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ORIGINAL RESEARCH PAPER

Seismic performance of the infill masonry walls and ambient vibration tests after the Ghorka 2015, Nepal earthquake

Humberto Varum[1] · André Furtado[1] · Hugo Rodrigues[2] ·
João Dias-Oliveira[1] · Nelson Vila-Pouca[1] · António Arêde[1]

Abstract Recent earthquakes have demonstrated a significant contribution of the masonry infill walls in the structural response of the reinforced concrete (RC) buildings. Field observations after the 25th April's earthquake in Nepal reinforce the conclusion regarding the influence of the infill walls increasing significantly the structural stiffness, which has a direct impact on the natural frequencies of the structure and of structural elements. Firstly the manuscript focuses on the performance of the infilled RC structures, describing common Nepalese architectural configurations and the major damages observed in infilled RC buildings. Secondly, it shows results from data collected on ambient vibration tests performed in seven infill panels with different characteristics, such as geometric dimensions, openings and levels of damage. These results are used to study the influence of each parameter in the out-of-plane frequency of the wall panels. The present studies along the manuscript are followed by a presentation of the study cases (buildings and the walls), test setups, main results and conclusions.

✉ Hugo Rodrigues
  hugo.f.rodrigues@ipleiria.pt

  Humberto Varum
  hvarum@fe.up.pt

  André Furtado
  afurtado@fe.up.pt

  João Dias-Oliveira
  j.diasoliveira@fe.up.pt

  Nelson Vila-Pouca
  nelsonvp@fe.up.pt

  António Arêde
  aarede@f.up.pt

1 CONSTRUCT-LESE, Faculty of Engineering (FEUP), University of Porto, Porto, Portugal

2 RISCO, School of Technology and Management, Polytechnic Institute of Leiria, Leiria, Portugal

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1 Introduction

The April 25th of 2015 the massive earthquake with the magnitude of 7.8M$_w$, with the epicenter located in Barpark, Ghorkha's district (Fig. 1a), 75 km to the northwest of Kathmandu, and a focus depth of 8.2 km. It caused around 9000 deaths, 23,000 injuries and thousands of collapsed buildings. According to geologic reports, the earthquake occurred at the subduction of Main Himalayan Thrust, the main mega-thrust fault along northern India is pushing beneath Eurasia (Avouac et al. 2015), which caused also more than 150 deaths and 200 injured in some areas of India, China, and Bangladesh. The KATNP station (27.7N, 85.3E) recorded the ground motion, and it was measured the horizontal peak ground acceleration of 0.164 and 0.158 g along the north–south and east–west direction respectively (Fig. 1b, c). The corresponding response spectra of both components are illustrated in Fig. 2, where it is possible to observe that in the period range between 0.1 and 1 s the spectral acceleration exceeded 0.6 g, which was critical for the low-rise infilled RC buildings that have similar natural period between those range time.

On May 12th of 2015, another powerful earthquake struck with a magnitude of 7.3M$_w$. This occurred in the district of Dolakha (Fig. 3a) with a focus depth of 18 km located 75 km to the northeast of Kathmandu. The powerful shock caused around 200 deaths and 2500 injuries, and increased the previous number of collapses of buildings. More than 400
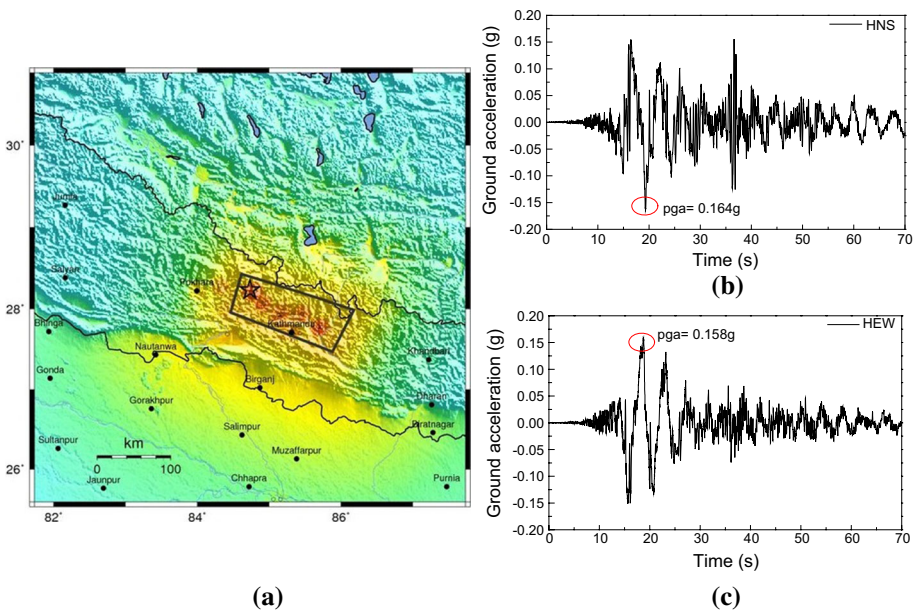

Fig. 1 Shake intensity map of **a** 25th April of 2015; Acceleration of the main shock for **b** north–south (HNS); and **c** east–west direction (HEW), according to USGS (USGS 2015)

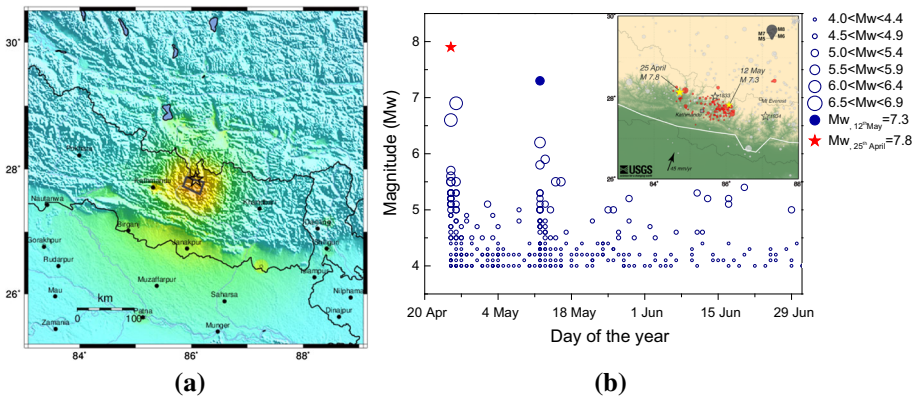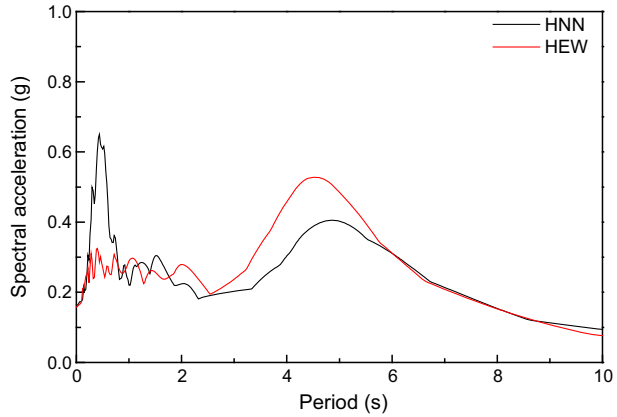**Fig. 2** Response spectra of the 25th April of 2015 ground motion

Fig. 3 Ghorka earthquake seismic sequence **a** Shake intensity map of 12th May of 2015; and **b** distribution of the aftershocks with magnitude higher than 4Mw (adapted from (USGS 2015))

aftershocks, with magnitude larger than 4Mw including another one of 6.6Mw in Ghorka district, were registered during the following 2 months, as it can be observed in Fig. 3b.

According to the most recent updates, over than 500,000 houses were fully destroyed and about 270,000 houses were partially destroyed, leaving homeless hundreds of thousands of people. A significant number of centuries-old buildings were destroyed at UNESCO World Heritage sites causing a strong socio-economic impact in Nepal. Some studies were performed during the last year focused on the seismic loss estimation for the region of Kathmandu Valley, which concluded that the RC buildings built before the introduction of the Nepal Building Code have higher vulnerabilities, alerting for the need of mitigation actions (Chaulagain et al. 2015a; b).

The main purpose of the present research manuscript is to present a survey report of the infill masonry (IM) walls' performance and influence on the seismic response of buildings in Nepal. For this, some considerations regarding the infill materials properties, infills typologies and construction methodology are given, as well as a complete damage report performed to RC buildings is described along the manuscript. Finally, findings are presented regarding ambient vibration tests performed in IM walls with different characteristics.

