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Multi-tiered Nepalese temples: Advanced Numerical investigations for assessing performance at failure under horizontal loads

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Abstract

In this study, the performance at failure of old multi-tiered temples in Nepal under horizontal loads mimicking a seismic action has been addressed using three different computational approaches, including a) linear elastic; b) nonlinear static; and c) nonlinear dynamic analyses. Also, a sensitivity study was undertaken to understand the influence of wall thickness and height of Nepalese temples on their behavior at failure. Vertical oscillating modes using the elastic response spectrum of the Nepalese Building Code were obtained using linear analysis. Nonlinear static analysis (NLSA) were implemented to obtain the load carrying capacities of different in geometry temples e.g. different thickness of central core walls and number of tiers. Additionally, nonlinear dynamic analysis (NLDA) using the Finite Element Method (FEM) were performed to evaluate the characteristic tensile damage patterns. The results comparatively indicate the weakest zones depending on wall thickness, central core slenderness, opening distribution, box-like confinement, vertical misalignment of walls and so forth. Also, the results of the NLDA affirm high vulnerability of the multi-tiered temples showing extensive cracks at relatively low peak ground accelerations. It is anticipated that outcomes of this study can help practicing engineers to understand how these structures behave at failure when subjected to seismic loads and provide insights towards their strengthening and retrofitting.

Keywords: Behavior at failure; Cultural heritage Nepal temples; Failure under seismic loads; Masonry; Finite element failure analysis

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1. Introduction

The preservation of cultural heritage is a crucial issue all over the world, such as Nepal, whose territory is characterized by both significant seismicity and presence of a large amount of monumental masonry constructions of major historical importance, such as temples. Despite the importance of such kind of structures, research has recently been more focused on architectural heritage buildings belonging to Mediterranean countries (industrially developed but at the same time in high seismicity regions), and hence on masonry churches [1-5], towers [6-10], etc. The lessons learnt from previous earthquakes together with deep study of dynamic behavior and structural weaknesses of monumental buildings lead to proper design of potential interventions which should guarantee their conservation [11]. In this context, experience and expertise gained from understanding the seismic behaviour of structures of architectural heritage can be used and applied in understanding the complex and highly non-linear behaviour of Nepalese temples when subjected to earthquake loading.

Multi-tiered temples (See Figure 1) built in Nepal are traditional and monumental structures which apart from their architectural significance, present both valuable historic and archaeological importance [12]. In addition, such structures play vital roles in the life of thousands of people, since they are centers of religious activity. However, the 2015 Gorkha Earthquake and its related aftershocks dramatically altered the iconic historic structures in Nepal. According to Nepal's Department of Archaeology, around 750 heritage structures of significant cultural and religious value (e.g. temples and shrines) were affected. For example, Bhaktapur, a UNESCO World Heritage Site, was identified as one of the 14 most affected districts in the country. Discussions to place it on UNESCO's List of "*World Heritage in Danger*" are already underway. Moreover, Nepal's Department of Tourism, reported that the consequences of the earthquake had a direct impact on cultural tourism. During the last year, the number of non-SAARC visitors (excluding China) entering the Bhaktapur district dropped by more than 80%, resulting in a loss of income exceeding more than US$ 3M on entry fees alone. Religious, cultural, social, and economic aspects are hence all interconnected here. With Overseas Development Assistance pledges of $2.5 billion US dollars, Nepal's Government approved the rehabilitation of much of Kathmandu's historic infrastructure, but there is continued tension between interpretations of Sendai's '*Build Back Better*' framework and the obligation to preserve the authenticity and intangible values of its UNESCO World Heritage Properties.

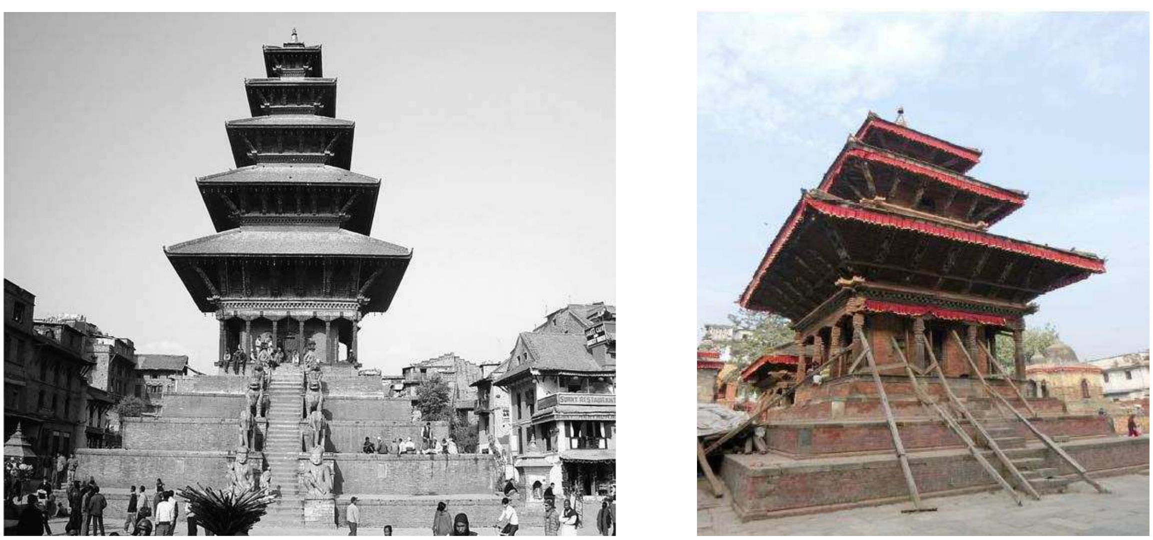


Figure 1 – Multi-tiered temples: Nyatopol temple (left) [12] and Gopinath temple (right) [13].

The architectural style of multi-tiered temples is quite different from other Asian countries in vicinity of Nepal [14] [15]. In particular, Nepalese multi-tiered temples are characterized by a combination of brickwork constructed with mud mortar. The masonry walls are very thick providing enough stiffness to the system. Although mud mortar is susceptible to weathering and has low bond strength characteristic, it has been extensively used in Nepalese architecture since it is both a cost effective and energy efficient material [16]. In addition, these structures are characterized by: a) geometric symmetry; b) asymmetry in mass distribution along the height of the structure i.e. the mass of the structure increases from top to bottom; and c) the temples are resting on a huge and massive plinth (or exposed foundation) [17] [18]. What is recognized in [19] is that this huge plinth acts as a dissipating device which reduces the earthquake risks associated with soft soils. According to Tiwari [20], the foundation of the tiered temples is often just as wide as the plinth base itself, while [21] suggests that most of the tiered temples constructed on top of a wide plinth base have very good seismic performance. The plinth shape varies from temple to temple and depending on the dimensions and material constructed can significantly affect the response of the temple during an earthquake. Roofs are covered with tiles or in some cases with metal sheeting. Timber elements are used mainly to form floors and roofs. Timber joists are found in such temples which are connected to the masonry wall with the floors. The columns which cover the exterior perimeter of the temples are also made of timber. Traditionally, timber beams can also be found at the last floor supporting the top part of the temple. An element that is the most common addition in all tiered temples is the ring beam or laced beam, which aim is to ensure box-like behavior by confining the central masonry core walls. The roof system is very flexible and have large overhanging eaves of about one meter, which serves primarily to protect the structure from severe monsoons as well as exposure to the sun during the dry periods of the year [22]. According to [18] [23] [24], Nepalese temples are vulnerable to earthquakes since:

I. Walls are eccentric in vertical direction;
II. Mud mortar between bricks is relatively week;
III. The roof is of considerable size and mass;
IV. Soft-storey effects could be applicable due to presence of large openings; and
V. Often poor connection between structural elements exists.

Modelling multi-tiered Nepalese temples is a very complex task due to numerous uncertainties and lack of accurate material characterization [25]. In the past, linear FE models have been developed in which solid finite elements were used to model the masonry walls, shell elements were used to represent the roof, linear element were used to simulate the beams and columns while the soil-structure interaction effects neglected (e.g. [12] [19] [26]). In addition, in most cases, the exposed foundations of the temples were not taken into consideration. Linear parametric studies were also undertaken in order to estimate the influence of material parameters into the dynamic response of the temples (e.g. [19]) taking into account the global stiffness of the structure. Aligned with the UN Sustainable Development Goal 11.4 "Strengthen efforts to protect and safeguard the world's cultural and natural heritage" and Nepal's national strategy for disaster risk management (2009) the research presented herein aims to assess seismic performance of old multi-tiered temples in Nepal using different computational approaches, including: a) linear elastic; b) nonlinear static; and c) nonlinear dynamic analyses. A sensitivity study is also undertaken to understand the influence of wall thickness and height of Nepalese temples on their seismic behavior. Vertical oscillating modes using the elastic response spectrum of the Nepalese Building Code were obtained using linear analysis. Also, nonlinear static analysis (NLSA) were implemented to obtain the load carrying capacities of different in geometry temples e.g. different thickness of central core walls and number of tiers. The development of the different computational models and the results and analyses are presented below.

2. Numerical modeling of multi-tiered temples

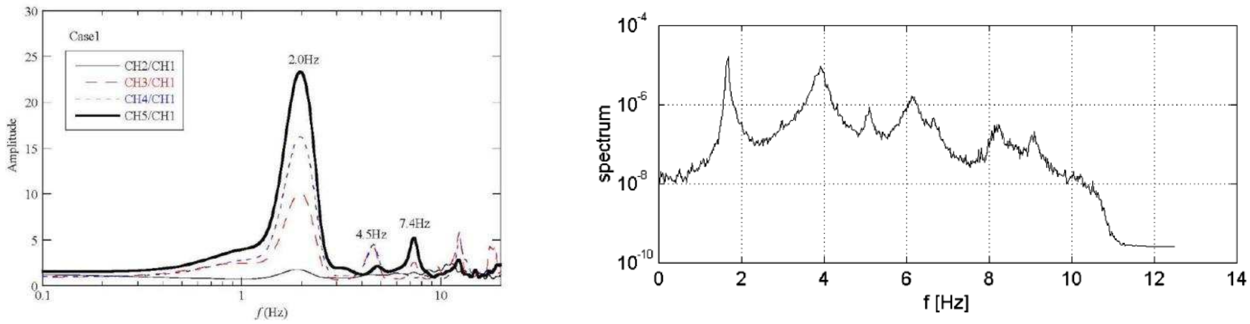
Initially, elastic models of the Gopinath and Nyatopol temples were developed and modal analysis were undertaken in order to verify the dynamic properties of the structures. Eccentric effects of the walls and the top of the temples were considered. Also, it was assumed that the walls of the temples will allow stresses to be distributed from one level to the other (see Figure 2). In addition, in both temples, geometric eccentricity exists at the top floor level; i.e. the walls at the top are not aligned with the ones at the level below. Numerical models based on the macro-modelling approach have been developed in which masonry represented using the Concrete Damaged Plasticity law (CDP), available in ABAQUS software. C3D4 tetrahedron linear type elements were used. Additionally, tetrahedron finite elements were used to generate the complex geometrical shapes of the temples. In addition, for the development of the numerical models it was assumed that: a) the plinth is rigid; b) the floors are rigid, since timber beams are attached to walls; c) the roof is considered as non-structural element; and d) the weight of the wooden pinnacle is neglected at the very top since its weight is small relatively to the other parts of the structure. From observable evidences after earthquakes of temple structures that are founded on soft soils it is can be seen that the effect of massive plinth is such that eliminates earthquake risks. Nevertheless, its actual impact on the total behavior of temples is still not clear until the end and it is currently under investigation. It is known that some plinths are made exclusively of solid bricks while others consist of the combination of solid bricks and softer materials, such as mud. For these reasons, it is necessary to do more refined numerical model of the plinth within the complete structural model using linear and nonlinear springs in order to account for its actual mechanical properties and to properly estimate its impact on the dynamic response of the temple.

