



Field observations of the structural performance of RC constructions in Nepal after 25th April 2015 earthquake

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Abstract

On April 25 a devastating 7.8 Mw shallow earthquake struck Nepal, causing over 9000 deaths and almost 23000 injuries. More than 400 aftershocks, with magnitude larger higher than 4.0Mw, occurred in the following two months. The earthquake and the aftershocks caused a significant number of damaged and collapsed buildings. In this context the present manuscript pretends to be an overview of the buildings damages observed over the course of 10-days reconnaissance trip that took place 2 months after the earthquake. A general summary of the tectonic region and seismicity characteristics will be provided as well as the reinforced concrete (RC) buildings will be characterized in terms of typologies, and more detailing aspects, with and overview of the constructive practices adopted in Nepal and of the current standards and codes. The structural performance of the RC buildings during the seismic events, particularly the influence of the masonry infill walls in the structural response will be reported as well as the main findings that could relate with other similar constructions in other countries. Additionally, during the reconnaissance activity ambient vibration tests were performed in buildings damaged and not after the Gorkha earthquake, and the global results regarding to the dynamic properties of the buildings will be presented.

Keywords: *Gorkha Nepal earthquake, damage survey reconnaissance, RC buildings, Ambient Vibration tests, Dynamic identification*

1. Introduction

On the April 25th of 2015 a massive earthquake struck with a magnitude of 7.8Mw, with the epicenter located in Barpark, Gorkha's district (Fig. 1a), 75km to the northwest of Kathmandu, and a focus depth of 8.2 km. It caused around 9000 deaths, 23000 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 [1], 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 0.164g and 0.158g along the north-south and east-west direction respectively (Fig. 1a and b).

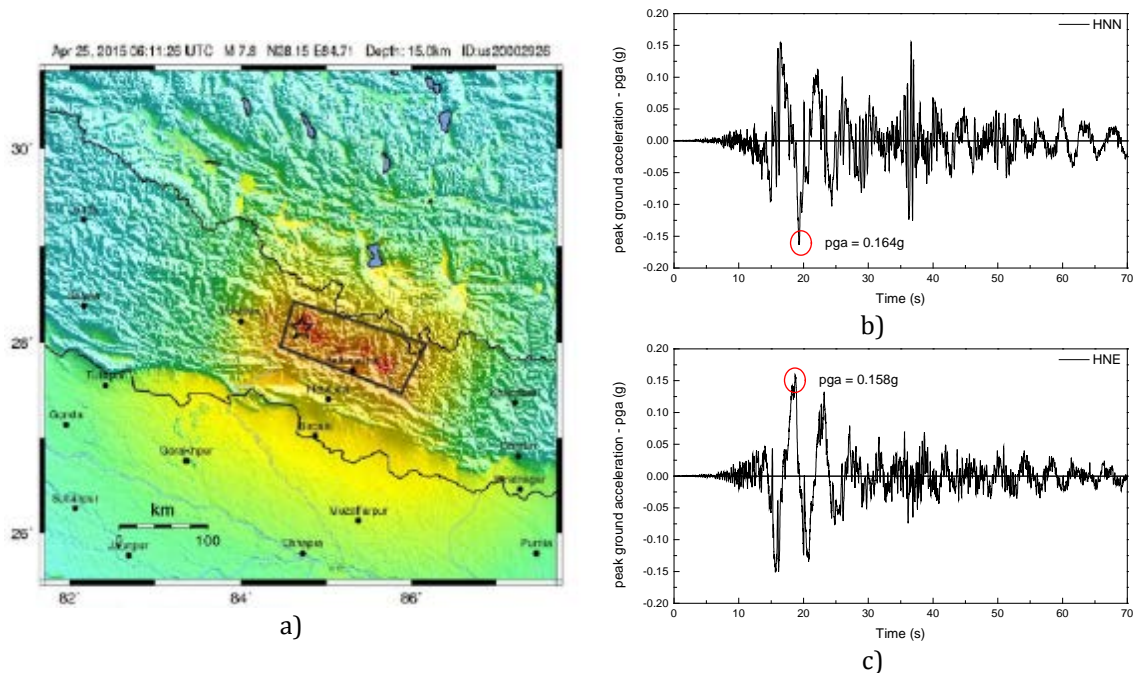


Fig. 1 – Shake intensity map of a) 25th April of 2015; Acceleration of the main shock for b) north-south, and c) east-west direction, according to USGS [2].

On May 12th of 2015, another powerful earthquake struck with a magnitude of 7.3Mw. This occurred in the district of Dolakha with a focus depth of 18km located 75km 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 aftershocks, with magnitude larger than 4Mw including another one of 6.6Mw in Gorkha district, were registered during the following two months. 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 [3, 4].