2 Infill masonry walls influence in the seismic response of RC structures during the Ghorka earthquake

As observed in recent earthquakes during the last years, the performance of the RC structures is conditioned by the IM walls presence, in particular by their distribution, and material main particularities (Romão et al. 2013; Vicente et al. 2012; Hermanns et al. 2014; De Luca et al. 2014). This catastrophic event proved again that the IM walls have an important contribution on the structural response of existing RC buildings in Nepal.

The used materials to construct the infill walls are a lot alike the ones used in Europe. The bricks are solid instead of hollow, and exterior walls are composed by double-leafs linked by alternative disposition of the bricks, instead of having an interior gap. Generally, damage observations showed that regular distribution of IM walls in height, provided a beneficial structural response, justified by the increase of the structural stiffness (which sometimes is not favorable to the structure, depending on some structural issues) and strength provided this non-structural elements. Nonetheless, in situations where this assumption was not verified, it was observed a poor response of the structures leading to a significant number of damages or collapse. Different types of failure mechanisms were observed at the local and global level. The local failure mechanisms are composed by the detachment between the IM wall and the surrounding RC frame, diagonal cracking, shear failure, and out-of-plane failure (although of few examples observed of this type of damage). The global mechanisms observed are soft-storey and short-column mechanisms.

Particularly focused on the performance of the infilled RC buildings, several research works have been performed that highlight the contribution of the IM walls in the structural response of existing buildings, particularly the interaction with the surrounding RC frames (Asteris et al. 2011, 2013; Furtado et al. 2015a; Rodrigues et al. 2010; Asteris et al. 2012). As performed by others studies (Furtado et al. 2015b; Silva et al. 2015; Bal et al. 2008), the risk mitigation research applied particularly to Nepal is an important contribute to the improvement of the global behavior of the buildings and decrease of the IM walls damages.

Throughout this section it is summarized the different buildings typologies of RC buildings with IM walls that were observed in the region of Kathmandu and Bakthapur, as well as a brief considerations regarding to the materials used in the construction of the IM walls. The local and global damages and failure mechanisms observed in the RC buildings subjected to earthquakes are reported as well along the section.

2.1 IM walls in Nepal: general considerations

In the region of Kathmandu and Bakthapur the RC buildings are usually between 3–4 (low rise buildings), 4–6 storeys (medium rise buildings) and in isolated locations and considerably smaller number it was found some high rise buildings which clearly stand out from the surrounding buildings. In Fig. 4 it is observed four typical buildings according to the occupations (a) regular residential, (b) regular residential buildings with commercial stores in the ground level, (c) high rise residential buildings and (d) Schools or hospital buildings with large implantation dimensions.

The masonry infill walls are usually composed by solid bricks with traditional cement mortar. These types of bricks are very stiff providing a significant increase of the global stiffness. The construction process of the walls according to local's starts with the placement two rows of bricks disposed alongside in the longitudinal way, alternated by a

Fig. 4 RC building typologies in Nepal: **a** residential buildings, **b** residential and commercial buildings, **c** residential buildings and **d** secondary school

Fig. 5 Typical IM walls construction process in Nepal

row of bricks disposed on the transversal. In some cases, it was observed the existence of RC lintels with thickness around 5–7.5 cm, built at half-height of the panels or in the bottom/top of the openings as can illustrated in Figs. 5 and 6. The longitudinal reinforcement detailing of the lintels is usually composed by 206 mm, and a poor concrete quality.

It was observed some cases of IM walls corners without RC columns on buildings boundaries or balconies (Fig. 7).

Fig. 6 Detail of lintel constructed in the top and bottom of the IM wall window

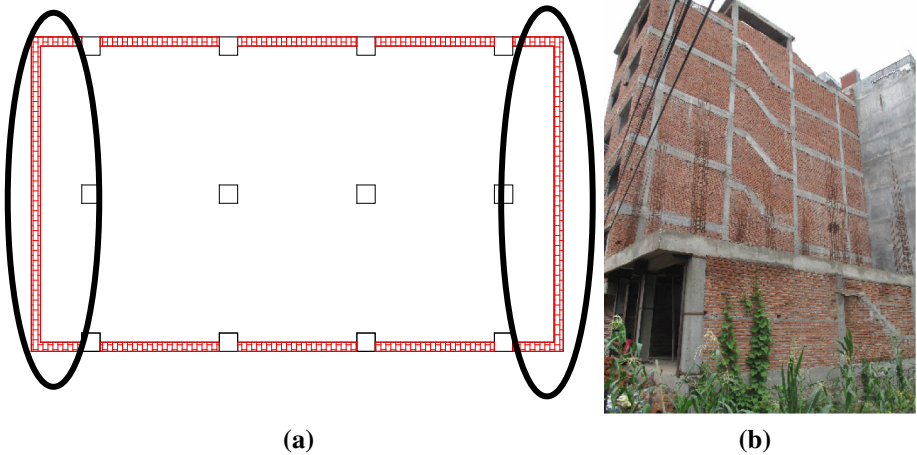**(a)**   **(b)**

Fig. 7 Six storey building with IM wall corners with no RC column on the building boundaries

Regarding the IM walls, several typologies were found according to the existence of openings, the disposition and the size of the openings. Most part of the infill panels observed were composed by: (1) full panels without openings, (2) panels with a central window placed in the middle of the wall or next to the columns and (3) large door in the middle of the wall in the case of existence of balconies (Fig. 8).

2.2 IM walls failure modes

In the following sub-sections, local and global failure mechanisms observed in buildings affected by the last earthquake in Nepal.

2.2.1 Local failures/mechanisms

One of the most common damages observed was the detachment of the IM panel from the surrounding RC structure (Fig. 9), which occurred in interior and exterior partition walls, with and without openings. This type of damage demonstrates the real behaviour of the

Fig. 8 IM walls typologies most common in Nepal

(a)   **(b)**

Fig. 9 Detachment of the IM panel from the surrounding RC structural elements by the **a** Exterior IM walls from a 16 storey building façade, and **b** Interior partition wall

structure, since there is no connection between the wall panel and the surrounding panel. The joints interfaces when reach certain level of in-plane drift demand tens to reduce/eliminate the monolithic behaviour by transforming this into two independents elements behaviour (RC frame and IM panel).

After the detachment of the panels, several local failures can occur, namely the diagonal cracking, caused by tensions stresses along the perpendicular direction of the diagonal compressive strut (Fig. 10).

The shear-friction failure was also observed, namely caused by the shear forces that tended to separate the behaviour of the IM wall in two distinct parts. According to the observation of damages it was possible to assess that due to the high stiffness of such non-structural elements, their deformation was non uniform in height which lead with the shear cracking of the IM wall (Fig. 11).

The accumulation of stress in the top and bottom corners leads to some general cracking's and plaster detachment, as observed in Fig. 12.

The out-of-plane collapse of the IM walls was not observed in the majority of the buildings, because of the low in-plane strength degradation and strong arching mechanism.

Fig. 10 Diagonal cracking of the IM walls in a 16 storey high-rise building façade

Fig. 11 Shear friction failure observed in interior partition walls of low and medium-rise residential buildings

Fig. 12 Stress accumulation in the top and bottom corners observed in interior partition walls of a high-rise residential building
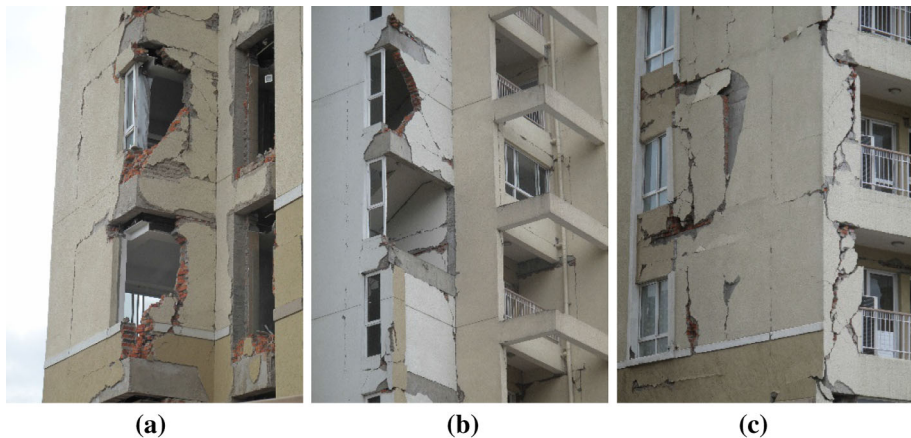
Fig. 13 Out-of-plane collapses of IM walls on **a**, **b** Corners and **c** Confined

Since no brick crushing occurs, the volumetry of the infill and bare frame locks together creating a limitation of flexural deformation, which do not allow the out-of-plane return.