Another issue is related to the mechanical quality of the underlying soil. Seismic waves travels from the bedrock through deformable deposit of soil above, finally hitting the structure with modified frequency content compared to the original one. Such phenomenon could have relevant influence on the response of the structure above the soil surface and this strongly depends on the quality of the soil. Being in a possession of detailed soil properties data one can estimate influence of the soil on structural response by modelling soil with linear springs.

Structural fragility can be performed by taking into account damages along this structural element, thus adopting lower value of elasticity modulus.

2.1 Dynamic identification of the multi-tiered temples

Figure 2a shows the frequency spectrum for Gopinath temple obtained from the Tokyo National Research Institute for Cultural Properties (TNRICP). The Japanese researchers used portable vibration monitoring system (SPC-51) and servo-velocity meters (VSE-15D) and the measurements were taken from the sensors placed at 20 positions during the micro tremors [3]. Figure 2b, shows the frequency spectrum for Nyatopol temple, which is the tallest temple in Nepal with five tiers. The spectrum was obtained using advanced stochastic subspace identification method (SSI) which gives more accurate results comparing to peak picking method (PP) that determines natural frequencies by taking maximum values on the graph of the average normalized power spectral density for all measured channels [2].



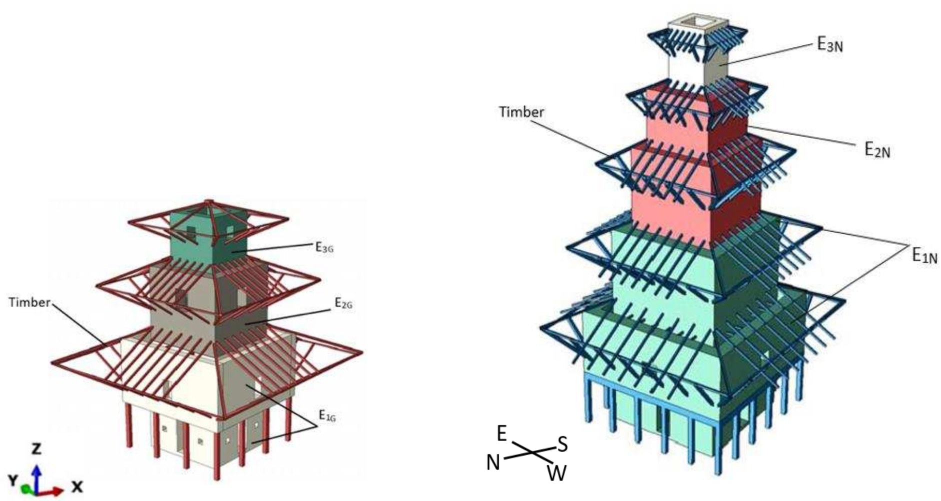
(a) (b)

Figure 2 - (a) Representative Fourier amplitude ratio for Gopinath temple (East-West direction) with the first three natural frequencies [13] and (b)re-sampled data and modified power spectrum an the top of Nyatopol temple [12]

Table 1 – Identified and calculated frequencies for Nyatopol temple using subspace identification (SSI) method [12]

Direction	SSI f(Hz)
First bending (N–S)	1.677
First bending (E–W)	1.739
Second bending (E–W)	3.872
Second bending (N–S)	3.89
Third bending (E–W) coupled with torsion	6.358
Third bending (N–S) coupled with torsion	6.128

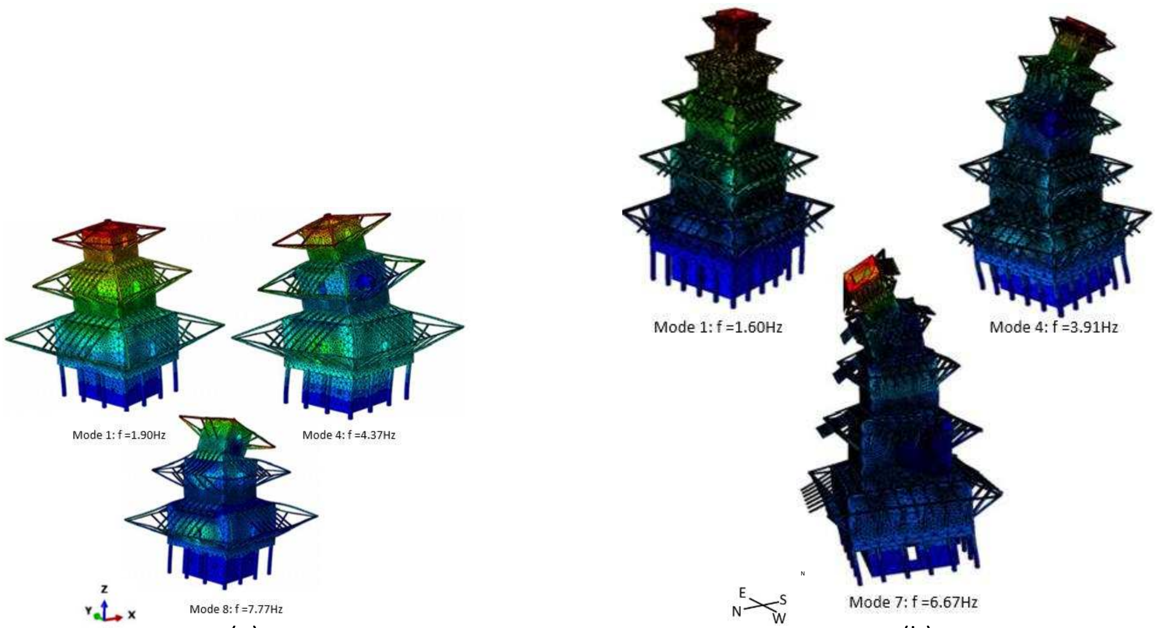
Initially, the mechanical properties of the masonry material to be inputted into the numerical model calibrated. Material calibration was performed by dividing the structure into three regions. A different modulus of elasticity has been selected for each case (see
Figure 3).



(a) (b)

Figure 3 – Three dimensional FE models of: (a) Gopinath and (b) Nyatopol temples. Different colors represent the different modulus of elasticity assigned to the models (see also Tables 2 and 3)

According to [28], the modulus of elasticity for mud masonry found in Nepalese temples is approximately 274 MPa. However, an experimental campaign on material properties of Nepalese temples carried out by UNESCO showed that the elastic modulus of mud mortar ranges from 87 MPa to 150MPa [13]. Density of masonry is taken as 1800 kg/m3. A model calibration study has been undertaken and the obtained material properties for the Gopinath and Nyatopol temples are shown in Table 2 and 3 accordingly. The elasticity modulus of the timber elements of the temples was taken as 12,500 MPa [13]. Figure 5 shows the Eigen-shapes and corresponding Eigen-frequencies as obtained from the numerical simulations for the Gopinath and Nyatopol temples. The numerical frequencies obtained from the numerical model are less than 5% different to the ones obtained experimentally.



(a) (b)

Figure 4 – Calibrated modes matching with experimental ones and frequencies of: (a) the Gopinath and (b) the Nyatopol temples

Table 2 – Calibration of parameters for Gopinath temple– E_1=140MPa; E_2=141MPa; E_3=140MPa

Mode	Direction	Numerical frequency (Hz)	Error (%)
Mode 1	Horizontal	1.90	-5.00
Mode 4	Horizontal	4.37	-2.88
Mode 8	Horizontal	7.77	5.00

Table 3 – Calibration of parameters for Nyatopol temple –E_1=265MPa; E_2=245MPa; E_3=245MPa

Mode	Direction	Numerical frequency (Hz)	Error (%)
Mode 1	Horizontal	1.60	-4.59
Mode 4	Horizontal	3.91	-0.98
Mode 7	Horizontal	6.67	4.90

3. Linear analyses

Linear analysis of the temples were undertaken to understand the local dynamic response and identify the possible local failure mechanisms that can occur before their global collapse. The analysis was performed using tetrahedron finite elements and linear elastic material model described with parameters from Section 2. From the analysis results, four possible failure mechanisms can develop in these temples, including:

a) Crushing of masonry at the connection between external walls and central masonry core (with timber links);
b) Out-of-plane overturning of the external walls supported by the timber beam at the first floor;

c) Local failure due to eccentricity phenomena at the connection between the top tower and the central masonry core; and
d) All of the above or combinations of the above.

During dynamic identification of the structure, it was found that the timber links connecting the central core with the external wall have essential role in lacing the structural parts, thus forcing all structural parts to displace together. However, concentration of compressive and tensile stresses in the masonry can occur, if there is no smooth continuous load transfer along the upper edge of external walls. In the case when the timber links were not present (non-existing as structural elements or broken due to excess of load carrying capacity), the external walls were not restrained along the upper edge and unable to resist lateral loads alone. This may lead to overturning as a result of out-of-plane failure associated with extensive cracking pattern in the vertical direction at the corners.

The masonry tower at the top of the temples is supported by orthogonal set of timber beams that are embedded into masonry core and are eccentrically positioned with respect to the central masonry core. During seismic loading, cutting (slit cracks) and cutting-compression combination cracks can developed in the masonry [29] (see
Figure 5). Significant oscillations can also be observed in the Z direction (see Figure 6), which may additionally contribute to the development of such cracks and failure of the walls.

