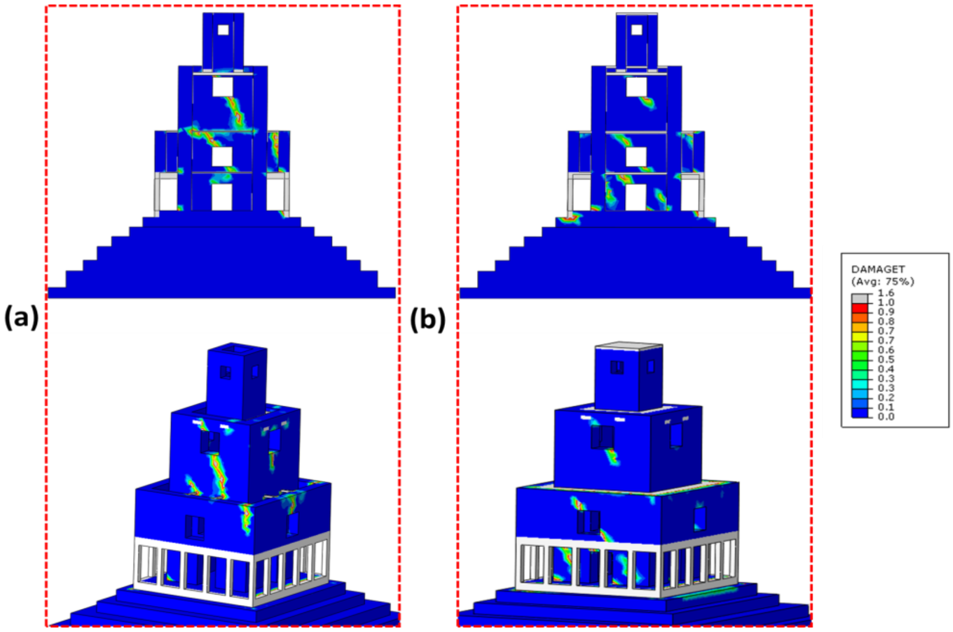

Figure 8: The distribution of tensile damage for cross-section (top) and perspective view (bottom) of the deformed structure when drift 0.5% is attained during the pushover analysis for (a) the initial, and (b) the retrofitted structure.

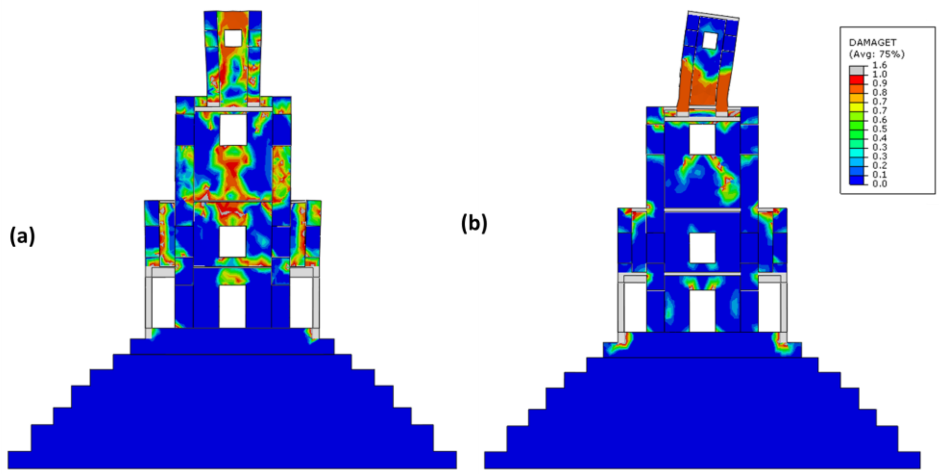

Figure 9: The distribution of tensile damage for cross-section of the deformed structure during the time-history analysis for (a) the initial structure at t=10 s, and (b) the retrofitted structure at the end of the analysis.

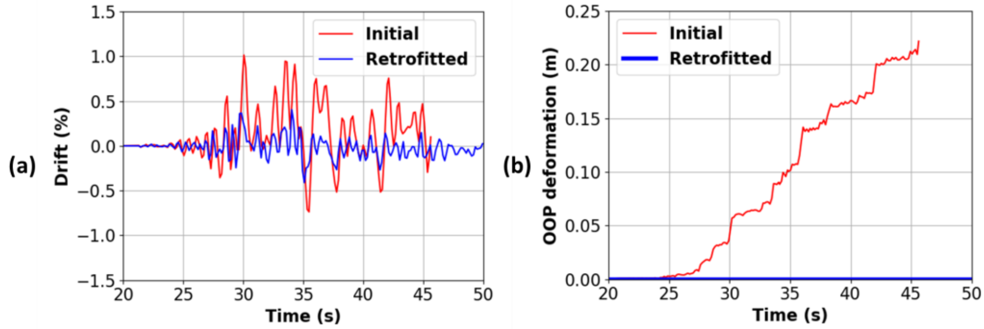

Figure 10: From the time-history analysis (a) the drift values of the main core, and (b) the OOP deformation of the peripheral wall are given for the for the strong part of the record, that is from 20 to 50 s.

Conclusions

During the strong Gorkha earthquake that hit Kathmandu Valley several historical monuments suffered extensive damage and numerous temples collapsed. Although a significant amount of work has been done to understand the seismic vulnerability of such historic temples in Nepal, little work has been done to investigate simple measures to retrofit them and improve their stability for future earthquakes. This study aims to investigate the seismic vulnerability of repaired and retrofitted historic temples in Nepal, providing insights into whether the existence of small interventions improves the seismic resistance of the temples and changes their dynamic characteristics. The three-tiered Jaisedewal Temple, typical example of the Nepalese architectural style, was chosen as a case study. A three-dimensional numerical model of the Jaisedewal Temple has been developed in the commercial code ABAQUS. Pushover and time-history analyses were performed, the results of which offered some interesting findings.

The existence of flexible connections along the structure needs to be taken into consideration during the modelling in order to retrieve a realistic simulation of the seismic behaviour of the structure. Modelling the top part of the temples that typically sits on timber elements and is not connected directly to the rest of the main masonry core is of paramount importance in order to obtain a precise evaluation of the damage patterns of the structure.

Pushover analysis cannot simulate the real kinematic nature of a seismic excitation but can provide a rough estimation of some distinctive failure patterns. In particular, pushover failed to capture the collapse of the top part of the Nepalese temple under investigation. Therefore, time-history analysis is deemed essential to accurately reproduce the response of the structure during a seismic excitation.

The time-history analysis revealed the weak points of the initial structure. The capacity of the temple is exceeded during the very first seconds of the applied excitation. The damage propagates along the whole structure during the next seconds of the record and the collapse is considered as inevitable. A retrofitting scheme to improve the seismic capacity of the structure is regarded essential.

Interventions of low interference for the seismic retrofit of the structure were highlighted and their impact on the seismic behaviour of the structure was examined. The suggested interventions reduce the periods of the structure up to about 30%. Furthermore, a box-type behaviour is obtained, and the effect of local failure modes is limited. Nevertheless, the collapse of the inner masonry core at the top of the structure is not prevented. After retrofitting the structure, the drift of the main core and the OOP deformations at the peripheral wall are reduced. The wooden pillars participate more in carrying horizontal loads and thus extra stresses are transmitted to their foundation. Although the beneficial impact of the proposed strengthening scheme was proven, in order to meet the performance level prescribed by the Codes, further interventions should be considered. Historical monuments such as the Nepalese Pagodas are complex structures and a holistic approach is required in order to understand and improve their seismic response.

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