These exterior partition walls placed when subjected out-of-plane loadings they tends to separate of the surrounding frame as a rigid body behaviour. The majority of the collapsed walls happen to be these ones due to a rotation support non-provided by the surrounding RC frame (Fig. 13).

2.2.2 Global failures/mechanisms

Regarding the RC buildings' performance, it was observed several collapses caused by irregular distribution height distribution of the IM walls leading to soft-storey mechanism. The common buildings in Nepal are composed by commercial use in the ground level and residential apartments on the others (Fig. 14a, b). An isolated example of a soft-storey mechanism that lead to the collapse of the 3rd floor of a residential building was also observed (Fig. 14c).

The importance of the openings placements in the IM walls is origin to short-column mechanisms on openings adjacent to columns, increasing their shear stresses, and damages (or even their collapse) as observed in Fig. 15.

3 Dynamic characterization of IM walls in Nepal: in situ ambient vibration tests

3.1 Introduction

The dynamic characterization of the IM walls is a very important tool to calibrate numerical models of such type of elements, evaluate their out-of-plane stiffness and young modulus. It is also important to quantify the variation of natural frequencies of the walls, depending on the geometric dimensions, boundary conditions, existence of openings such for example doors or windows, and last but not the least the level of damage. The out-of-plane performance of the IM walls is a topic of big importance since the collapse of this
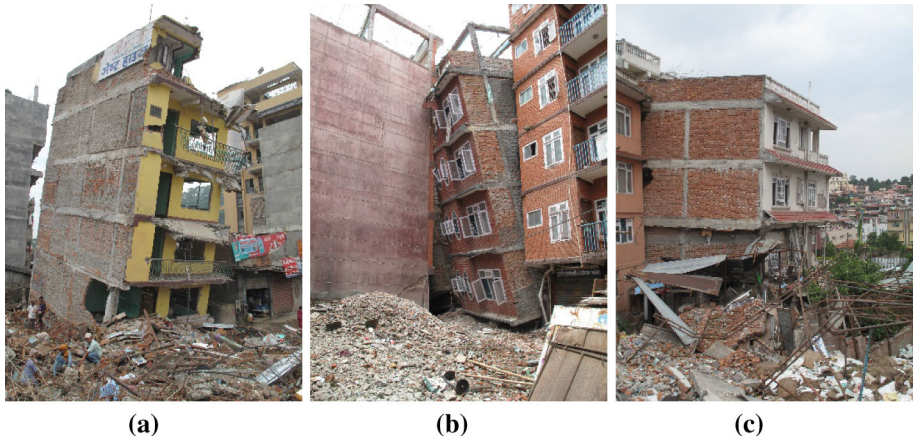
Fig. 14 Examples of RC buildings with soft-storey mechanism: **a** Collapse of the 1st storey, **b** collapse of the 1st storey of both of the buildings and **c** collapse of the 3rd storey—isolated example

Fig. 15 RC buildings subjected to short-column mechanism

elements has been observed as one of the most critical failures observed in the last decade, and can result in catastrophic sequences for the population.

Ambient vibration tests were performed in IM walls of two damaged buildings 2 months after the earthquake, and evaluate the natural frequencies and the vibration modes. The data acquisition was performed by accelerometers PCB Electronics Force Balance ±5 g, through the system cDAQ-9172, from the National Instruments, and using two modules 9234. The acquisition time was conducted during 15 min with a sampling frequency of 2048 Hz. The modal identification was performed through the application of the Enhanced Frequency Domain Composition method (EFDD) in Artemis (ARTeMIS 2009).

Information regarding detailed description of the tested buildings, tests setups are provided together with comparisons between geometric dimensions, openings and level of damaged.

3.2 Non-engineered three-storey building with damages: Bahktapur

3.2.1 Description of the building

The building under study is a non-engineered (NE) three storey damaged building (NE3D) after the Ghorka earthquake, and is located in Bahktapur. A general view of the building is illustrated in Fig. 16a and b and is composed two longitudinal bays and three transversal bays, according to the building's plan of in the building plan in Fig. 16. The first and second natural period of the structure are 0.27 s (transversal direction) and 0.38 s (longitudinal direction) respectively and are indicated in the response spectra of the main shock of 25th April 2015 in Fig. 17. From the visual inspection it was evaluated the level of damage according to EMS-98 (Grünthal 1998) which was classified as Grade 2: Moderate damage. This classification is justified by the observation of cracks in the ground-floor RC

Fig. 16 General view of the NE3D building, **a** front view, **b** lateral view and **c** building plant and localization of the IM walls under study
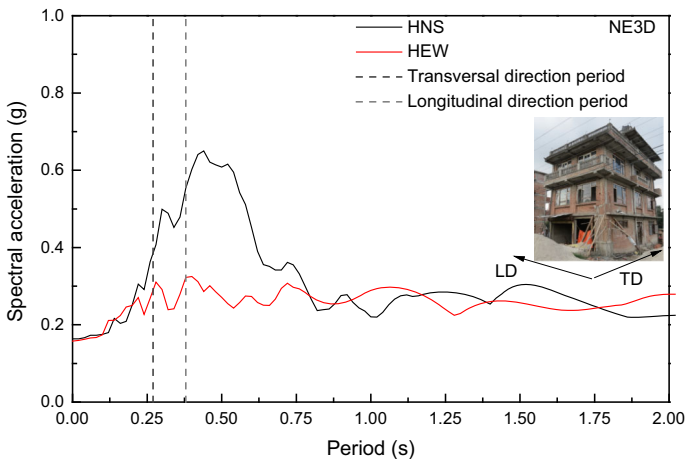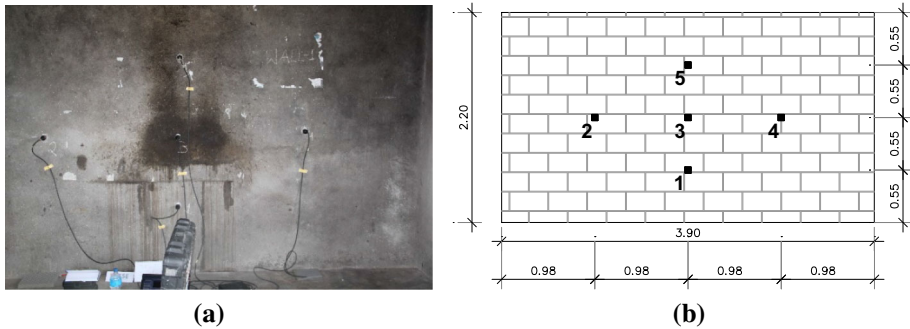

Fig. 17 Response spectra of the main shock ground motion with the fundamental periods of the NE3D building

Table 1 General description of the tested IM walls in Nepal—NE3D

Wall	Dimensions (m)	Damage	Openings
1	3.9 × 2.2	No	No
2	2.7 × 2.2	No	No
3	3.9 × 2.2	No	Yes

Fig. 18 Wall 1 (NE3D) **a** geometric dimensions and **b** test setup

columns and IM walls. During the visual inspection of the building the damaged columns were already repaired.

Three IM walls were selected, all in the 3rd floor of the building. They were selected according to different characteristics, namely: a large and small dimensions (Wall 1 and Wall 2 respectively) and another with the same dimensions as the Wall 1but with a central window (Wall 3), with locations illustrated in Fig. 16c. The tested walls were constructed as described in Sect. 2.1, however it was not observed the use of RC band in the middle of the panels. Table 1 present global information concerning each tested IM wall.

3.2.2 Wall 1: large non-damaged

For the ambient vibration test it was used 5 accelerometers measuring the out-of-plane accelerations of the wall 1, and only one experiment test setup was adopted (Fig. 18a). The scheme is illustrated in Fig. 18b.

Figure 19 illustrates the singular and normalized values curves from the spectral matrix for all the accelerations measured. The identification of the natural frequencies are computed by comparing each peak with the corresponding vibration mode, and checking the agreement between them. The natural frequencies and mean damping coefficient found are presented in Table 2 and in Fig. 20, respectively.

3.2.3 Wall 2: small non-damaged

The ambient vibration test of the Wall 2 was performed by using also 5 accelerometers and an only one test setup (Fig. 21b).