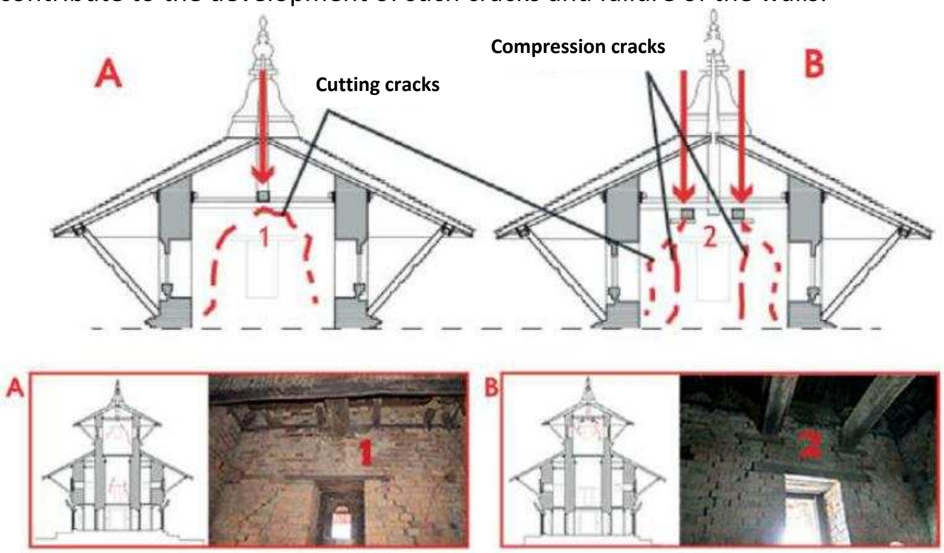


Figure 5 –Example of Jagannath temple A: Cutting cracks; B: Compression cracks [29]

Figure 6 shows the relevant natural modes of vibration for the Gopinath temple. From Figure 6, the natural modes in the Z direction significantly influence the global dynamic response of the temple. Also, in Figure 6, all the three plotted modes in Z direction have more or less substantial effective mass and fall at the plateau of the elastic response spectrum. This is due to the fact that the top tower is supported by timber beams, thus being eccentrically positioned with respect to the substructure (central masonry core). It is the eccentricity of the masonry walls which produce discontinuity in vertical stress transmission. Consequently, static and dynamic effects coming from the structure above are concentrically transmitted to the masonry core being applied as patch loads before being smeared through disturbed region. This effect can cause local crushing of masonry material at the points of stress concentration. Crushing of masonry at the contact between timber links and external masonry wall can be observed from the outcomes of the pushover analysis performed. The results indicate crack pattern due to excess stress concentrations. In addition, Figure 6 presents the influence of relevant modes on the total dynamic response of the temple in terms of distribution of effective masses in all three directions comparing to elastic spectrum. In Figure 6 the normalized displacements are shown. In particular, the regions with blue color are those were no displacement occurred. On the other hand, regions with red color are those were maximum displacement occurred.

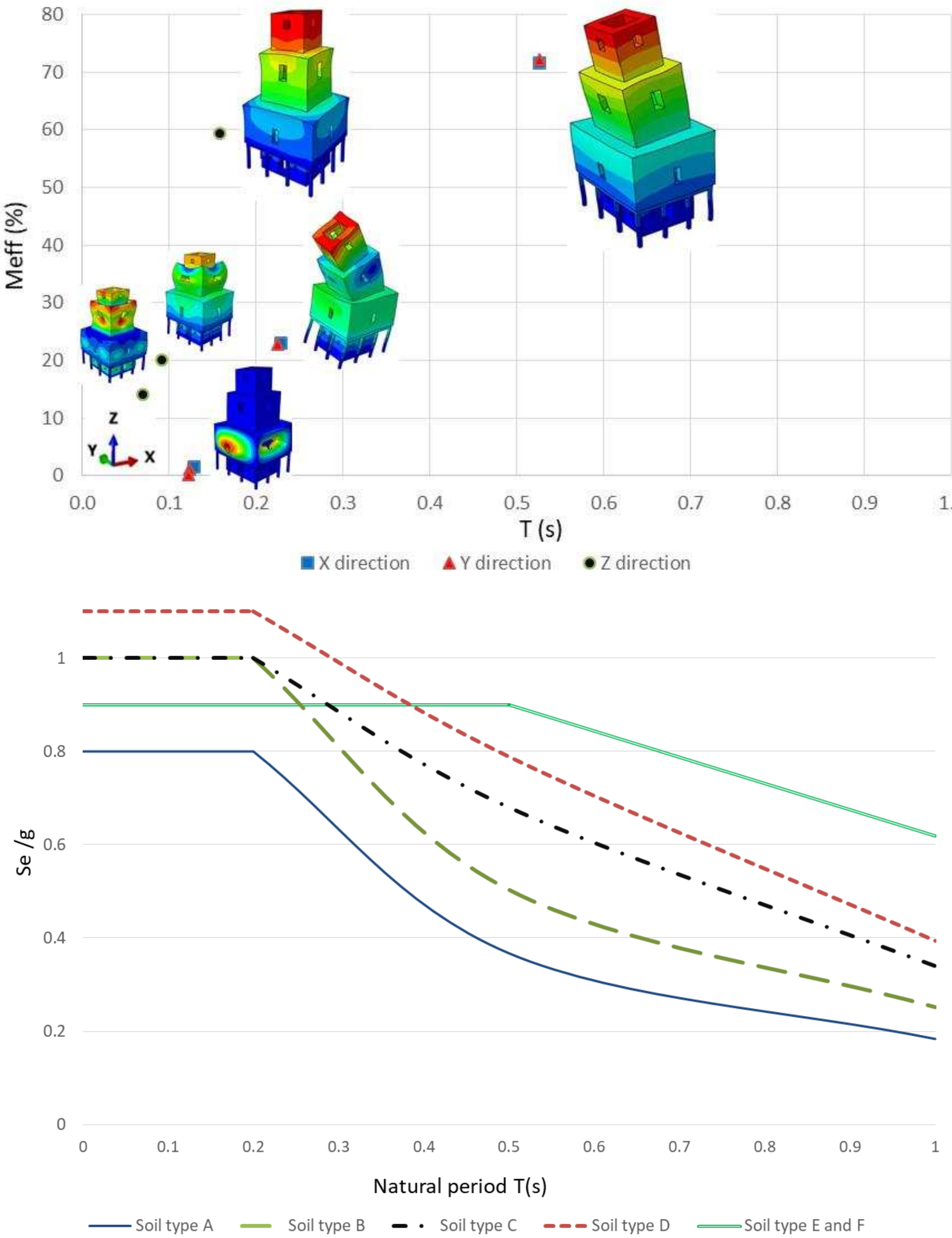


Figure 6 - Distribution of relevant effective masses in X, Y and Z directions of Gopinath temple with emphasis on the most influential global and the most critical local modes (blue color: zero displacement region; red color: maximum displacement regions) vs. Elastic response spectra taken from NBC 2005

4. Pushover analysis

Nonlinear static procedure (NSP) or push over analysis can be used for seismic demand estimation (e.g. [30] [31]). This type of analysis is quick and can provide an indication of the areas to be more vulnerable during an earthquake. However, the analysis does not take into account the change of modal frequencies due to yielding and cracking of the structures. For this reason, results from the push over can be conservative when non-uniform cracking in the structure occurs [32]. In the context of reliable evaluation of seismic capacity, nonlinear static analyses with substantial computational effort is frequently used instead of performing dynamic analyses or looking for analytical solutions [33]. Lateral load pattern chosen for this analysis is linearly distributed along the height with the maximum value at the top of each temple.

A macro-element model has been developed and the concrete damage plasticity model (CDPM) [34] implemented in ABAQUS has been used [35]. General uniaxial stress-strain behavior of this model (see Figure 7) is able to reproduce actual non-linear behavior of masonry in both tension and compression (see Figure 8).

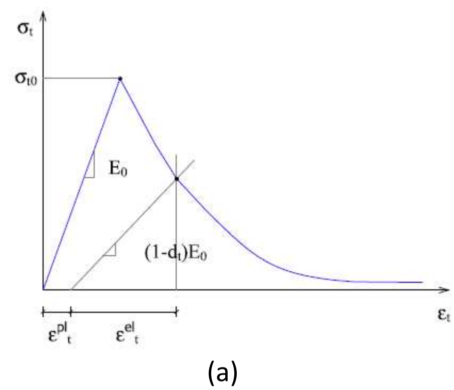


Figure 7 - Non-linear uniaxial stress-strain general behavior ((a) in tension; (b) in compression) [36].

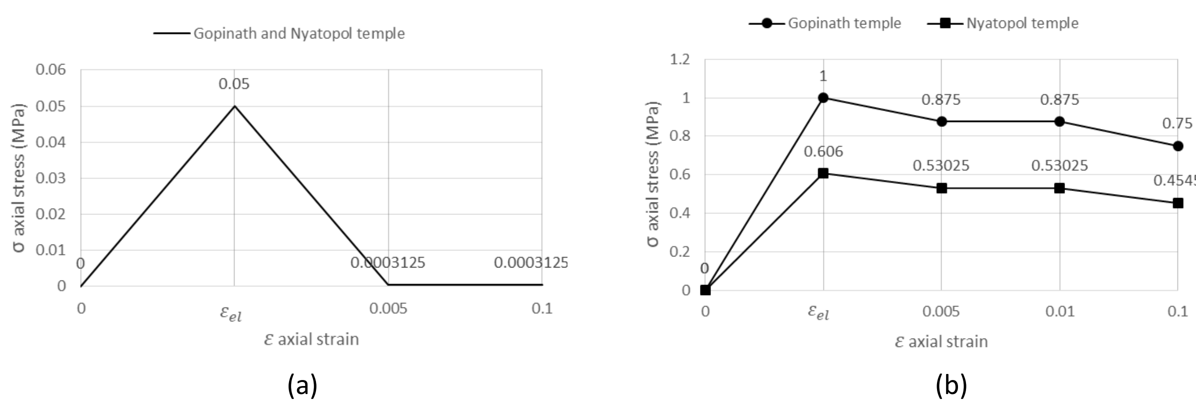


Figure 8 - Non-linear uniaxial stress-strain actual behavior of masonry ((a) in tension; (b) in compression) [12] [13].

Comparative pushover analyses have been performed on Gopinath and Nyatopol temples by varying their height and wall thicknesses. These are the two main parameters that govern dynamic behavior and structural resistance against lateral loads of this type of structures. This is evident in capacity curves diagrams in Figure 12 and Figure 13. Varied values for structural height were obtained by adding or reducing one or more characteristic tiers in Gopinath and Nyatopol temple, respectively. Wall thickness was reduced by subtracting rounded numbers, finally reducing its value approximately 2 times in both cases. Additionally, in order to substantially simplify numerical calculations, models without roof were adopted. The geometric parametric configuration used as part of this study is shown in Figure 9.