The present manuscript presents some findings regarding the damage assessment of RC buildings in the regions indicated in Fig. 2 and additionally data were collected within the project for Rapid Post-Disaster Performance Data Collection. The team focused on reinforced concrete (RC) buildings with infill masonry (IM) walls by gathering ambient vibration tests data from affected areas, comparing damages and weaknesses from the observed structures. Due to the technical variety, it was collected data from other types of buildings which will not presented in this paper. Along the manuscript data results from ambient vibration tests on infill masonry walls and in one RC structure will be presented. The numerical modelling of the building will be presented using the

computer software SeismoStruct [5] and will be presented the influence of the presence of the infill panels in the structural response.

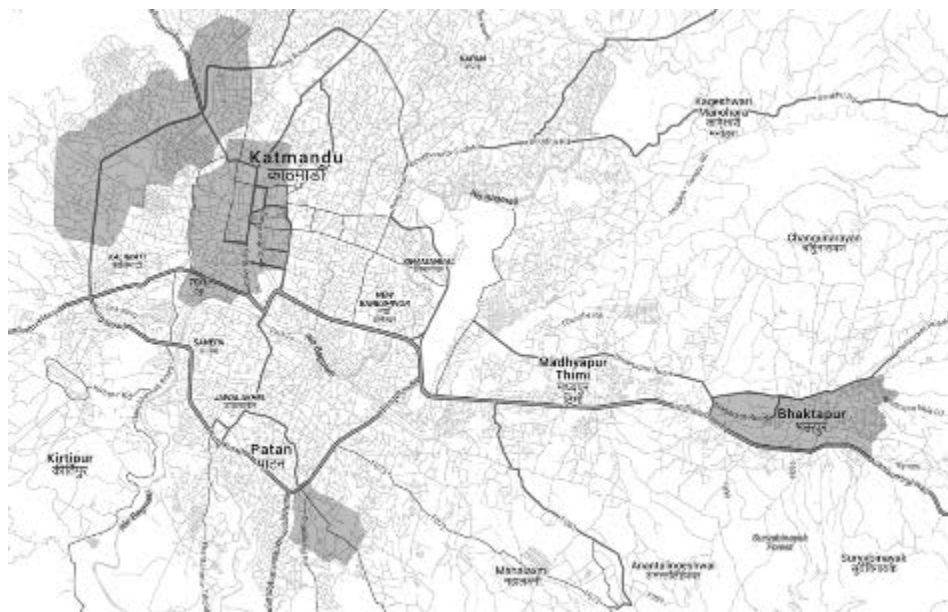


Fig. 2 – Map of Kathmandu Valley highlighting areas from districts of Kathmandu, Bhaktapur and Lalitpur where was focused the Rapid Post-Disaster Performance Data Collection.

2. Rapid assessment of RC structures in Nepal after the Gorkha 25th April earthquake

2.1 Structural typologies

During the past three decades the number of RC buildings in Nepal has increased considerably, mostly built by the landowners or local builders, too dependent on knowledge gain by previous experience which may be too susceptible to insufficient detailing, bad quality of materials or improper assessment of design. Recent studies indicate that the percentage of buildings built by the owners in urban, suburban or rural areas is higher than 80% [6]. These owner-built structures are non-engineered, increasing the level of uncertainty on the vulnerability of buildings to seismic hazards. Although the licensing for construction of new buildings has been introduced in the municipalities during the past decades, these are usually only applied on the architectural side of the project and not for the structural design, perhaps justified by the small number of structural engineers compared with abrupt increase in construction RC buildings in Nepal.

Apart from non-engineered buildings (Fig 3a), there are two different structural types of engineered buildings in Nepal, these in minority compared with the former. They can be categorized as predesigned (Fig 3c) and as well designed (Fig 3d).

The level of vulnerability of the buildings in Nepal to seismic actions has been assessed many years before. Due to impracticability to apply complex structural design for small buildings because of limited resources from owners, two Nepalese standards were introduced in 1994, the NBC-205 [7] for bare-frame RC buildings and the NBC-201 [8] for RC buildings with infilled walls. These standards have the particularity of proposing a new set of general rules and drawings called Mandatory Rules of Thumb (MRT) enabling a pre-seismically-designed for RC buildings, which allow owners to build their structures within some restrictions. The regulations were only officially adopted in 2003.