Regarding the experimental test results it is illustrated in Fig. 21a the spectral of single values of spectral density matrices and the vibration modes obtained are expressed in Fig. 22 with the corresponding vibration mode shapes in Fig. 23.
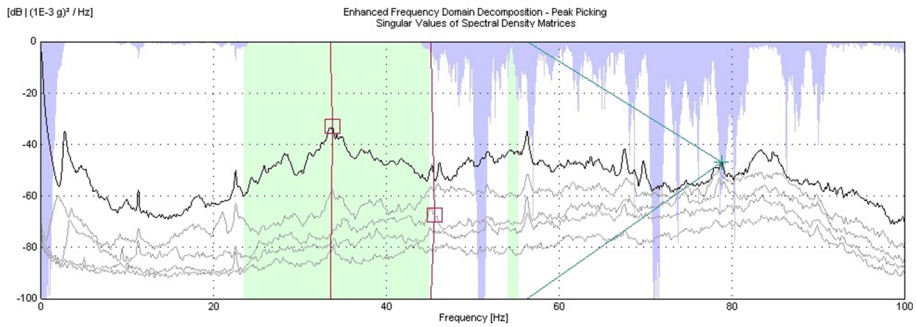
Fig. 19 Spectral of single values of spectral density matrices of the Wall 1—NE3D

Table 2 Experimental modal parameters: Wall 1—NE3D

Vibration mode	Frequency (Hz)	Damping ratio (%)
1	31.83	2.003
2	45.02	0.5108
3	79.91	0.1655

3.2.4 Wall 3: large non-damaged with opening

The Wall 3 has the same geometric dimensions of the Wall 1, with a central window with the dimensions 2 × 1.45 m in the center of the wall as it can be observed in Fig. 24. Only 4 accelerometers were used for the ambient vibration test, which test setup scheme is illustrated in Fig. 24b (Table 3).

Experimental test results are illustrated in Fig. 25, the spectral of single values of spectral density matrices and the vibration modes obtained are expressed in Table 4, and the corresponding vibration mode shapes in Fig. 26.

3.2.5 Conclusions

From the results of ambient vibration tests, it can be stated that the first natural frequency of the larger Wall 1 is unexpectedly 46 % smaller than the corresponding of smaller Wall 2. Regarding the influence of the openings in the variation of the natural frequencies, it was found that the presence of the window reduce 70 % the first frequency of the wall to about 10 Hz, as summarized in Fig. 27. Comparing the out-of-plane frequency of the wall 1 which dimensions are 3.90 × 2.20 m with those obtained by Furtado et al. (2016) for a rectangular infill wall 4.20 × 2.30 m composed by hollow horizontal bricks, typically used in the Southern region of Europe it can be observed that the Nepalese wall is 33 % stiffer. This stiffer characteristics associated with the use of some poor metallic mesh in their construction process justify the reduced number of out-of-plane collapses observed after the Ghorka earthquake.

3.3 Engineered twelve-storey building with damage: Nakhkhu

3.3.1 Description of the building

The second building under study is one of the towers that compose the CityScape Apartments in Nakhkhu, and is an engineered (E) structure with twelve storeys and
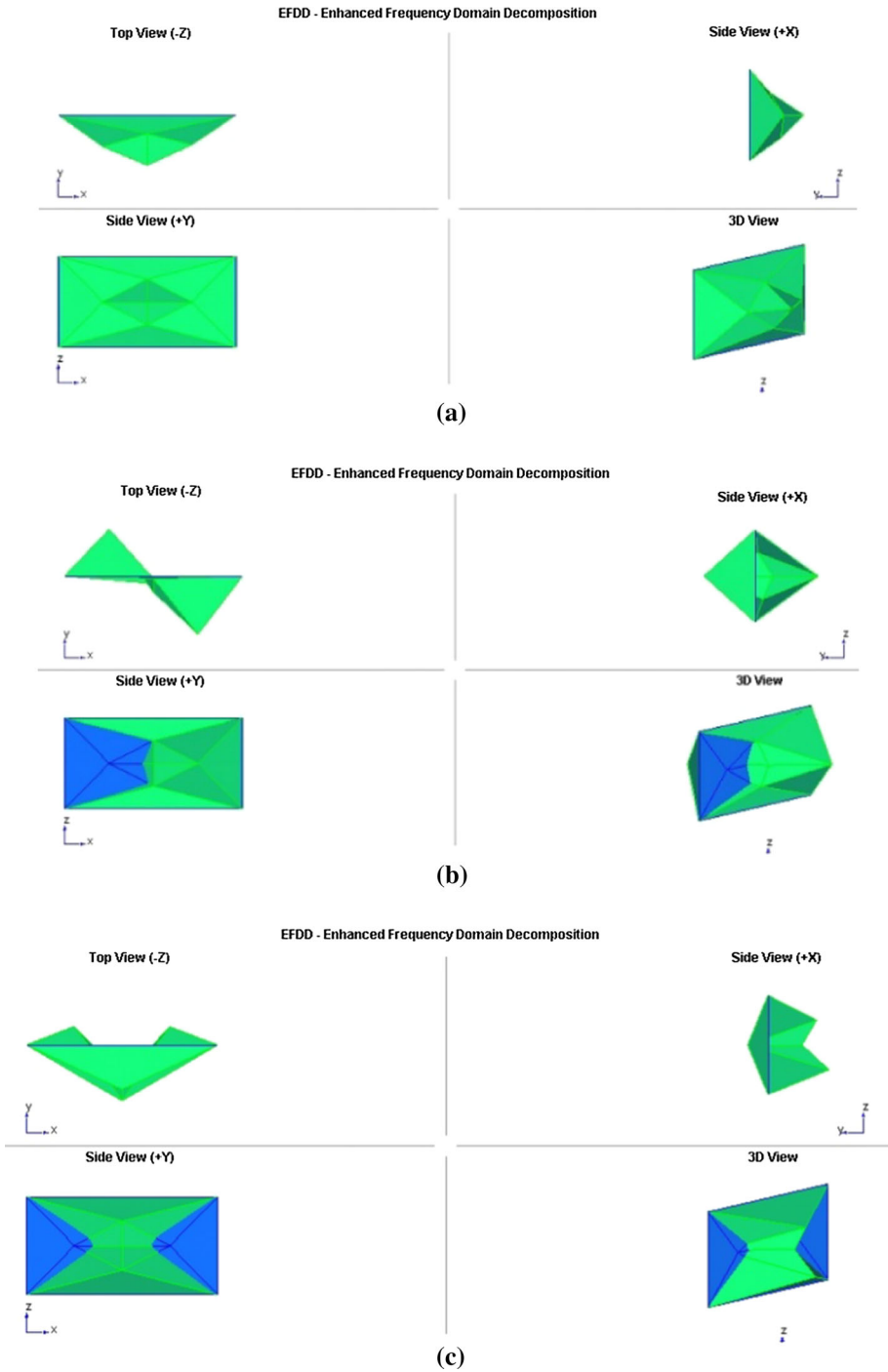
Fig. 20 Wall 1 (NE3D): Natural frequencies and corresponding vibration modes. **a** $f_1 = 31.83$ Hz; **b** $f_2 = 45.02$ Hz and **c** $f_3 = 79.91$ Hz
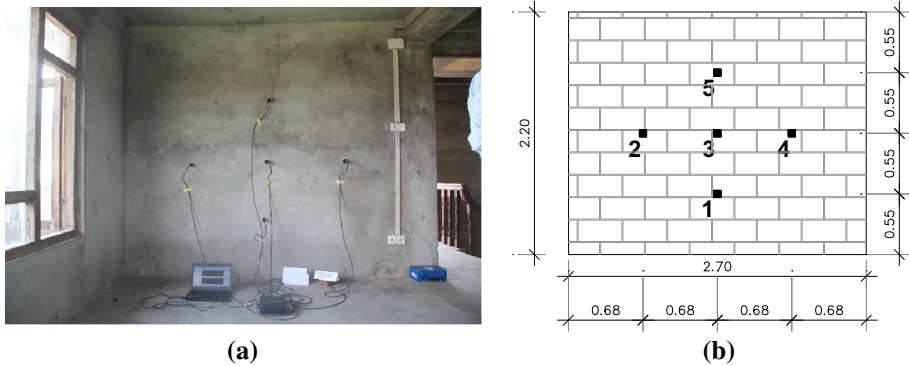
Fig. 21 Wall 2 (NE3D), **a** geometric dimensions and **b** test setup
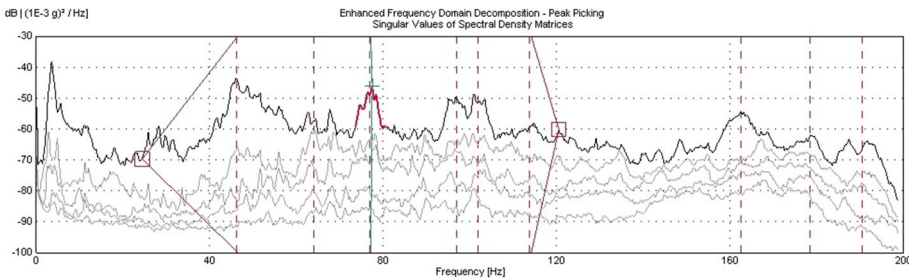

Fig. 22 Spectral of single values of spectral density matrices of the Wall 2—NE3D

damaged (E12D), illustrated in Fig. 28. The CityScape apartments are composed by four high-rise apartment blocks, with different number of storeys (10–15). Tower 3 was selected for this study because it was the one with more damages along the entire building's height. The building plant is illustrated in Fig. 29. The level of damaged observed in this building was classified as Grade 2, according to the EMS-58 (Grünthal 1998) classification, due to the several cracks in the IM walls from the ground-floor until the 6th floor. No significant structural cracks were found in the columns or beams of the building.