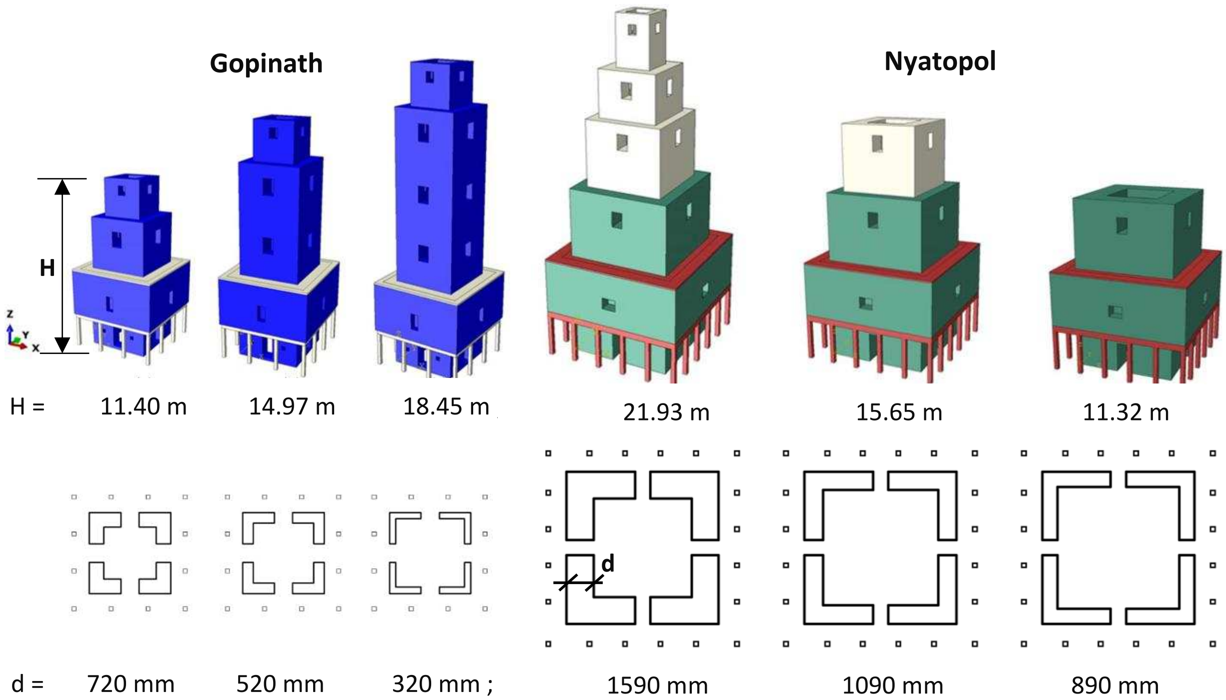


Figure 9 - Gopinath and Nyatopol temples: Different heights (H) and thicknesses (d) of masonry walls

Figure 9 shows the different geometrical configurations of temples obtained by the variation of two main parameters: (a) the total height (H) of temple; and (b) the thickness of the wall (d) at the central masonry core at the ground level. Finite element meshes with number of nodes and elements are presented in Figure 10 and Figure 11 for Gopinath and Nyatopol temples accordingly. The average size of the tetrahedron elements used in the numerical models are 300 mm for Gopinath and 400 mm for Nyatopol temples.

Mesh									
Height H (m)	11.4			14.97			18.54		
Thickness d (mm)	720	520	320	720	520	320	720	520	320
Number of nodes	10759	10001	9460	12439	11541	10582	14396	13221	11789
Number of FE	42947	37955	34280	49820	43645	37186	58361	50318	40776

Figure 10 - FE models for Gopinath temple with varied height (H) and central masonry core wall thickness at the base (d)

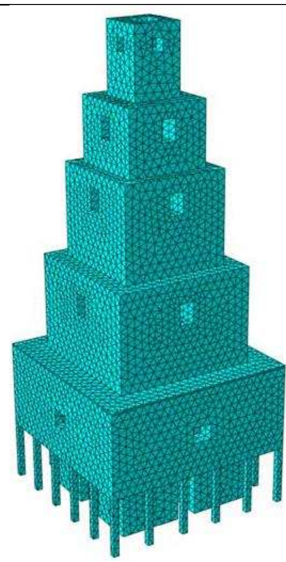
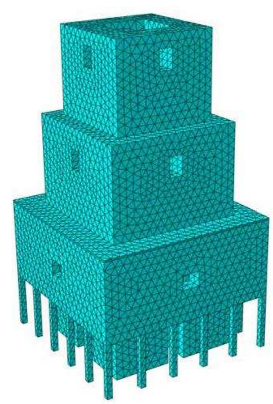
Mesh									
Height H (m)	21.93			15.65			11.32		
Thickness d (mm)	1590	1090	890	1590	1090	890	1590	1090	890
Number of nodes	18233	15919	15106	16735	14343	13501	14525	12571	11716
Number of FE	81309	65844	60430	75256	59486	54031	65079	52539	46489

Figure 11 - FE models for Nyatopol temple with varied height (H) and central masonry core wall thickness at the ground (d)

Figures 12 and 13 show the capacity curve diagrams (normalized as V/W=Se/g where V is total base shear and W total weight of the structure) and tensile damage patterns of the corresponding structures in three-dimensional view. Capacity curves give information about variation of ultimate base shear denoted with black circles at the position of ultimate displacements. The determination of the ultimate displacement is not an easy task, because softening is rarely visible in global capacity curves for quasi no-tension materials. The Italian Code for the Built Heritage (2011) helps in the determination of the stopping criteria in presence of complex 3D FE models and damaging materials with almost vanishing tensile strength and relatively good compressive resistance, i.e. in all those cases where - as for masonry structures belonging to the built heritage - pushover curves with global softening are hardily obtainable.
Stopping criteria by the Italian code NTC (2018) and the most diffused international building codes where displacement based design is applied are generally not applicable for complex historical buildings and this is the reason with the Italian Code for the Built Heritage (2011) suggests to push the structure up to "meaningful displacements" of the control point. Considering the difficulties in the definition of the displacement at the ultimate limit state, the same code suggests to bracket the ratio between the elastic force and the maximum force in the equivalent bi-linear system in the range 3-6. The procedure is a recursive one, but after few iterations the ultimate displacement of the control point is reasonably bracketed according to the aforementioned indications.

Tensile damage patterns consist of cracks at the beginning of each floor (see Figure 12). This is evident for the Nyatopol temple (see Figure 13) which possesses characteristic pyramidal configuration and abrupt change of stiffness at each tier with vertical misalignments of walls of central masonry core. Taller structures behave similarly to towers forming plastic hinges at its bases accompanied with horizontal and diagonal cracks. Figure 12 shows that stocky temples experience shear failure, while in the case of slender temples flexural failure with formation of plastic hinge at the level of second floor was observed. The intermediate cases which were investigated found to undergo a combination of shear and flexural behavior at collapse. Similarly, Figure 13 shows that Nyatopol temples with original heights exhibit flexural failure almost at each tier regardless the thickness of the walls of the central masonry core. In the case of the stockiest temples, shear failure prevails with horizontal and diagonal cracks. However, moderately slender temples fail as a result of a combination of the two failure modes. Finally, in both groups of temples progressive change of failure mechanism is evident, passing from shear failure for stocky to flexural failure for slender temples. Conveniently established failure mechanisms for certain typology of structures are crucial step towards simplified analysis using pre-assigned failure modes, which must be justified by nonlinear dynamic analysis.

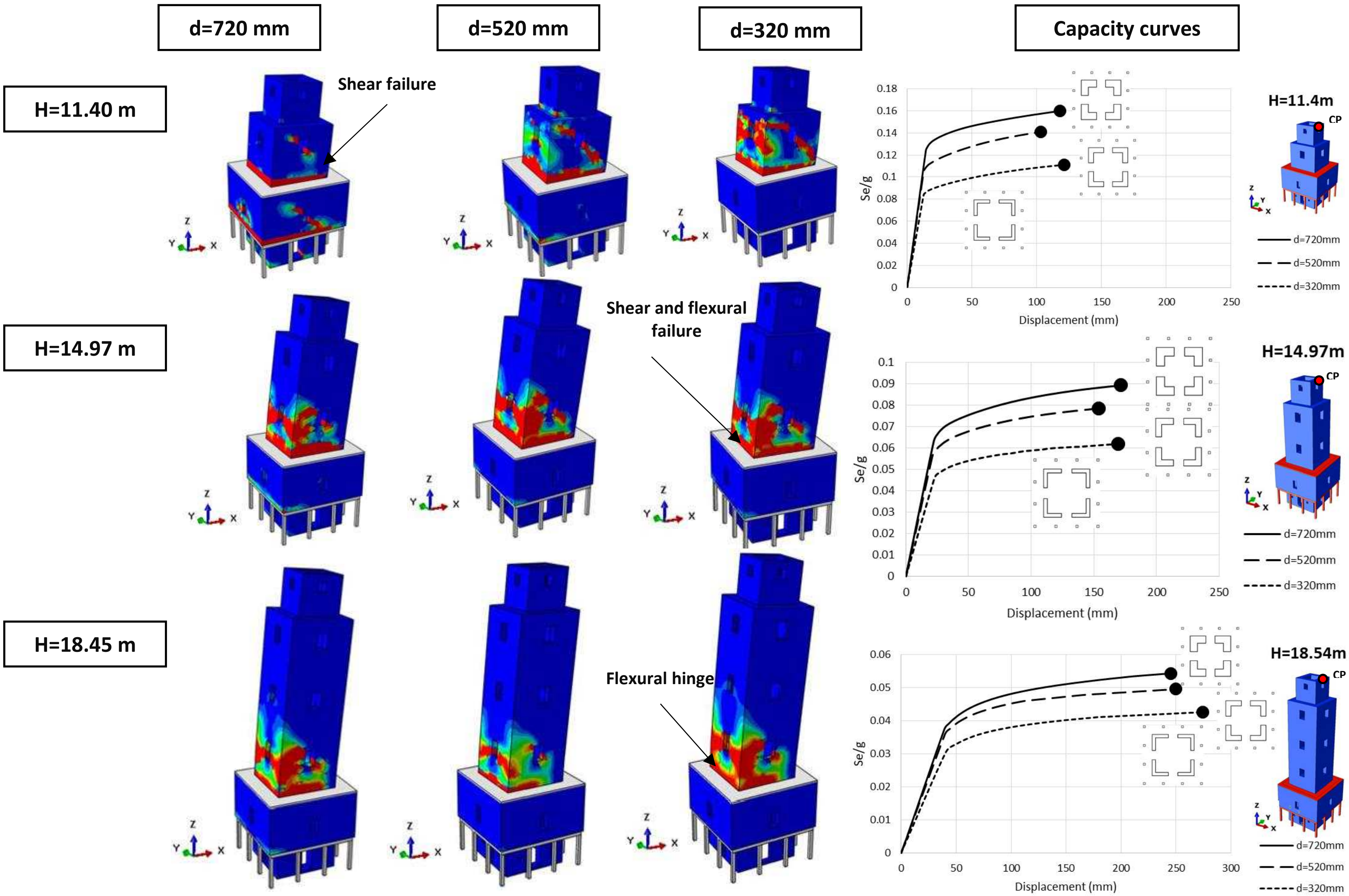


Figure 12 – Tensile damage patterns and capacity curves for **Gopinath temple** with varied thickness at the ground floor d=720 mm, d=520 mm and d=320 mm and varied heights H=11.40 m, H= 14.97 m and H=18.54 m (The blue color stands for undamaged structural regions while the red color indicates fully damage regions. Control points are denoted at each structure)