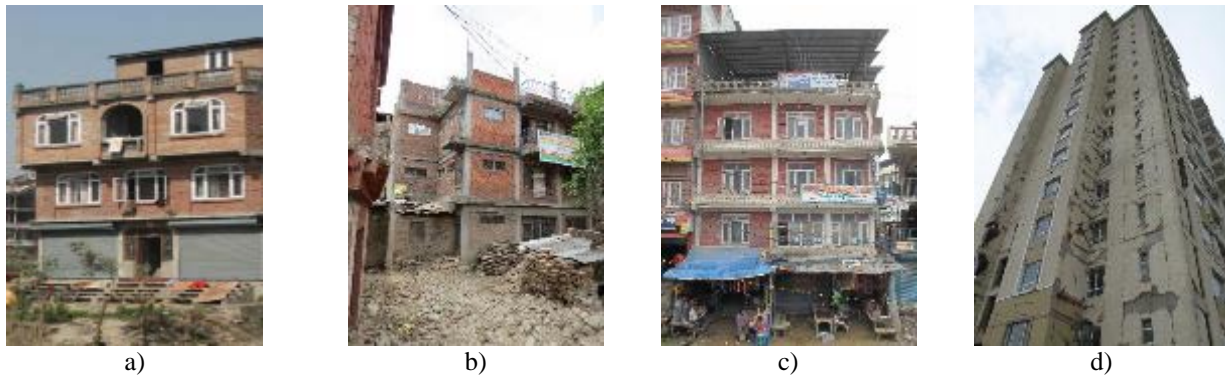


Fig. 3 – Typologies of RC buildings: a) Non-engineered residential with 2-3 storeys, b) Pre-designed residential with 2-3 storeys, c) Pre-designed residential with stores at the ground level with 4-6 floors, and d) Well-designed tall structures.

These pre-designed structures must comply with certain minimum dimensions, constructive detailing for both structural and non-structural elements, and reinforcement percentage, following fixed standard procedures. The typical dimensions for beams and columns varies between 23 and 27cm, and for reinforcement 4x16mm on beams and 4x16mm or 8x12mm on columns. The concrete cover should be around 3cm. The standards require some criteria regarding irregularities in both height and plan, number of floors (maximum of three storeys) and implantation area. The main objective of NBC standards was to provide a simple and cheap tool to build properly designed structures, considering minimum requirements for seismic safety to Nepal at a lower cost to the owners.

The well-designed buildings have detailed structural projects, designed according to the Indian standard IS4326 [9] or by the NBC-105 [10] (based on IS4326). Each one of these structures have their own individual projects, considering more accurate seismic actions and providing higher level on the constructive detailing given to builders, increasing the ductility of structures as a direct consequence [6]. Well-designed structures are usually limited scaled buildings or perform specific sets of functions for the society.

Regarding the observed typologies, most RC buildings have residential occupancy with 3-4 floors (Fig. 3a-b), or 4-6 floors (Fig. 3c) with stores in the ground floor. A few examples of larger structures with 10-18 floors (Fig. 3d) are located in isolated areas. Other RC structures are found serving social purposes with higher investment and quality, such as schools and hospitals.

2.2 Observed damages

The construction of RC buildings has accelerated in the last 15/20 years, in a country dominated by masonry buildings. Although RC structures are generally less vulnerable to earthquake hazards than masonry buildings, the risks involved are higher due to bigger dimensions (occupancy of more families and economic impact) and a more accelerated degradation of strength if poor materials are used.

The main weaknesses are often linked to the quality control of materials (improper vibration of concrete, improper size of the aggregates and steel bars with insufficient ductility) and reduced construction quality (reinforcement detailing and insufficient percentage of reinforcement) which have a direct impact on the bearing capacity on the structural elements, as shown on Fig. 4.

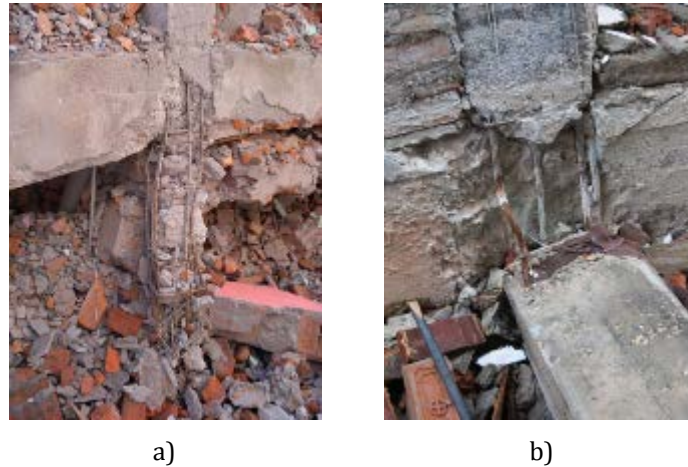


Fig. 4 – Poor quality of materials and construction detailing on a) Columns and b) Joints, with shear failure.

Another influential factor increasing its vulnerability was linked with the reduced amount of IM walls on the ground floor, leading to the formation of soft-storey mechanism, and subsequent partial/total collapse of some buildings, as illustrated in Fig. 5. The failures from soft-storeys are very destructive making these buildings irrecoverable from the retrofitting perspective and also creating damage to adjacent buildings.