The main purpose of the choice of this building was to compare the natural frequencies of IM walls with the same dimensions/properties/construction with and without damage, thus to evaluate the effect of the damage. The first and second natural period of the structure are 0.34 s (transversal direction) and 0.44 s (longitudinal direction) respectively and are indicated in the response spectra of the main shock of 25th April 2015 in Fig. 30.

The Indian Seismic Code IS-1893-1:2002 (IS 1893), in sub-section 7.6.2. propose an empirical equation to estimate the fundamental period of vibration (Ta) of moment-resisting frame buildings with brick infill panels, expressed in Eq. 1:

$$T_a = \frac{0.09 \times h}{\sqrt{d}} \tag{1}$$

where, h is height of the building and d is base dimension of the building at the plinth level, along the considered direction of the lateral force.
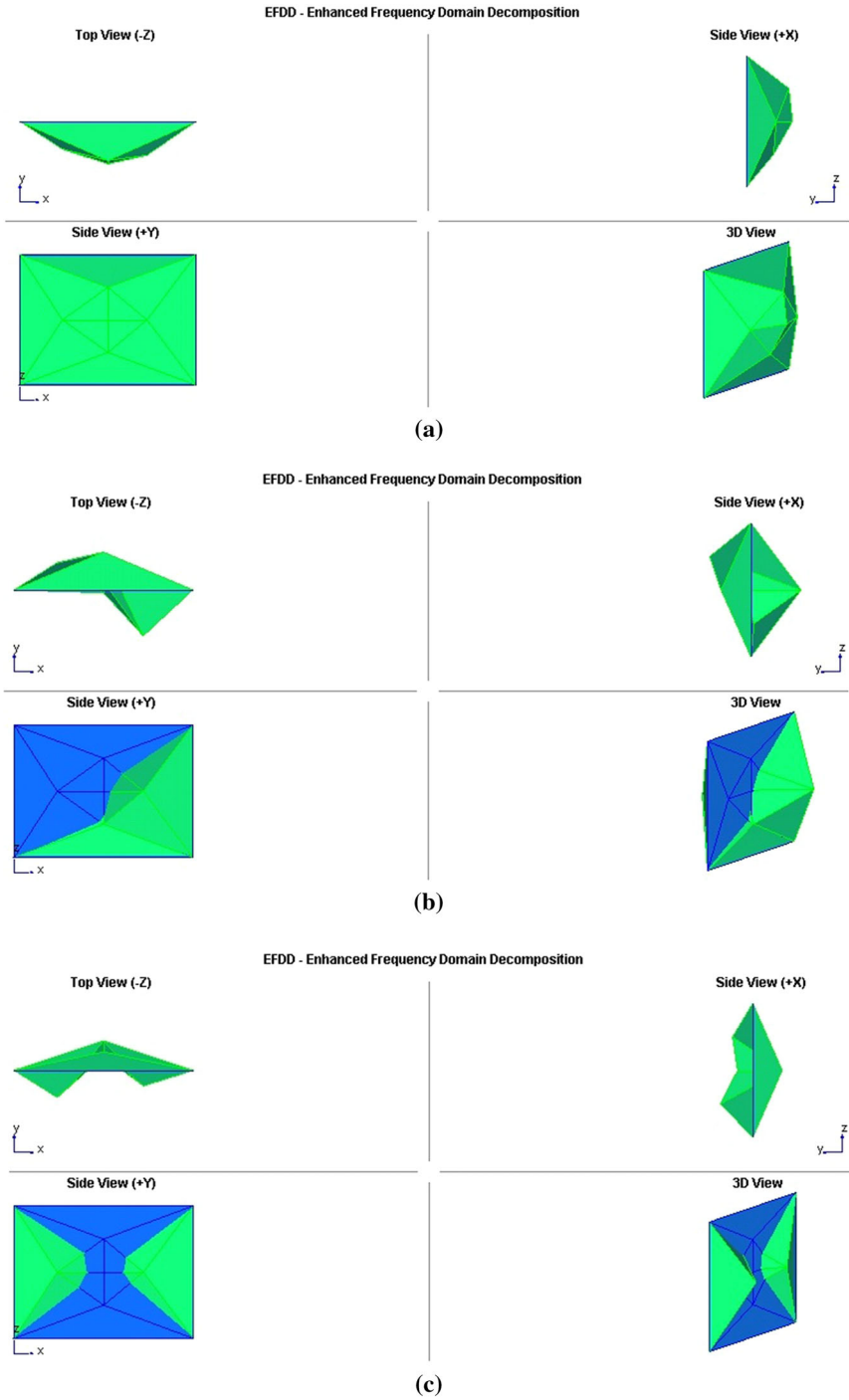
Fig. 23 Wall 2 (NE3D): natural frequencies and corresponding vibration modes. **a** $f_1 = 46.43$ Hz; **b** $f_2 = 77.11$ Hz and **c** $f_3 = 114.10$ Hz
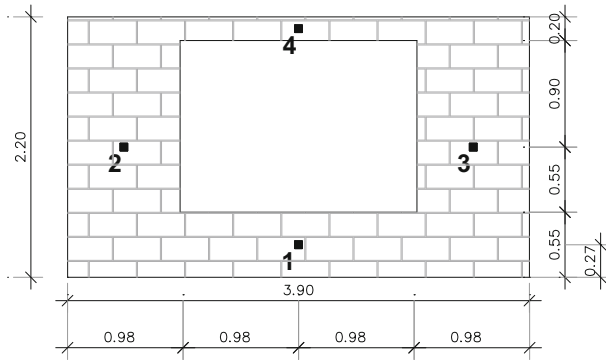
Fig. 24 **a** Geometric dimensions and **b** test setup of the Wall 3—NE3D

Table 3 Experimental modal parameters: IM Wall 2—NE3D

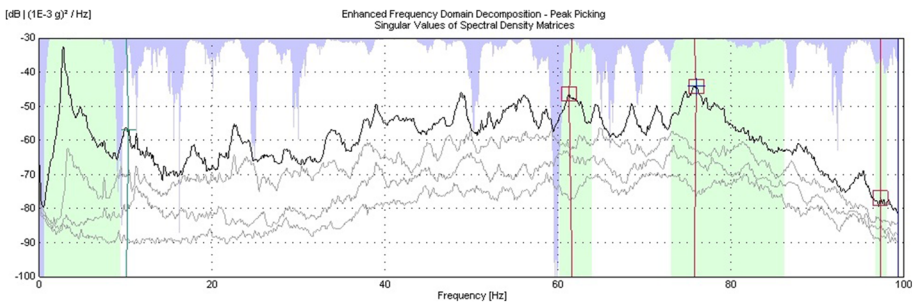| Vibration mode	Frequency (Hz)	Damping ratio (%)
1	46.43	4.931
2	77.11	1.559
3	114.11	1.55

Fig. 25 Spectral of single values of spectral density matrices of the Wall 3—NE3D

Table 4 Experimental modal parameters: Wall 3—NE3D

Vibration mode	Frequency (Hz)	Damping ratio (%)
1	10.07	5.170
2	61.39	1.491
3	135.3	0.129