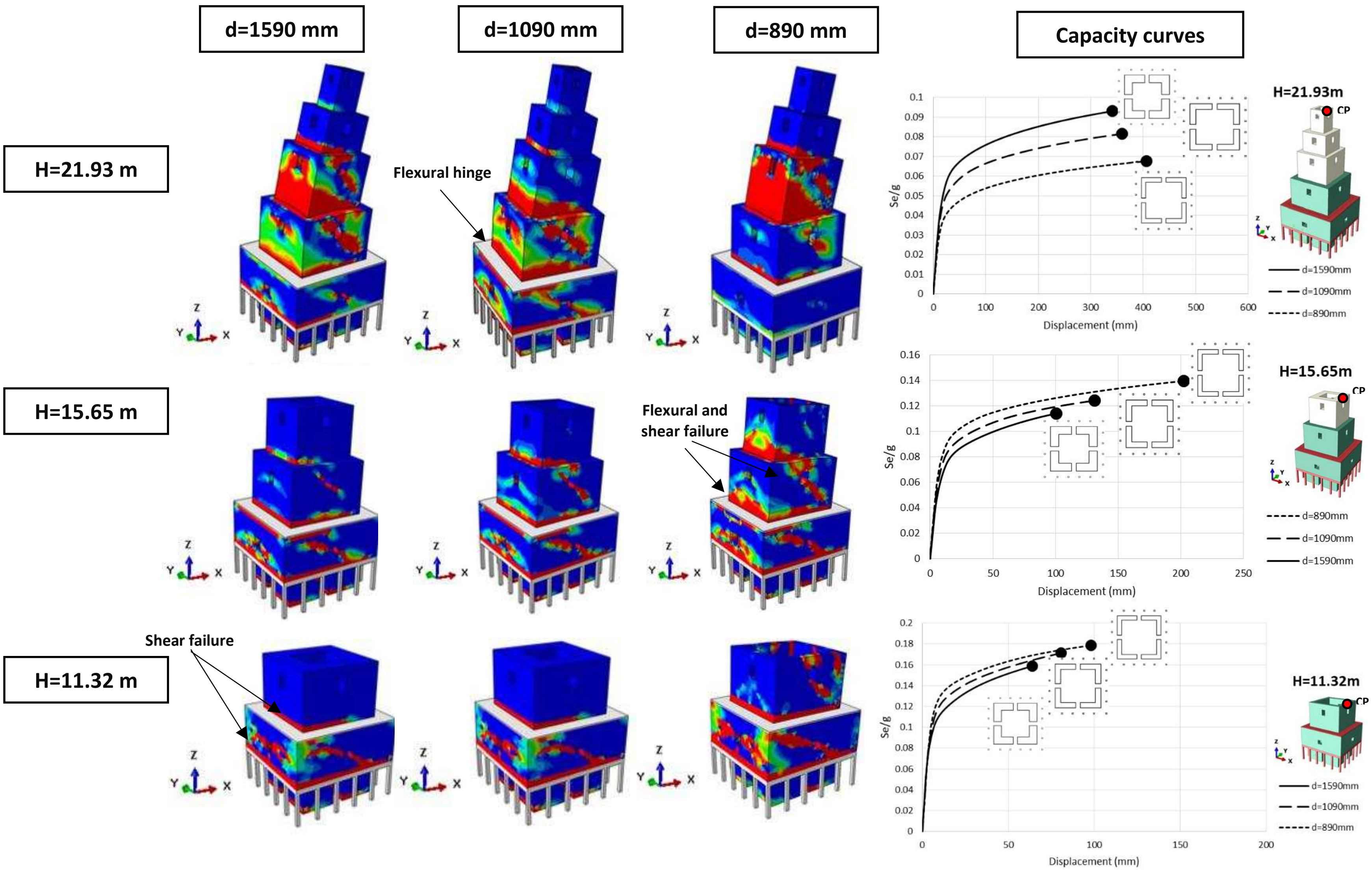


Figure 13 – Tensile damage patterns and capacity curves for **Nyatopol temple** with varied thickness at the ground floor d=1590 mm, d=1090 mm and d=890 mm and varied heights H=21.93 m, H= 15.65 m and H=11.32 m (The blue color stands for undamaged structural regions while the red color indicates fully damage regions. Control points are denoted at each structure)

The numerical results from the pushover analysis related to variation of base shear at elastic limit and the ultimate base shear are shown in accordingly.

Table 4 and Table 5 for the Gopinath temple and Table 6 and Table 7 for the Nyatopol temple accordingly.

Table 4 – Comparison between total base shear at elastic limit and at ultimate for fixed wall thickness (d) and variation of total height (H) for Gopinath temple (decrease (%) of base shears is given with respect to the structure with original height)

			V/W = base shear/structural weight (g)					
	Structure	**Height (m)**	**d=720 mm**	**Decrease (%)**	**d=520 mm**	**Decrease (%)**	**d=320 mm**	**Decrease (%)**
At elastic limit	**Original**	**H=11.40**	0.130	-	0.110	-	0.085	-
	+ 1 floor	**H=14.97**	0.065	50.000	0.058	47.270	0.048	43.520
	+ 2 floors	**H=18.45**	0.040	69.230	0.037	66.360	0.033	61.170
Ultimate	**Original**	**H=11.40**	0.160	-	0.140	-	0.117	-
	+ 1 floor	**H=14.97**	0.090	43.750	0.079	43.571	0.062	47.009
	+ 2 floors	**H=18.45**	0.055	65.625	0.045	67.857	0.043	63.247

Table 5 - Comparison between total base shear at elastic limit and at ultimate for fixed heights (H) and variation of wall thickness (d) for Gopinath temple (decrease (%) of base shears is given with respect to the structure with original thickness)

			V/W = base shear/structural weight (g)					
	Thickness	**d (mm)**	**H=11.4 m**	**Decrease (%)**	**H=14.97 m**	**Decrease (%)**	**H=18.54 m**	**Decrease (%)**
At elastic limit	**Original**	**720**	0.130	-	0.065	-	0.040	-
	- 200 mm	**520**	0.110	15.380	0.058	10.770	0.037	7.500
	- 400 mm	**320**	0.085	34.610	0.048	26.150	0.033	17.500
Ultimate	**Original**	**720**	0.160	-	0.090	-	0.055	-
	- 200 mm	**520**	0.140	12.500	0.079	12.222	0.050	9.091
	- 400 mm	**320**	0.117	26.875	0.062	31.111	0.043	21.818

From Table 5, the ultimate base shear decreases as the thickness of walls of central masonry core decreases, but particularly decreases in the case of increase of the total height (see Table 4). In particular, for the case of the Gopinath temple with fixed values of thickness at the ground floor (see Table 4), the total base shear at ultimate decreases from 43% to 48 % when one floor was added and from 63% to 68 % when two floors were added. In addition, in the case of temples with fixed heights (see Table 5), the base shear drop ranges between 9% and 13 % when the thickness of the walls was reduced by 200 mm and from 21% to 32 % when the reduction was 400 mm.

Regarding variation of the total height, similar trend can be observed in Table 6, but oppositely with respect to the first group of temples (if the height is reduced then the ultimate base shear increases). However, for the second group of temples, thickness reduction has beneficiary effect on the ultimate base shear expressed as a function of gravity acceleration (g). The reason is that structural weight becomes strongly reduced which has a more dominant influence on the growth of the ration V/W (see Table 7).

Table 6 shows that for Nyatopol temple with fixed values of wall thickness at the ground floor, the total base shear at ultimate increases from 26% to 103 %, if the two last floors were removed. Also, when three floors removed then the total base shear at ultimate increases from 72% to 161 %. If the heights were kept constant (see Table 7), and the thickness of the wall is equal to 500 mm, then the total base shear at ultimate ranges from 4% to 12 %, while when the wall thickness is equal to 700 mm, the total base shear decreases from 12% to 26 %.

Table 6 - Comparison between total base shear at elastic limit and at ultimate for fixed wall thickness (d) and variation of total height (H) for Nyatopol temple (increase (%) of base shears is given with respect to the structure with original thickness)

	V/W = base shear/structural weight (g)							
	Height	**H (m)**	**d=1590 mm**	**Increase (%)**	**d=1090 mm**	**Increase (%)**	**d=890 mm**	**Increase (%)**
At elastic limit	**Original**	**H=21.93**	0.058	-	0.040	-	0.035	-
	- 2 floors	**H=15.65**	0.080	37.930	0.070	75.000	0.065	85.710
	- 3 floors	**H=11.32**	0.110	89.650	0.100	150.000	0.090	157.140
Ultimate	**Original**	**H=21.93**	0.093	-	0.082	-	0.069	-
	- 2 floors	**H=15.65**	0.118	26.882	0.123	50.000	0.140	102.899
	- 3 floors	**H=11.32**	0.160	72.430	0.173	110.976	0.180	160.869

Table 7 - Comparison between total base shear at elastic limit and at ultimate for fixed total height (H) and variation of wall thickness (d) for Nyatopol temple (increase and decrease (%) of base shears is given with respect to the structure with original thickness)

	V/W = base shear/structural weight (g)							
	Thickness	**d (mm)**	**H=21.93 m**	**Decrease (%)**	**H=15.65 m**	**Increase (%)**	**H=11.32 m**	**Increase (%)**
At elastic limit	**Original**	**1590**	0.058	-	0.080	-	0.110	-
	- 500 mm	**1090**	0.040	31.030	0.070	12.500	0.100	9.100
	- 700 mm	**890**	0.035	39.660	0.065	18.750	0.090	18.180
Ultimate	**Original**	**1590**	0.093	-	0.118	-	0.160	-
	- 500 mm	**1090**	0.082	11.828	0.123	4.237	0.173	8.125
	- 700 mm	**890**	0.069	25.806	0.140	18.644	0.180	12.500

5. Nonlinear dynamic analysis (NLDA)

Time-history nonlinear analyses were performed using spectrum compatible for the location of Kathmandu Valley accelerogram as shown in Figure 14. This spectrum compatible accelerogram was used mainly for comparative reasons and to assess the damage and preliminary load carrying capacity evaluation of the temples under consideration. Points denoted in the accelerogram are defined in the Table 18.