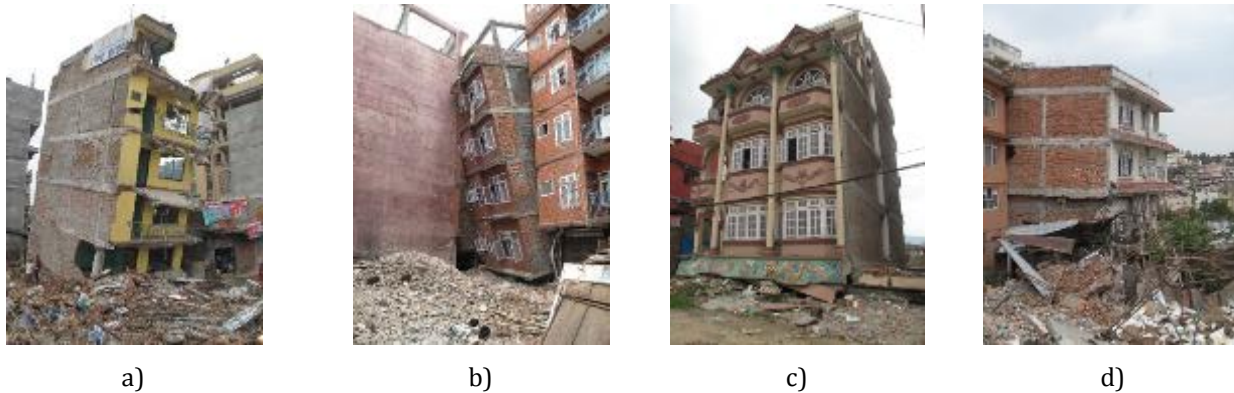


Fig. 5 – Examples of buildings subject to soft-storey mechanism: a) Collapse at the ground floor level, b) Collapse at ground floor and pounding effect, c) Collapse at ground level with side turning, and d) Collapse at the 3rd storey.

It is pointed out that engineered and pre-engineered buildings behaved substantially better compared with the non-engineered ones. However, this observation did not apply to every pre-engineered structure due to the fact that, even though the NBC-201 and NBC-205 limits the height to three storeys, a large number of owners decided to increase the height of the buildings without respecting the standard neither taking measures considering how extra storeys would affect the behaviour, shown on Fig. 6. These type of failures may not be connected to insufficient designing rules but to disregard and human errors. There were verified site-effects of high importance, connected with soil amplifications, that presumably has resulted in areas destroyed with the collapse of the majority of buildings.

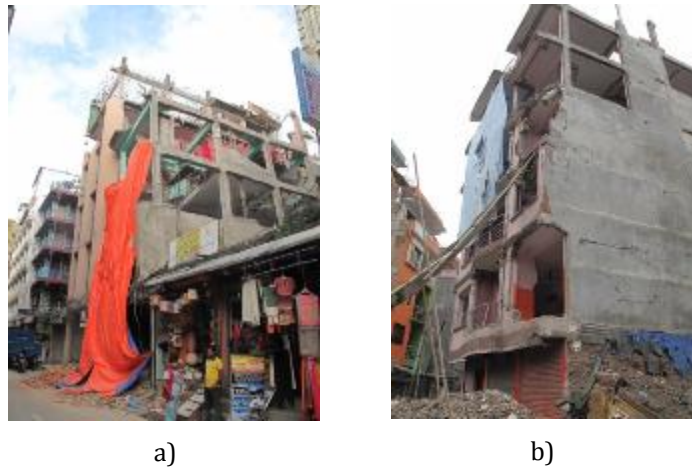


Fig. 6 – Examples of pre-designed a) Adjacent and b) Isolated buildings that have grown in height for which the last storeys are being removed.

The bricks of the infill walls used in Nepal are solid with dimensions of 22, 10 and 5.5 centimetres of length, width and height, respectively. The walls' thicknesses are about 14 and 28 centimetres for interior and exterior, respectively. The exterior walls have a combination of double-leaf and crossed bricks. The mortar cover is about 2 centimetres on each face of the wall. The solid bricks change the behaviour to the infill walls when compared to the hollow bricks used in some countries of Europe. Due to it, these walls have a decreased and slower deterioration of its strength. For this reason, in regular buildings in height, the walls could have saved a large number of buildings.

The majority of damages on IM walls (non-structural elements) were registered as shear-cracking at half height (Fig. 7a), detachment between wall and surrounding RC elements (Fig. 7b), and some with diagonal cracking (Fig. 7c). Given the characteristics of these walls – such as high stiffness provided by the interlock of double-leaf walls of solid bricks –, a reduced number of out-of-plane failures were registered, occurring mainly on the case of some non-confined walls.