According to Eq. 1 the fundamental periods obtained for each dimension were 0.57 s and 0.59 s for the transversal and longitudinal direction respectively which are higher than the experimental frequencies determined in situ. This can be due to the irregular geometry of the building where it can be observed that the building is composed by two major blocks connected by the central element (stair cases and RC cores). Additionally it was observed that the RC columns cross-section are significantly robust, such for example in the ground
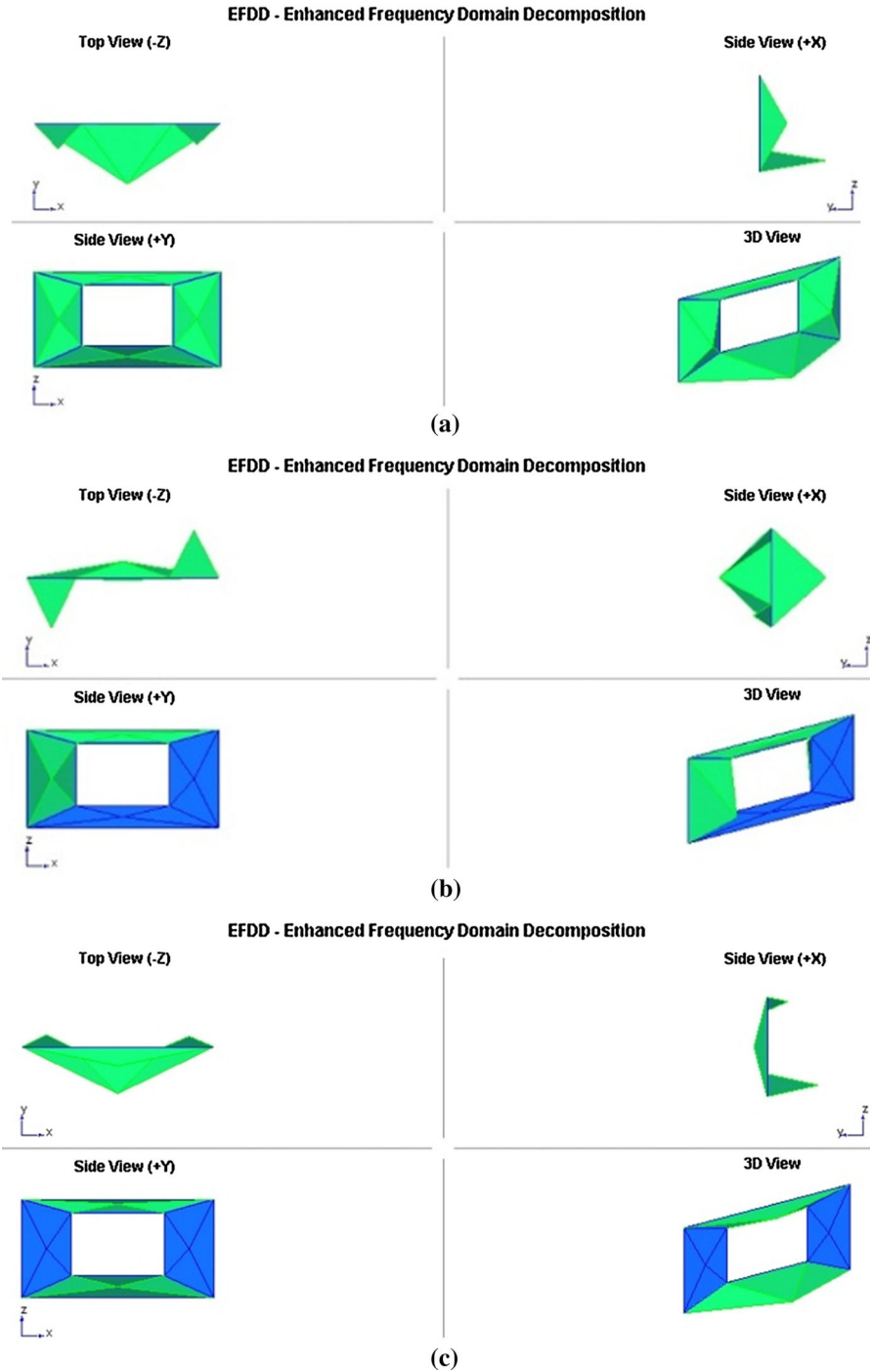
EFDD - Enhanced Frequency Domain Decomposition

Top View (-Z)   Side View (+X)

Side View (+Y)   3D View

(a)

EFDD - Enhanced Frequency Domain Decomposition

Top View (-Z)   Side View (+X)

Side View (+Y)   3D View

(b)

EFDD - Enhanced Frequency Domain Decomposition

Top View (-Z)   Side View (+X)

Side View (+Y)   3D View

(c)

Fig. 26 Wall 3 (NE3D): natural frequencies and corresponding vibration modes. **a** $f_1 = 10.07$ Hz; **b** $f_2 = 61.39$ Hz and **c** $f_3 = 135.30$ Hz
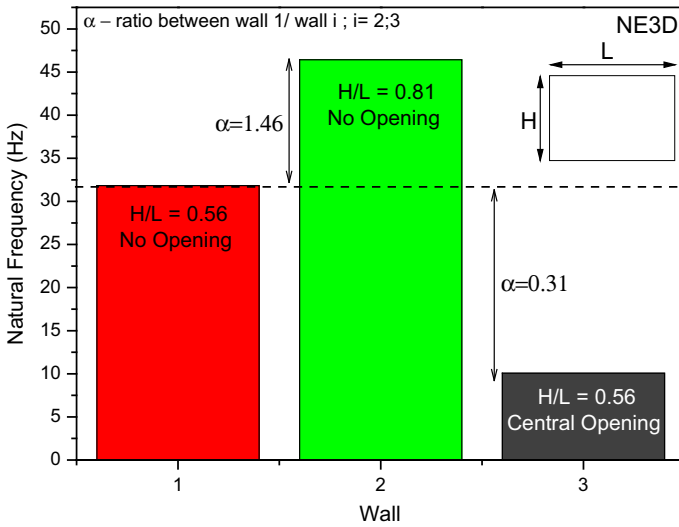
Fig. 27 IM walls natural frequencies obtained from NE3D

Fig. 28 General view of the building E12D. **a** Front view and **b** lateral view

floor the columns cross section are: 900×300, 970×300, 1125×300, 855×300, 1100×300, 800×300, 645×300 mm. This stiffness contribution of the RC elements (columns and cores) with the innumerous IM walls used to partition purposes will increase the stiffness of the building, which besides affected by the damages caused by the earthquake keep the natural periods of the building in each direction lower than the values
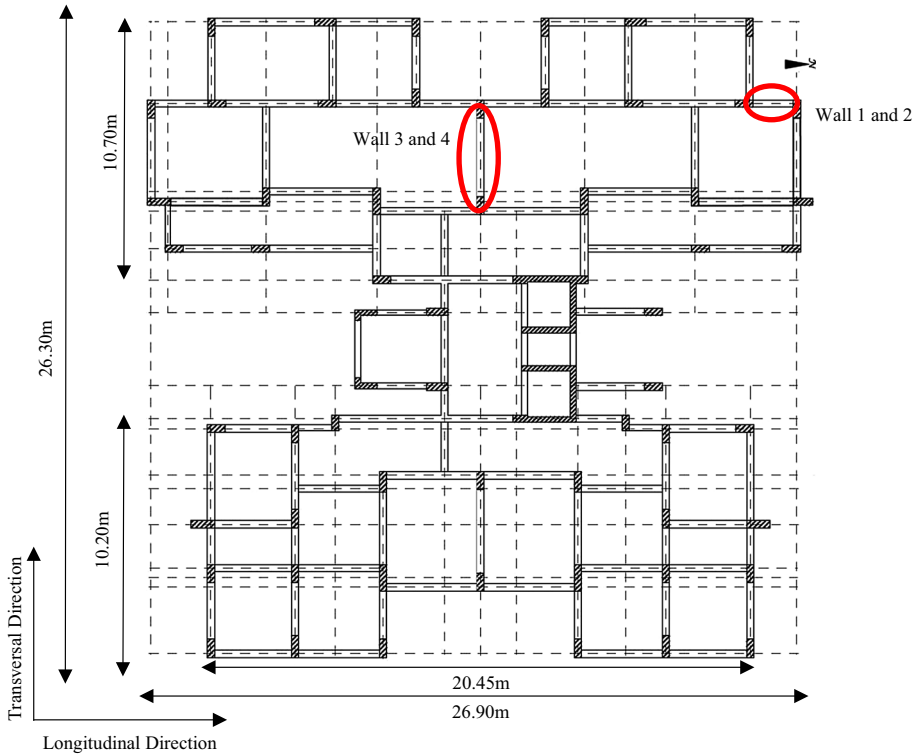
Fig. 29 Building E12D plant and disposition of the IM walls under study (Wall 1 and 3 located at the 6th Floor and Wall 2 and 4 at the ground-floor)
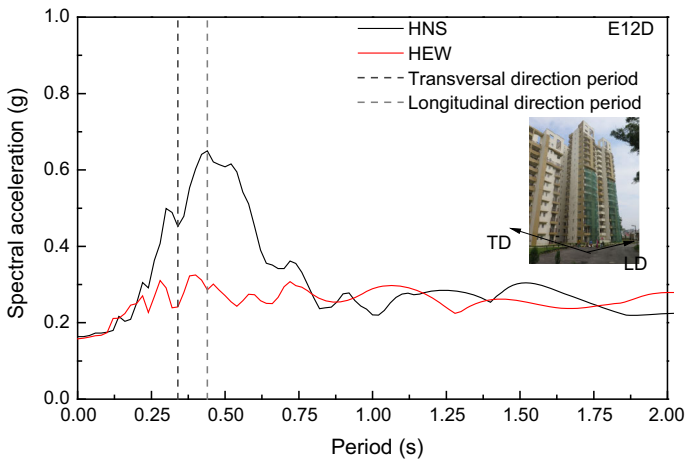

Fig. 30 Response spectra of the main shock ground motion and the fundamental periods of the 12D building

obtained by the empirical equation. In fact, this building can be considered a particular case because the structural design not represent the typical design performed to the RC buildings according to the Indian Codes and due to his geometry.