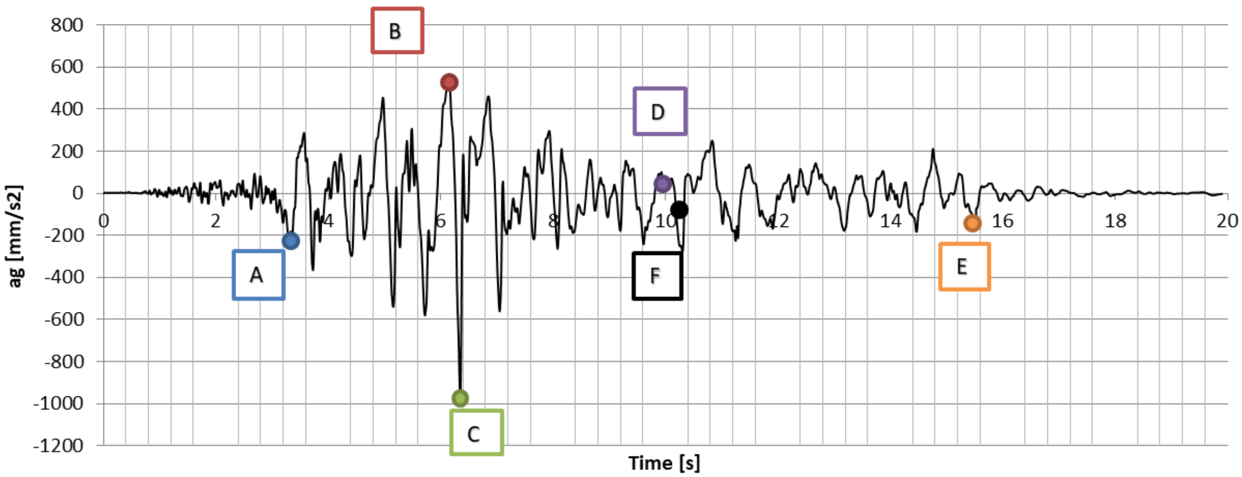


Figure 14 –Spectrum compatible accelerogram PGA=0.1 g

Table 18 – Definition of points on the spectrum compatible accelerogram

Point	Description	Time (s)
A	The first significant peak	3.36
B	Maximum positive peak	6.14
C	Maximum negative peak	6.35
D	Middle point of the record	10
E	The last significant peak	15.48

In the NLDA, the peak ground acceleration has been take as 0.1g for all 18 structures in each group of temples in order to compare tensile damages. In other words, the analysis was related to comparative damage analysis of two groups of temples with different dynamic responses under the uniform seismic recording.

Figure 15 shows the tensile damage patterns and time history of relative drift for the first group of temples. Relative drift has been defined as the displacement normalized with the total height, expressed in percentages. When drift at the top of the structure is greater than 0.8 %, the structure fails. From Figure 15, it can be observed that for the original height which is equal to 11.40 m, damages are dominantly concentrated along the perimeter of the central masonry core at the level of the second floor, independently of the thickness of the walls. Nonetheless, if the thickness decreases, damages with lower intensity occur diagonally, starting from the window opening corners, especially when the thickness of the wall is equal to 320 mm.

For the structures with the height equal to 14.97 m, or in other words when one floor was added in the original geometry of the temple, damage patterns are to the great extent similar to the ones observed in the group of structures with the height equal to 11.40 m. Besides these, there are inclined damages in the case with the thickness of 320 mm which corresponds to transition from shear failure to flexural failure modes.

In the case when the height of the temple was equal to 18.54 m, damages migrate at the level of the lowest window openings. Cracks start from the end of the window openings and propagate to the top, allowing the upper part of the temple to overturn. The two temples with 18.54 m height experience incipient collapse (drift surpasses 0.8 % of the total height) with overturning of the tower-like upper part, which can be observed in the final row of Figure 15. Also, from Figure 15 it is observed a similar progressive change of failure mechanism as with the pushover analysis, for the same group of temples studied, with the important difference related to the position of concentrated damages. Evidently, NLDA shows that plastic hinges for the most slender Gopinath temples occur at the level of the lowest set of window openings, while in pushover analysis overturning takes place around the hinge at the level of second floor.

Figure 16 shows that in stockier structures of the second group of temples, various damage patterns that precede shear failures of entire structural parts (exterior walls and walls of central masonry core) are visible. However, Nyatopol temple with original height which is equal to 21.93 m is characterized by vertical misalignments and thickness reduction of walls of central masonry core at each floor. These are the reasons for these zones to be the weakest in the structure, which can be justified observing concentration of damages (see the first row of Figure 16). However, certain closed crack patterns in the shape of ellipse are visible from the side perpendicular to ground motion direction. That is the consequence of oscillations of the wall free surface which is far from its constraint zones. These cracks are particularly pronounced in the case of the smallest thickness of the base wall which equals 890 mm. This is in opposition with the damage maps obtained by pushover analysis, which forces flexural failure at each level. The reason is irregular pyramidal configuration of the structures with progressive reduction of cross section along the height. Such irregular configuration induces more shear resistance in its behavior than regular slender configuration leading to shear failures of certain structural parts.

Consequently, unlike towers with simple and regular geometry, seismic performance assessment of Nepalese temples with numerous irregularities cannot be evaluated by applying pushover analysis, hence more accurate NLDA is necessary for the evaluation of the dynamic behavior of such temples. This comparative damage analysis shows that one has to be particularly careful while choosing analysis method in order to obtain convenient results. Finally, reliable results are of the utmost importance for implementing intervention strategies which are essential for the preservation of such invaluable built cultural heritage.

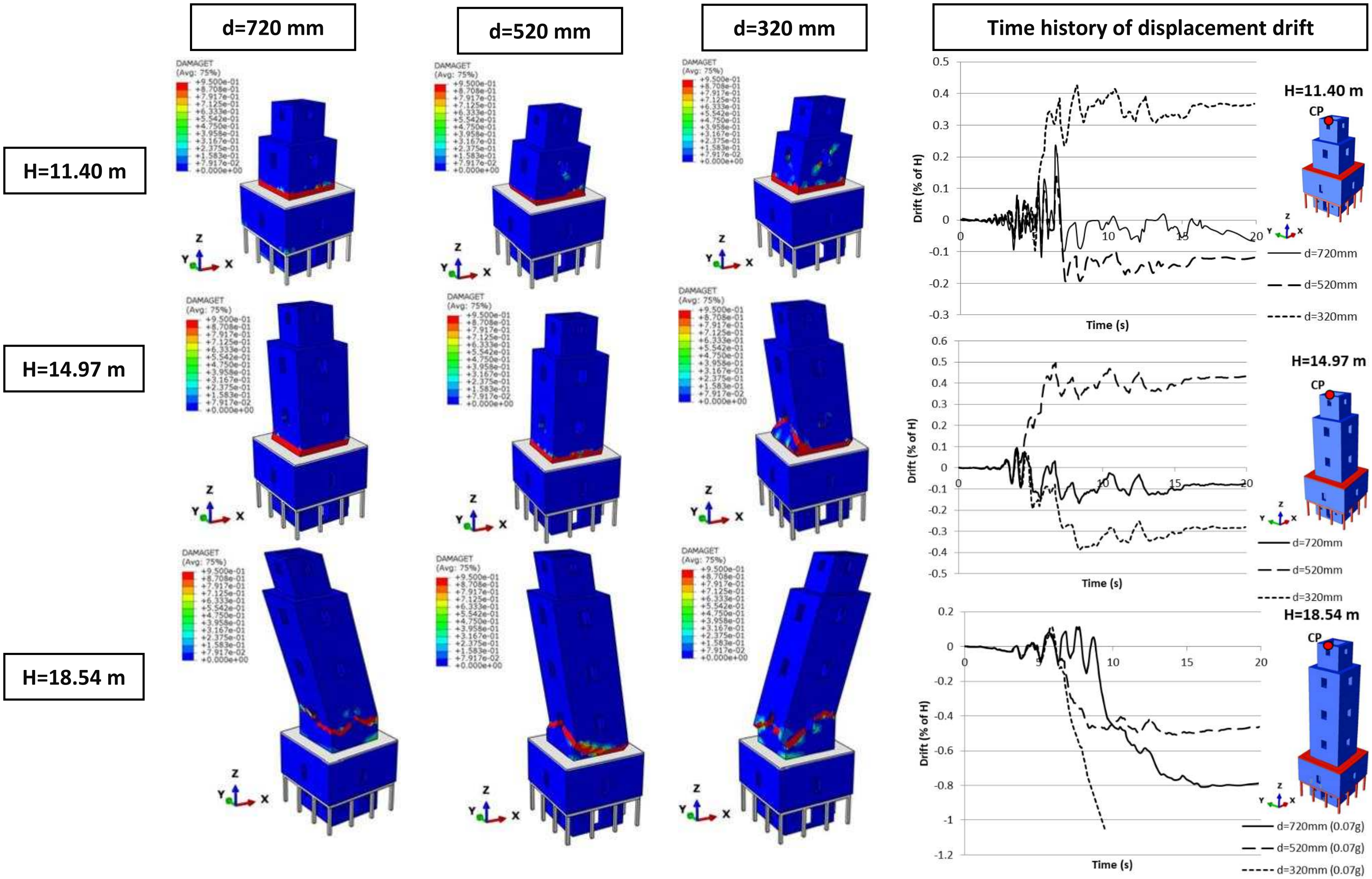


Figure 15 – Final tensile damage patterns and time history of displacement drift of the control points for Gopinath temple with varied thickness d=720 mm, d=520 mm and d=320 mm at the ground floor and varied height H=11.40 m, H= 14.97 m and H=18.54 m