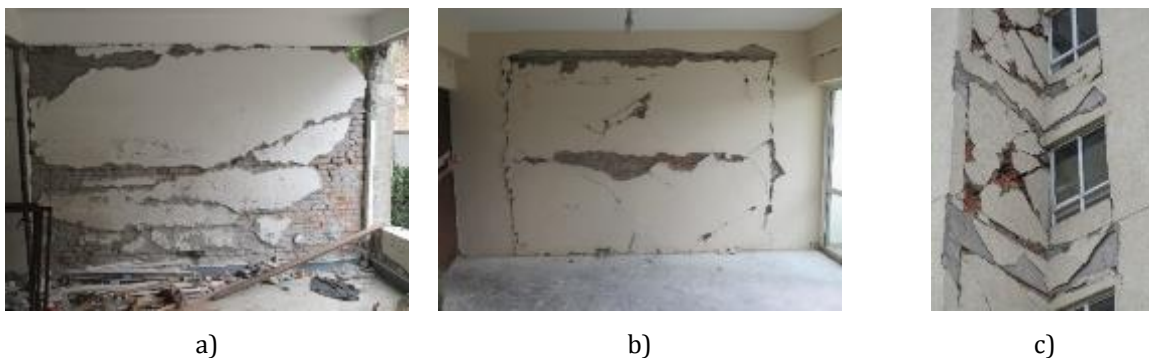


Fig. 7 – Examples of damage in infill masonry walls: a) Shear cracking at half height, b) Detachment of the wall and surrounding RC elements, and c) Diagonal cracking.

Large scale buildings up to 18 storeys filled with IM walls along height, damages to the walls were observed, however in most cases none too little damage occurred on the structural elements. The reduced level of damage suits the proper seismic behaviour of these structures which were designed by the Indian National standards. Although the damage of structural elements was very limited, the consequences of damage to masonry walls meant that many of these buildings had to be closed until repairing of the buildings were completed.

3. Ambient Vibration tests on infill masonry walls

The dynamic characterization of the IM walls is a very important tool to calibrate numerical models of such type of elements, to 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 these elements has been observed as one of the most critical failures observed in the last decade, and can result in catastrophic consequences for the population. Ambient vibration tests were performed in two IM walls of a damaged building, and the natural frequencies and the vibration modes were determined. The data acquisition was performed by accelerometers PCB Electronics Force Balance $\pm 5g$, through the system cDAQ-9172, from the National Instruments, and using two modules 9234. The acquisition time was conducted during 15 minutes 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 [11]. Information regarding detailed description of the tested IM walls, tests setups are provided together with information of the geometric dimensions.

3.1 General view of the case study

The building A under study is a non-engineered three storey damaged building, located in Bhaktapur. A general view of the building is illustrated in Fig. 8a and b, composed two longitudinal bays and three transversal bays, according to the building's plan as illustrated in Fig. 8c. The first and second natural period of the structure are 0.27s (transversal direction) and 0.38s (longitudinal direction) respectively. Two IM walls were selected, all in the 3rd floor of the building. They were selected according to different characteristics, namely: a large wall (Wall 1) and another with the same dimensions as the Wall 1 but with a central window (Wall 2), with locations illustrated in Fig. 8c.

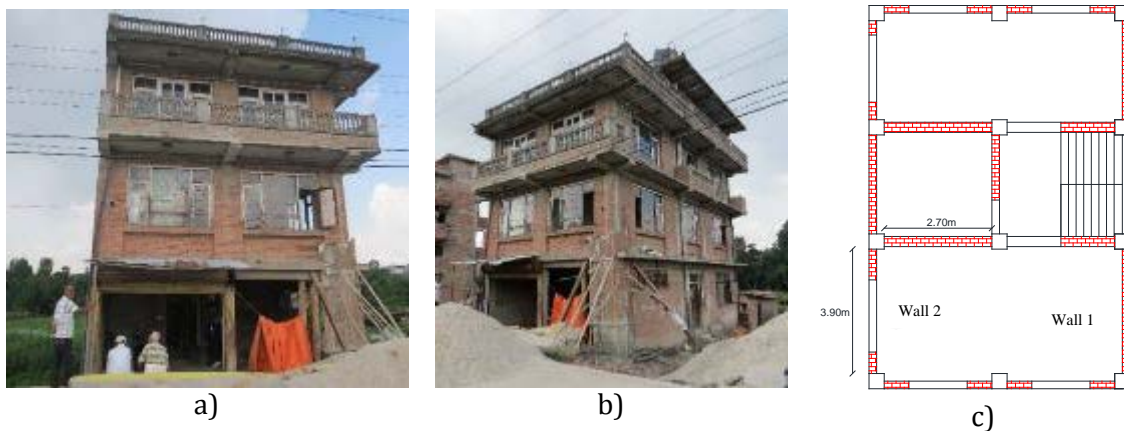


Fig. 8 –General view of the building A under study a) Front view, b) Lateral view and c) Building plant and localization of the IM walls under study.

3.2 Wall 1 – Large without opening and non-damaged wall

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. 9a). The scheme is illustrated in Fig. 9b.