Four walls were selected according to their dimensions and level of damage. It was selected a small non-damage and damaged wall (Wall 1 and Wall 2 respectively) located the 6th storey and ground-floor respectively. It was also selected non-damaged and damaged (Wall 3 and Wall 4) located also in the 6th storey and ground-floor of the building respectively. From the visual observation of the tested walls, it was observed a discontinuity use of a poor metal mesh in the plaster of the panels. The use of poor metal mesh can increase the out-of-plane performance of the IM walls due to the increase of the out-of-plane stiffness, strength and ductility as observed in some experimental studies (Pereira et al. 2012; Anil et al. 2012). Regarding the ambient vibration tests and taking into account that the use of a poor metal mesh can increase the stiffness of the element, consequently the out-of-plane frequencies of the wall increase when compared with a similar wall with no use of this solution. The RC band was not constructed in the middle of the tested panels. Table 5 present the tests performed in the building E12D followed by a general overview of each wall in Fig. 31.

3.3.2 Wall 1: small and non-damaged

The Wall 1 is a non-damaged specimen, located in the 6th floor of the building with the geometric dimensions of 1.6 × 2.6 m. A setup composed by 8 accelerometers was used to perform the ambient vibration test, and the corresponding disposition is illustrated in Fig. 32.

The results are presented in Fig. 33 and Table 6, as in previous sections.

3.3.3 Wall 2: small and damaged

The Wall 2 has the same geometric dimensions of the Wall 1, but is significantly damaged as it can be observed in Fig. 34a. Visible detachment of the panel from the surrounding RC elements and detachment of the plaster in the center of the panel was observed.

The results are presented in Fig. 35 and Table 7.

3.3.4 IM wall 3: large and non-damaged wall

The IM wall 3 is rectangular wall and non-damaged wall located in the 6th storey, with the geometric dimensions of 3.1 × 2.6 m, and one test setup composed by 5 accelerometers was adopted (Fig. 36).

The results are presented in Fig. 37 and in Table 8.

Table 5 General description of the IM walls tested in the building E12D

Wall	Dimensions (m)	Damage	Openings
1	1.6 × 2.6	No	No
2	1.6 × 2.6	Shear crackling Detachment of the panel	No
3	3.1 × 2.6	No	No
4	3.1 × 2.6	Shear crackling Detachment of the panel	No

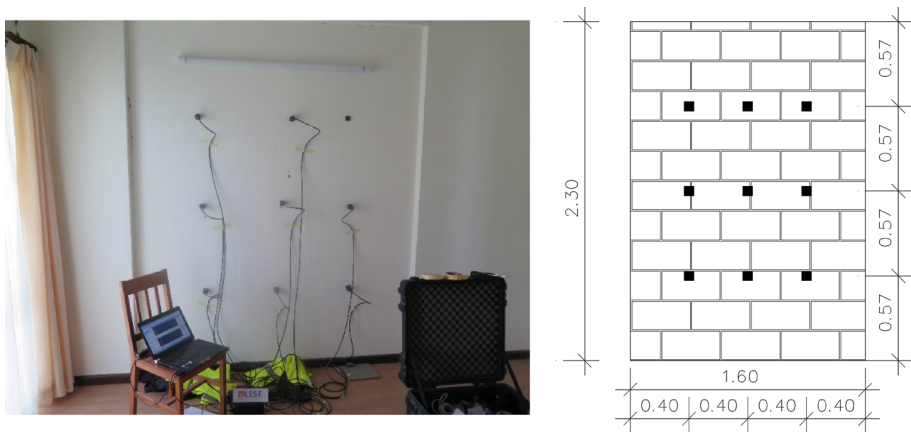Fig. 31 General view of the walls tested in E12D. **a** Wall 1, **b** Wall 2, **c** Wall 3 and **d** Wall 4

Fig. 32 General view of the Wall 1—E12D

3.3.5 Wall 4: large and damaged wall

The Wall 4, located in the ground-floor, has the same dimensions of Wall 3, but with damages Fig. 38a. For the ambient vibration test it was used only 5 accelerometers and only one experiment test setup was adopted (Fig. 38b).
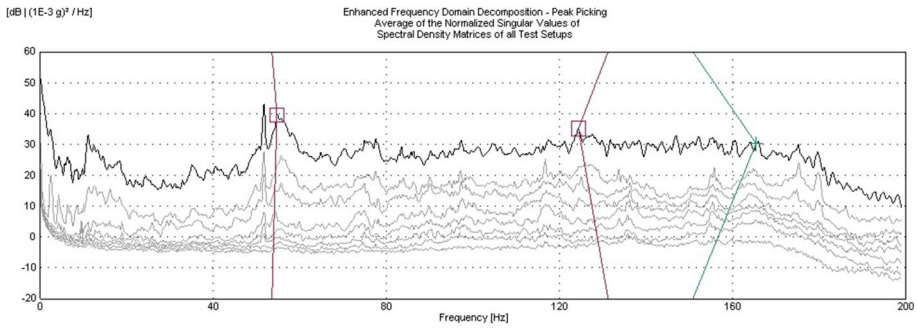
The results are presented in Fig. 39 and in Table 9.

Fig. 33 Spectral of single values of spectral density matrices of Wall 1—E12D

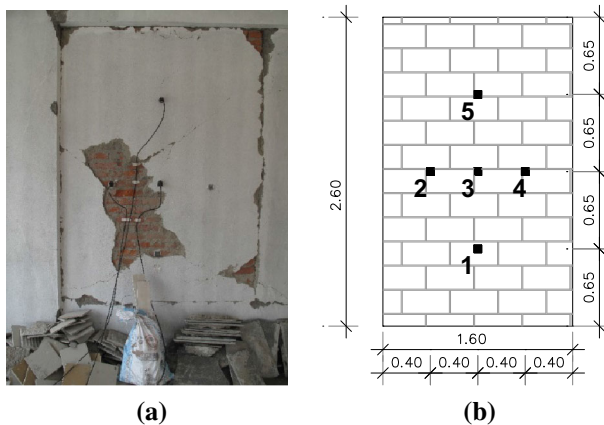| Table 6 Experimental modal parameters of Wall 1—E12D	Vibration mode	Frequency (Hz)	Damping ratio (%)
	1	53.49	1.232
	2	131.10	7.980
	3	150.70	8.928

(a)　　　　　　　　　　　**(b)**

Fig. 34 **a** Geometric dimensions and **b** test setup of the Wall 2- E12D

3.3.6 Conclusions

The Walls tested in the building E12D allowed to assess the impact of damage of the walls on their out-of-plane frequencies. These conclusions can be used on the analysis of the out-of-plane' stiffness reduction and on the fragility of the border constrains. General conclusions can be drawn:

- The first natural frequency of the small IM panels is 20–40 % higher than the observed in the larger panels for the same level of damage;
- The first natural frequency is significantly affected by the damage state, with reductions about 25 % and 50 % respectively for the smaller and larger walls, as observed in Fig. 40;
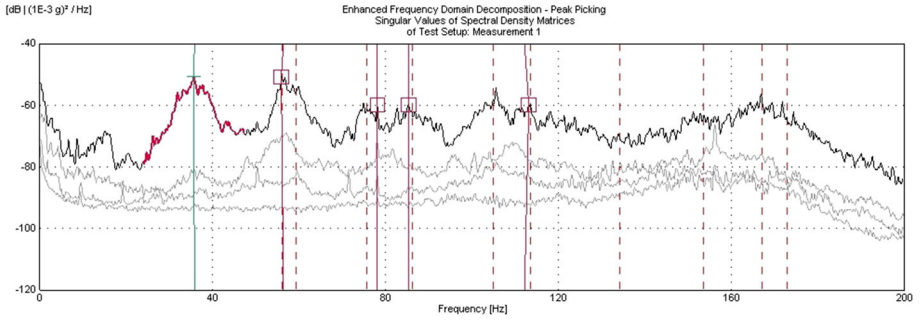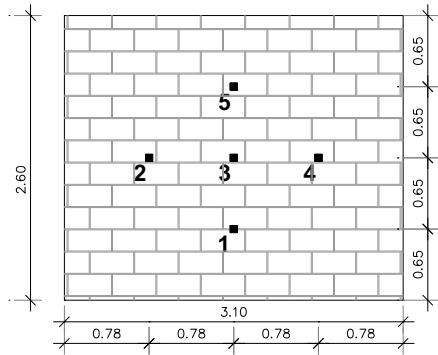
Fig. 35 Spectral of single values of spectral density matrices of Wall 2—E12D