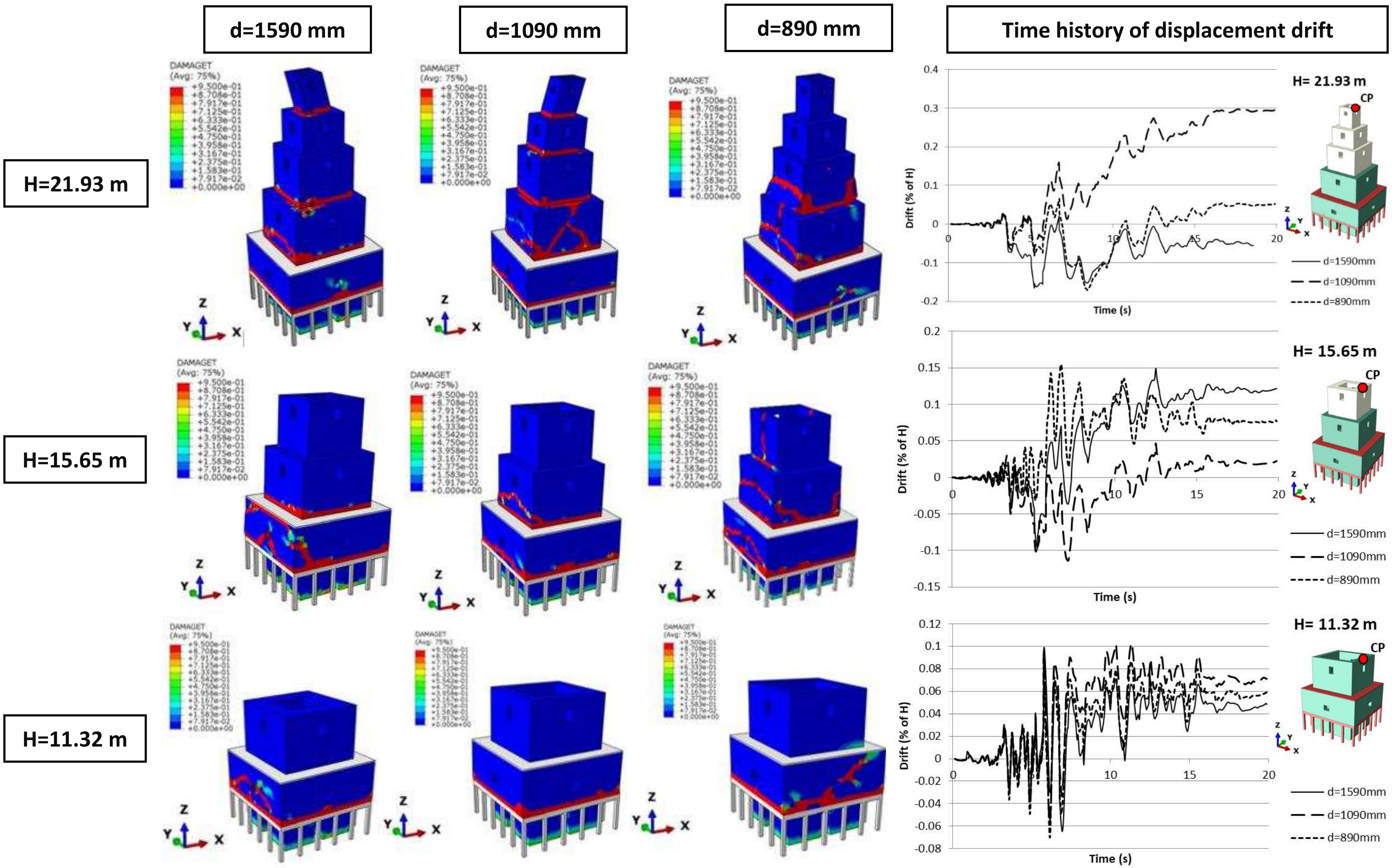


Figure 16 – Final tensile damage patterns and time history of displacement drift of the control points for Nyatopol temple with varied thickness d=1590 mm, d=1090 mm and d=890 mm at the ground floor and varied height H=21.93 m, H= 15.65 m and H=11.32 m

A comparative analysis was performed assuming stiff and flexible floors at second floor. Two extreme examples in terms of global structural slenderness investigated: a) Gopinath temple with height equal to 18.54 m; and b) Nyatopol temple with 11.32 m of the height with varied thickness. The main goal was to prove that failure mechanism which occurs in slender temples does not depend on the aforementioned assumption. The reason is the fact that tensile damages are concentrated at the section which is weakened by the window openings and that the structure has characteristic flexural failure mechanism.

Additionally, in the case of stocky temples, although a complex 3D behavior observed, flexural failure accompanied with plastic hinges didn't occur. In particular, temples failed due to the overturning of the exterior walls around their base, which is clearly visible in tensile damage contour plots and displacement drift for control points at the top of external walls (see Figure 17).

For the temples with height equal to 18.54 m, damages were dominantly concentrated along the perimeter of the central masonry core, forming a flexural failure mechanism for all three different thicknesses investigated. Additionally, incipient collapse situation was reached, which was visible by looking at the amount of drift at the control point at the top of the temples. For the temples with thickness of the walls (d) equal to 720 mm, cracks initiated at the lowest set of window openings, while at the the second floor there was a slight indication of damage. Low-intensity damages can be observed at re-entrant corners of the lowest windows. In the case of wall thickness equal to 520 mm, crack pattern was partially flat over the second floor onset and started to be inclined towards the corners of the window opening. At windows corners, there were significant cracks which propagated to the corners of the masonry core. When the thickness of the walls reduced to 320 mm, damage became more diffused in the wall, which also encompass the lowest window openings. Additionally, the prevailing failure mechanism is the flexural one.

The assumption of box-like behavior encompasses stiff floor assumption realized by means of wooden laces strengthened with metal plates at the corners of the external walls at the top of first floor. However, the lack of the constraints was also probable if strengthening measures were not already applied. In that case, another modeling approach was applied, which rendered out-of-plane overturning of external walls, especially when stocky structures were at stake. In other words, when slenderness was higher (e.g. Gopinath temple with H=18.54 m) the structure behaved similarly to towers (flexural hinge occurs), while when the slenderness was smaller (e.g. Nyatopol temple with H=11.32 m) the structure behaved in a more complex way and, if assuming lack of box-like behavior constraints regarding external walls, collapse mechanisms include out-of-plane overturning of external walls perpendicular to ground motion actions.

In the case of Nyatopol temple with height equal to 11.32 m, tensile damage pattern at incipient collapse situation was related to overturning of external walls (see Figure 17). The loss of interlocking between perpendicular walls was evident at both sides parallel to ground motion direction, while rest of damages are concentrated at the beginning of the first floor all around the central masonry core.

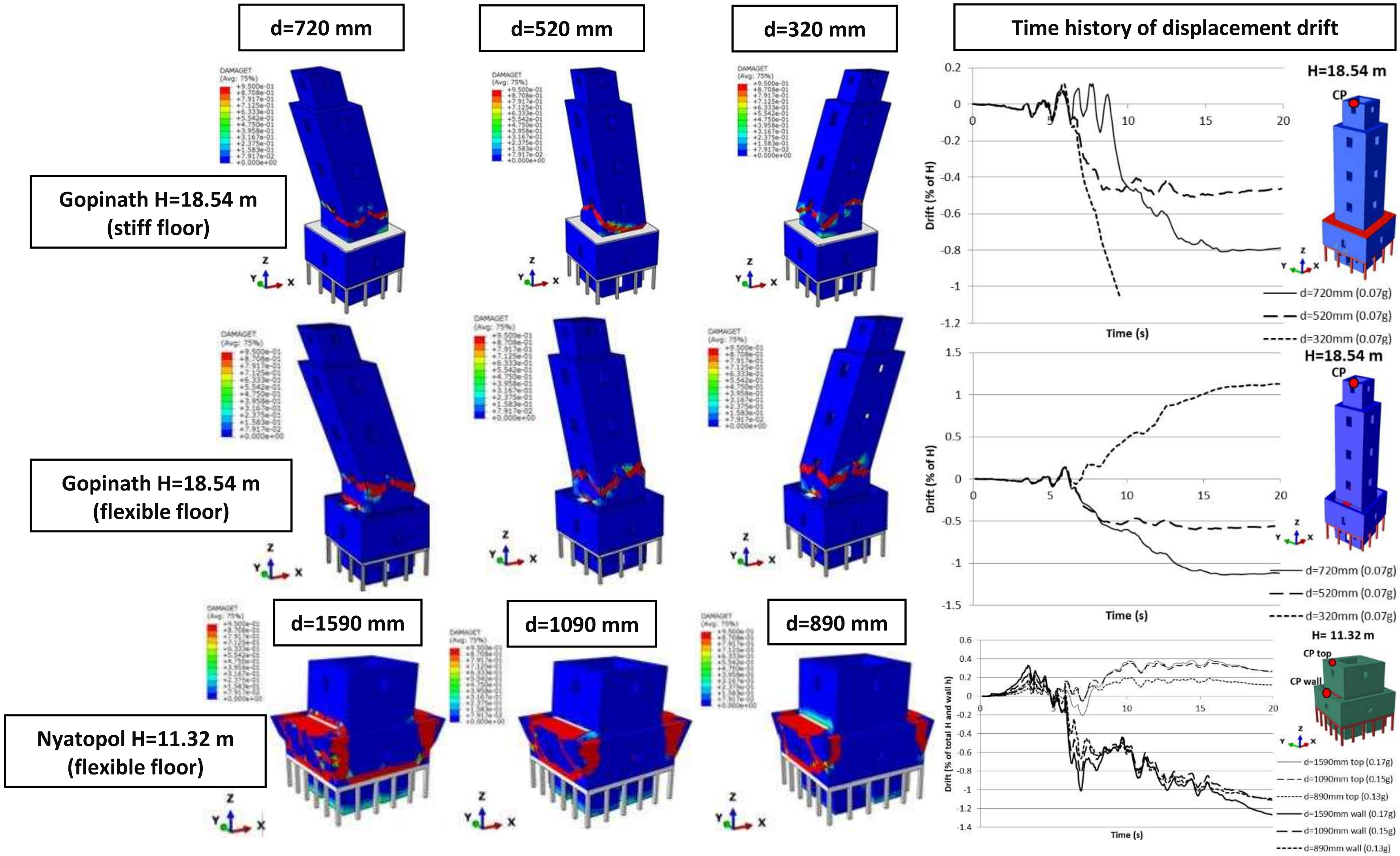


Figure 17 - Final tensile damage patterns and time history of relative drift of the control points for Gopinath temple with H=18.45 m and varied floor stiffness assumption and Nyatopol temple with H= 11.32 m with flexible floor assumption

Conclusions

In this study, the seismic performance of old multi-tiered temples in Nepal were assessed using different computational approaches including: a) linear; b) nonlinear static; and c) nonlinear dynamic analyses. A sensitivity study was also undertaken to investigate the influence of wall thickness and height of Nepalese temples on their seismic behavior. A series of macro-models based on the finite element method of analysis were developed. Masonry has been modeled based on the Concrete Damaged Plasticity Model (CDPM) available in ABAQUS. C3D4 tetrahedron linear type elements were used, due to the strong reduction of computational effort comparing to those with parabolic shape functions. In order to characterize the material properties of the temples, results from modal analysis were used. From the modal analysis results, it was found that the first periods are approximately in the range between 0.5s - 0.6s. Effective modal masses are dominant in global modes, while their values in the case of local modes are very small. The exceptions are modes in Z direction, which can be an indicator for local failure mechanism related to upper masonry tower eccentrically positioned with respect to central masonry core.