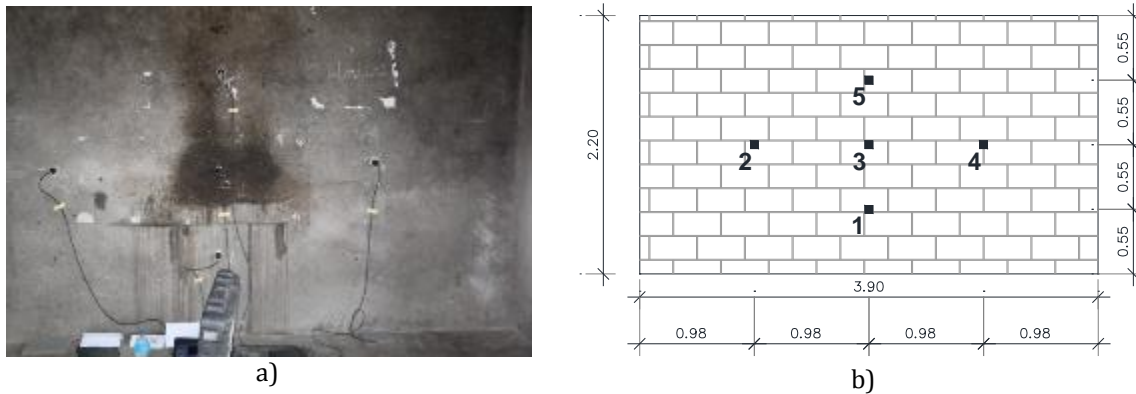


Fig. 9 – Wall 1 a) Geometric dimensions and b) Test setup.

Fig. 10 illustrates the singular and normalized values curves from the spectral matrix for all the accelerations measured. The identification of the natural frequencies is 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 1.

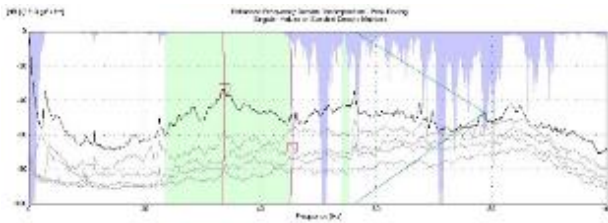


Figure 10 – Spectral of single values of spectral density matrices of the Wall 1.

Table 1 – Experimental modal parameters - Wall 1.

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

3.3 Wall 2 – Large with central opening and non-damaged wall

Wall 2 has the same geometric dimensions of the Wall 1, with a central window with the dimensions 2x1.45m in the centre of the wall as it can be observed in Fig. 11. Only 4 accelerometers were used for the ambient vibration test.

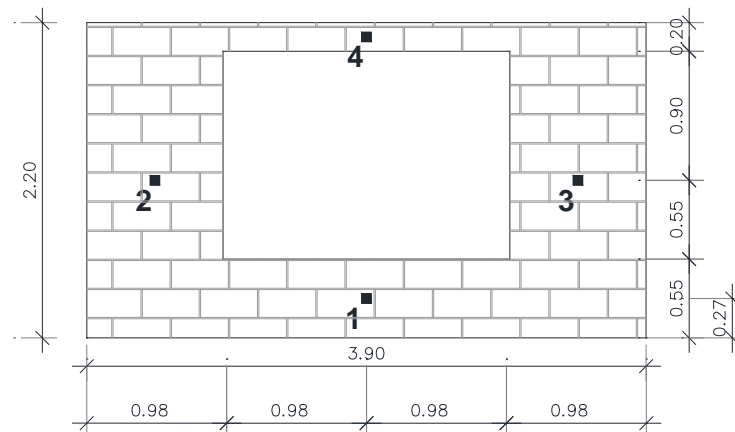


Fig. 11 – Geometric dimensions and test setup of the Wall 2.

The data results are illustrated in Fig. 12, the spectral of single values of spectral density matrices and the vibration modes obtained are expressed in Table 2.

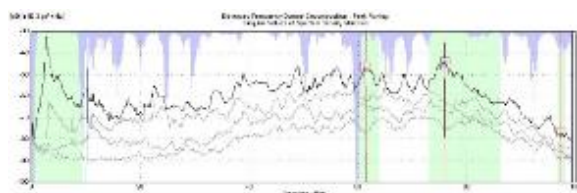


Figure 12 – Spectral of single values of spectral density matrices of the Wall 2.

Table 2 – Experimental modal parameters - Wall 2.

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

3. Modal identification of a bare frame building – Building B

3.1 Description of the building

The studied building is a RC moment-resisting frame (RC-MRF) which is located at south east of Bhaktapur about 600 away from Araniko Highway. The building is bare framed with vertical and mass irregularities. Beam to beam connection without column at those joints causes more vulnerabilities to the building, as illustrated in Fig. 13. The structural irregularity such as structural circular column at ground floor changed to rectangular column in second and third floor, similarly short column in staircase landing are ill practice which are still practice in Nepal. This building in general represents the construction practice prevailed in Nepal before 25th April, 2015 earthquake. The structure is three storey building ageing more than 12 years. The height of each storey is 2.74 m and same for each storey. The maximum span between the columns is 3.5 m and 4.7 in X and Y direction respectively. The rectangular column section used are 300x300 mm, 300x230 mm, 230x230 mm and circular column section of 230x230 mm. The beam is uniform and its dimension is 230x355 mm excluding the slab thickness. The slab used is 125 mm thick throughout the building.