Table 7 Experimental modal parameters of Wall 2—E12D

Vibration mode	Frequency (Hz)	Damping ratio (%)
1	35.74	6.652
2	56.32	1.324
3	78.02	0.232

(a)   (b)

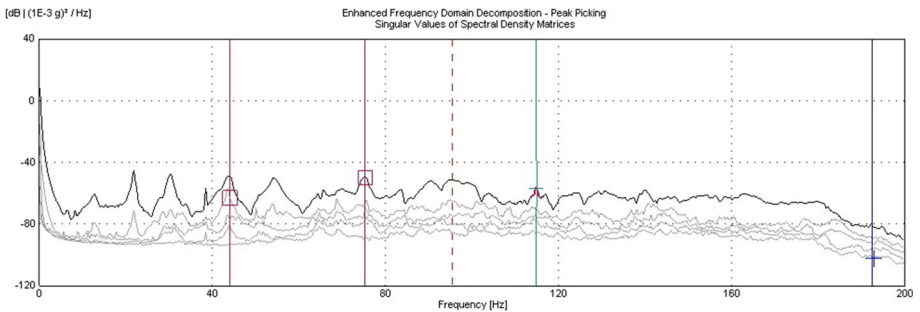**Fig. 36** **a** Geometric dimensions and **b** test setup of the Wall 3—E12D

Fig. 37 Spectral of single values of spectral density matrices of the Wall 3—E12D

Table 8 Experimental modal parameters of Wall 3—E12D	Vibration mode	Frequency (Hz)	Damping ratio (%)
	1	44.03	1.301
	2	75.01	0.784
	3	114.80	0.480

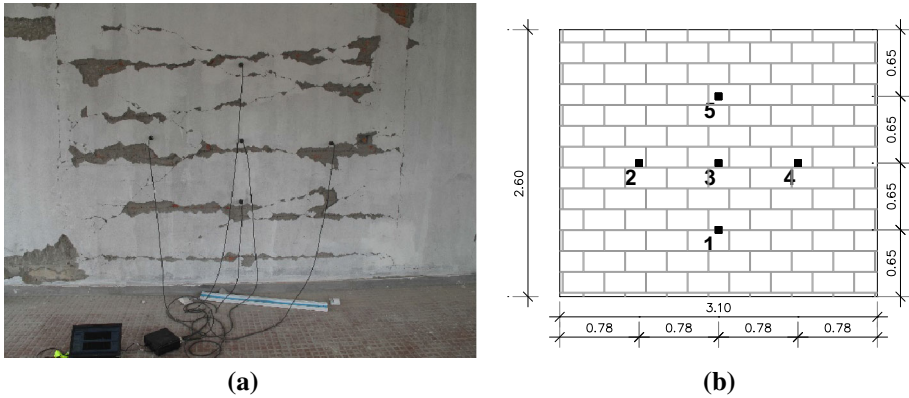**(a)**   **(b)**

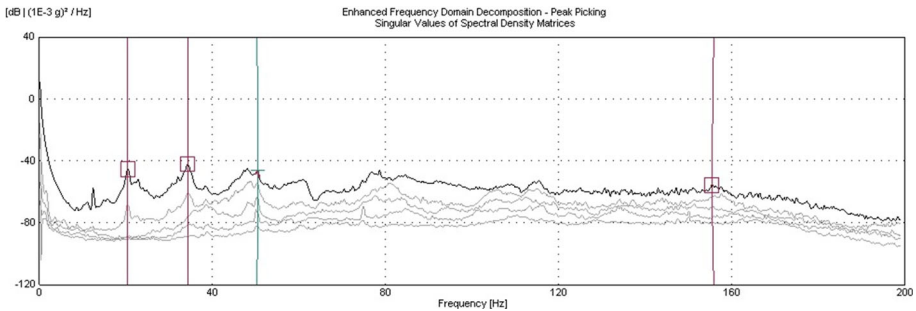**Fig. 38** **a** Geometric dimensions and **b** test setup of the Wall 4

Fig. 39 Spectral of single values of spectral density matrices of Wall 4—E12D

Table 9 Experimental modal parameters of Wall 4—E12D	Vibration mode	Frequency (Hz)	Damping ratio (%)
	1	20.41	1.101
	2	34.35	0.402
	3	50.30	0.602

- Similar conclusions performed in the Sect. 3.3.4 can be drawn here, since all the walls without damage feature higher first natural frequencies values than the ones obtained by Furtado et al. (2016). It appears that the introduction of damage on the panel reduces significantly the flexural stiffness thereof, however factors as the construction technique (described in Sect. 2.1) associated with intermittent use of poor metal mesh
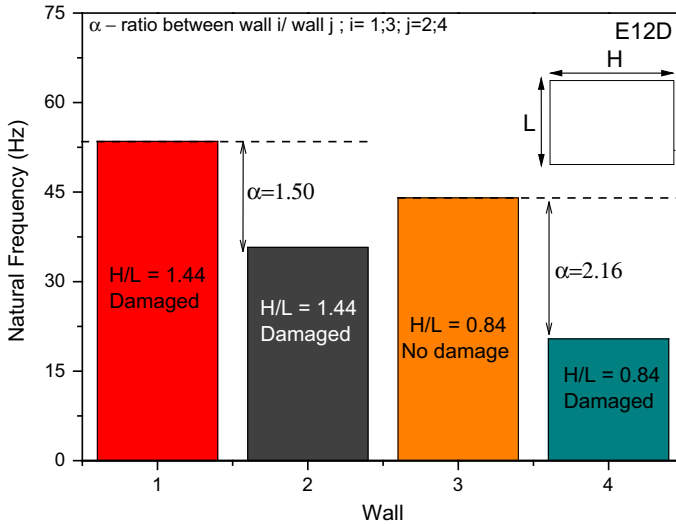
Fig. 40 Walls natural frequencies obtained from E12D

may have contributed to the very small number of out-of-plane collapses these elements.

4 Conclusions

The 25th April 2015 Ghorka earthquake caused significant levels of damaged and destruction to both older and recent constructions made by masonry and/or RC structures. The peak ground accelerations of the main shock response spectra exceeded 0.6 g for in the short-period range which is typical for low-rise infilled RC buildings, which associated with soil amplifications further enhance the observed damage after earthquake. Along the manuscript a survey report of damages observed in Nepalese infilled RC buildings were performed.

Regarding the seismic performance of those buildings, a general analysis based on observation made during a reconnaissance mission indicates that the IM walls played an important rule. The material properties of the infills and their construction process technique contributed to increase significantly the lateral stiffness of the buildings. For the cases of regular distributions such in terms of height or plant, their contribute was positive and no significant damages were observed, however the common practice of use the ground-floor of the buildings for commercial purposes originated vertical stiffness irregularities that was particularly catastrophic by causing several soft-storey mechanisms which lead to the collapse of a significant number of buildings around Kathmandu.

Additionally, ambient vibration tests were performed in seven IM walls of damaged RC buildings with the main goal of evaluate the effect of the geometry, openings and damage in the out-of-plane frequencies of the walls, and consequently gather more information regarding their bending stiffness and dynamic characteristics. This is an important contribute for Nepal since there is no information of experimental studies that determined the infills material and mechanical properties in the Literature.

Three IM walls were studied, assessing the influence of dimensions and openings and was observed that the opening (centre window) reduced significantly the out-of-plane frequency and that the wall with small dimensions have higher out-of-plane frequencies when with larger walls. Another four IM walls were tested with different levels of damaged and dimensions. It was observed that the first natural frequency of the IM walls is significantly affected by the introduction of damaged, which reduces their bending stiffness and changes the infills border constrains due to the detachment of the panel from the surrounding RC frames. Additional experimental tests are needed to validate the data results obtained and to increase the knowledge of the out-of-plane dynamic characteristics of the infill walls, as well as other experimental tests about the others infills properties that allow to understand their seismic behaviour.

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