Comparative damage analysis shows that the selection of the suitable approach for modelling such structures is critical to avoid misleading information about their seismic performance. NLDA shows that pyramidal configuration with progressive reduction of cross section at each floor activates more shear resistance in its behavior than the flexural one. Application of nonlinear static analysis thus is not justified since its results are in principle more oriented to flexural than to shear behavior, which is not in correspondence with the most accurate NLDA for this type of structures. Correct assessment of failure modes is essential for proper design of intervention strategy which is necessary to be done for many temples.

Under the assumption of stiff floor, failure mechanisms do not change in the case of the most slender temples among all investigated (Gopinath temples with H=18.54 m with varied thickness) comparing to the case of flexible floor. In the case of the stockiest temples (Nyatopol temples with H=11.32 m) the failure mechanism is reached only when flexible floor is assumed. Being closer to reality, it is evident that overturning of the external walls takes place.

Strengthening measures are necessary to be applied in order to increase load carrying capacity and enhance dynamic behavior of the structures. Adequate confining measures of central core walls have to be applied in order to reduce cutting and cutting-compression damages under horizontal seismic actions. Timber connections have to be strengthened in order to ensure better constraining effects between central masonry core and external walls, thus eliminating out-of-plane overturning of the external walls and loss of interlocking between two perpendicular external walls.

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References

[1] A. M. D’Altri, G. Castellazzi and S. Miranda, "Collapse investigation of the Arquata del Tronto medieval fortress after the 2016 Central Italy seismic sequence," *Journal of Building Engineering 18 (2018) 245–251.*

[2] J. Bijaya, R. Wei-Xin, Z. Zhou-Hong and M. Prem Nath, "Dynamic and seismic performance of old multi-tiered temples in Nepal," *Engineering Structures 25 (2003) 1827–1839,* 11 August 2003.

[3] A. Arce, P. Awal and A. Tamrakar, "Structural analysis and strengthening proposals for the Gopinath temple in Kathamndu Durbar Square," January 2018.

[4] C. Bonapace and V. Sestini, "Traditional Materials and Construction Technologies used in the Kathmandu Valley.," *United Nations Educational, Paris, France.,* 2003.

[5] W. Korn, The Traditional Architecture of the Kathmandu Valley. Ratna Pustak Bandhar, Kathmandu,, 1979, p. 104..

[6] S. Rashmi, K. S. Jagadish and S. Nethravathi, "Stabilized Mud Mortar," *International Journal of Research in Engineering and Technology eISSN: 2319-1163, pISSN: 2321-7308,,* vol. 3, May, 2014.

[7] S. Nienhuys, "Options for reconstruction and retrofitting of historic Pagoda temples, reconstruction of temples in Kathmandu.," HUYS ADVIES, Kathmandu, 2003.

[8] D. Sonda, D. Bothara, C. Chesi, V. Sumini and e. al, "ANALYSIS OF THE SEISMIC PERFORMANCE OF A STRENGTHENED PAGODA TEMPLE DURING GORKHA EARTHQUAKE," in *16th World Conference on Earthquake, 16WCEE 2017*, Santiago Chile, January 9th to 13th 2017.

[9] M. Shakyaa, H. Varuma, R. Vicentea and A. Costaa, "Structural Vulnerability of Nepalese Pagoda Temples," in *15 WCEE Lisaboa*, Civil Engineering Department, University of Aveiro, Portugal, 2012.

[10] S. Tiwari, Temples of the Nepal Valley, Kathmandu, Nepal: Himal Books. ISBN: 9789937814430, 2009.

[11] S. Suraj, "Performance of Cultural Heritage Structures, Nepal Earthquake and Aftershocks," EERI LFE Briefing, April 25, 2015 .

[12] S. Nienhuys, "Options for Reconstruction and retrofitting of Historic Pagoda Temples," HUYS ADVIES, Kathmandu, September 2003.

[13] R. Pradhan, "Seismicity and traditional buildings of Kathmandu Valley, Nepal.," in *ICOMOS international wood committee (IIWC), Earthquake-safe: lessons to be learned from traditional construction, international conference on the seismic performance of traditional buildings*, Istanbul, Turkey., 2000.

[14] E. Theophile and R. Ranjitkar, "Timber conservation problems of the Nepalese Pagoda temple.," in *ICOMOS international wood committee, 8th international symposium,*, Norway, 85–124., 1992.

[15] M. Valente and G. Milani, "Damage assessment and collapse investigation of three historical masonry palaces under seismic actions," *Engineering Failure Analysis 98 (2019) 10–37.*

[16] M. Shakya, "Seismic vulnerability assessment of slender masonry structures," *PhD thesis,* July 2014.

[17] R. L. Wood, M. E. Mohammadi, A. R. Barbosa, C. K. Kawan, M. Shakya and M. J. Olsen, "Structural Damage Assessment of a Five Tiered Pagoda Style Temple in Nepal," in *16th World Conference on Earthquake, 16WCEE 2017*, Santiago Chile, January 9th to 13th 2017.

[18] Y. Parajuli, "Bhaktapur development project (BDP): experiences in preservation and restoration in medieval town.," Deutsche Gesellschaft fuer Technisches Zuzammenarbeit (GTZ),, Frankfurt, 1986.

[19] R. Salvatore, B. Giosuè and S. Eleonora, "Surveys on the Jagannath temple's seismic response," *Int. J. Masonry Research and Innovation,* vol. Vol. 3, no. No. 4, 2018.

[20] E. Miranda, "Assessment of the vulnerability of existing buildings," in *Eleventh World Conference on Earthquake Engineering*, 1996.

[21] P. Fajfar and M. Fischinger, "N2-A METHOD FOR NON-LINEAR SEISMIC ANALYSIS OF REGULAR BUILDINGS," in *Proceedings of Ninth World Conference on Earthquake Engineering*, 1988.

[22] F. Clementi, V. Gazzani, M. Poiani and M. Lenci, "Assessment of seismic behaviour of heritage masonry buildings using numerical modelling," *Journal of Building Engineering 8,* p. 29–47, 2016.

[23] B. Pulatsu, V. Sarhosis, E. Bretas and N. Nikitas, "Non-linear static behaviour of ancient free-standing stone columns," *ICE Proceedings Structures and Buildings · February 2017.*

[24] P. Grassl, D. Xenos, U. Nystrom, R. Rempling and K. Gylltoft, "CDPM2: A damage-plasticity approach to modelling the failure of concrete," 26 Jul 2013.

[25] SIMULIA, Abaqus 6.11 Theory Manual.

[26] G. Milani and M. Valente, "Failure analysis of seven masonry churches severely damaged during the 2012 Emilia-Romagna (Italy) earthquake: Non-linear dynamic analyses vs conventional static approaches," *Engineering Failure Analysis 54 (2015) 13–56,* 31 March 2015.

[27] " Direttiva del Presidente del Consiglio dei Ministri - Valutazione e riduzione del rischio sismico del patrimonio culturale con riferimento alle Norme techice per le construzioni di cui al D.M. 14/01/2018," p. 29, 9 Febbraio 2011.

[28] M. Valente, G. Barbieri and L. Biolzi, "Seismic assessment of two masonry Baroque churches damaged by the 2012 Emilia earthquake," *Engineering Failure Analysis 79 (2017) 773–802.*

[29] G. Brandonisio, G. Lucibello, E. Mele and A. De Luca, "Damage and performance evaluation of masonry churches in the 2009 L'Aquila earthquake," *Engineering Failure Analysis 34 (2013) 693–714.*

[30] M. Betti, A. Borghini, S. Boschi, A. Ciavattone and A. Vignoli, "Comparative Seismic Risk Assessment of Basilica-type Churches," *Journal of Earthquake Engineering, 22:sup1, 62-95, DOI: 10.1080/13632469.2017.1309602,* 2018.

[31] G. Milani and M. Valente, "Damage survey, simplified assessment, and advanced seismic analyses of two masonry churches after the 2012 Emilia earthquake," *International Journal of Architectural Heritage, DOI: 10.1080/15583058.2018.1492646,* 2018.

[32] G. Milani and G. Venturini, "Automatic fragility curve evaluation of masonry churches accounting for partial collapses by means of 3D FE homogenized limit analysis," *Computers and Structures 89 (2011) 1628–1648,* 2011.

[33] G. Milani, R. Shehu and M. Valente, "A kinematic limit analysis approach for seismic retrofitting of masonry towers through steel tie-rods," *Engineering Structures 160 (2018) 212–228,* 2018.

[34] V. Sarhosis, G. Milani, A. Formisano and F. Fabbrocino, "Evaluation of different approaches for the estimation of the seismic vulnerability of masonry towers," *Bull Earthquake Eng (2018) 16:1511–1545.*

[35] M. Bocciarelli and G. Barbieri, "A numerical procedure for the pushover analysis of masonry towers," *Soil Dynamics and Earthquake Engineering 93(2017)162–171.*

[36] G. Castellazzi, A. M. D'Altri, S. Miranda, A. Chiozzi and A. Tralli, "Numerical insights on the seismic behavior of a non-isolated historical masonry tower," *Bull Earthquake Eng (2018) 16:933–961.*

[37] M. Valente and G. Milani, "Damage assessment and partial failure mechanisms activation of historical masonry churches under seismic actions: Three case studies in Mantua," *Engineering Failure Analysis 92 (2018) 495–519.*

[38] M. Valente and G. Milani, "Effects of Geometrical Features on the Seismic Response of Historical Masonry Towers," *Journal of Earthquake Engineering, 22:sup1, 2-34, DOI: 10.1080/13632469.2016.1277438,* 2018.

[39] F. Clementi, A. Ferrante, E. Giordano, F. Dubois and S. Lenci, "Damage assessment of ancient masonry churches stroked by the Central Italy Earthquakes of 2016 by the Non-Smooth Contact Dynamics method," *Bulletin of Earthquake Engineering, DOI: 10.1007/s10518-019-00613-4.,* 2019.

[40] A. Ferrante, F. Clementi and G. Milani, "Dynamic behavior of an inclined existing masonry tower in Italy," *Frontiers in Built Environment – Earthquake Engineering, 5:33, DOI: 10.3389/fbuil.2019.00033.,* 2019.

[41] F. Clementi, V. Gazzani, M. Poiani, P. A. Mezzapelle and S. Lenci, "Seismic Assessment of a Monumental Building through Nonlinear Analyses of a 3D Solid Model," *Journal of Earthquake Engineering ,* 2018.

[42] F. Clementi, E. Quagliarini, F. Monni, E. Giordano and S. Lenci, "Cultural Heritage and Earthquake: The Case Study of “Santa Maria Della Carità” in Ascoli Piceno," *The Open Civil Engineering Journal, 11(Suppl-5, M5):1079–1105, DOI:10.2174/1874149501711011079. ,* 2017.