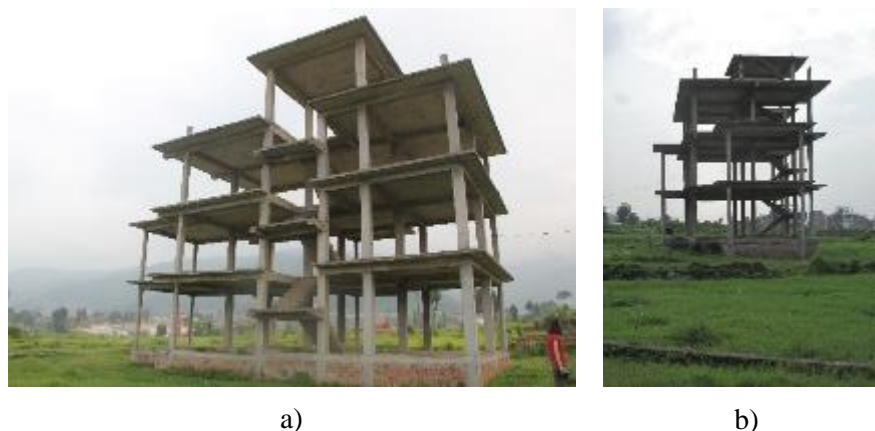
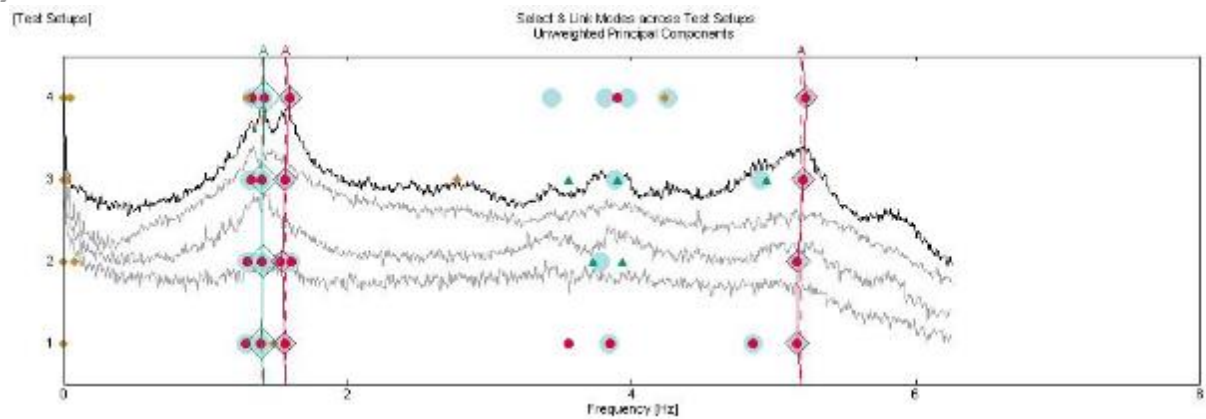


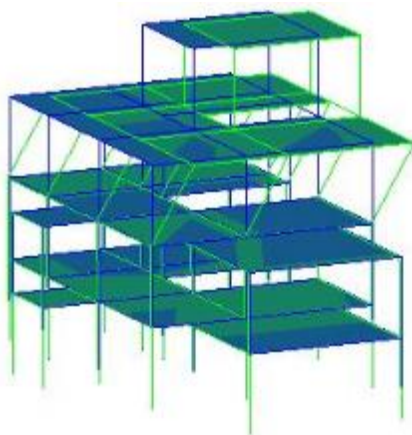
Fig. 13 – Building B: a) General view; and b) Lateral view.

3.2 Modal identification

The modal identification for the bare frame building was performed using computer program Artemis [11]. The seismograph device was used for ambient vibration test starting from top floor, 1 device at the centre as reference instrument and other at diagonal ends of corner column to observe the torsion of the building. The device was setup at each floor with respect to reference one. The main results are illustrated in Fig. 14, namely the experimental fundamental frequency of the building were found to be $f_1=1.404$ Hz and $f_2=1.575$ Hz.



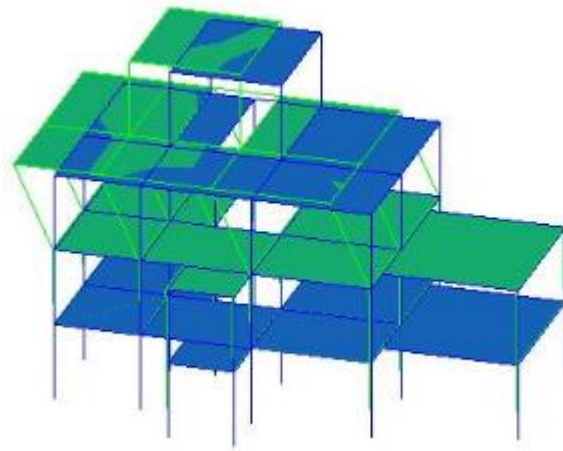
a)



Modal Values

Frequency = 1.4 Hz
Damping = 1.755 %

b)



Modal Values

Frequency = 1.561 Hz
Damping = 2.058 %

c)

Fig. 14 – Building B: a) Spectral of single values of density matrices; b) 1st natural frequency and mode shape; and c) 2nd natural frequency and mode shape.

Non-destructive concrete compressive strength was carried out using smith hammer test at site. The average compressive strength for beam and column was found to be 20 MPa and 15 MPa for slab after codal correction. Since, Schmidt hammer test is carried out at concrete plain surface so thus obtained result may not represent the global behaviour of the building. Poor workmanship and already this building faces two big earthquakes, taking due consideration the grade of concrete is taken as 15 MPa for beam and column for building analysis. The grade of the steel is taken as 415 MPa (yield strength of steel). The modulus of elasticity of concrete as per IS 456 (2000) is $E_c = 5000 \cdot \sqrt{f_{ck}}$ MPa, where f_{ck} is 28 days characteristic cube concrete. Poisson's ratio (μ) and unit weight (γ) for concrete are respectively taken as 0.2 and 24 kN/m³.

3.3 Numerical study of the influence of the IM walls in the structural response

The building B was modelled in the computer software SeismoStruct [5] as three dimensional frame building. Beam and column elements were modelled as inelastic force-based frame element. The mass of the structure was assigned namely the self-weight of the slabs, stairs and the remaining beam and columns. The numerical frequencies that were obtained were 1.42Hz and 1.63Hz which was very similar to the experimental ones.

A numerical model considering the presence of the infill walls was developed and the infill panels were modelled following the Crisafulli's model [12]. The brick infill were assigned namely the external wall thickness of 230

mm and internal wall thickness of 115 mm and the remaining properties suggested by Chaulagain [13, 14]. The visual numerical models are displayed in Fig. 15. From the comparison of the frequencies it was obtained a significant increase of the frequencies for the infill model. The contribution of the IM walls increase almost 5 times in each direction the corresponding natural frequencies, which allow to understand the importance of the non-structural elements in the building response.

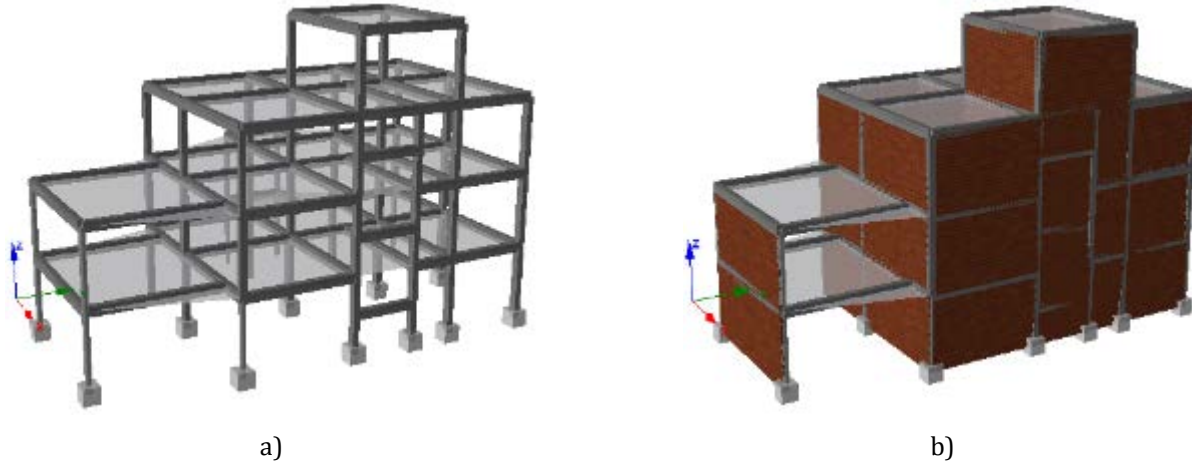


Fig. 15 – Building B numerical models: a) Bare Frame; b) Infill.

Table 3 – Numerical Frequencies of the Bare Frame and infill models.

Model	1 st Frequency (Hz)	2 nd Frequency (Hz)
(Bare Frame - Experimental)	1.40	1.58
Bare Frame	1.42	1.63
Infill	5.86	5.99

5. 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. 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 role. 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 contribution 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.

Additionally, ambient vibration tests were performed in two IM walls of damaged RC buildings with the main goal of evaluating the effect of the openings in the out-of-plane frequencies of the walls, and consequently gather more information regarding their bending stiffness and dynamic characteristics. In fact, the opening reduced significantly the out-of-plane frequency. Additionally, it was presented a dynamic characterization test was performed in a bare frame building and it was evaluated numerically that the presence of the IM walls could increase up to 5 times higher natural frequencies.

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